Gratitude

In appreciation and gratitude
to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence
& Aviation and Inspector General

For their continuous support and gracious consideration,
the Saudi Building Code National Committee (SBCNC)
is honored to present the first issue of
the Saudi Building Code (SBC).

Saudi Building Code Requirements

201	Architectural	
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PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11^{th} June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Structural requirements for Loads and Forces (SBC 301) were developed based on the standards of the American Society of Civil Engineers (SEI/ASCE). The American Society of Civil Engineers, through its Structural Engineering Institute (ASCE/SEI), grants permission to the SBCNC to utilize as reference ASCE 7-02 and ASCE 7-05 in the SBC and to include within the SBC provisions and materials from ASCE 7-02 and ASCE 7-05 modified by SBCNC. ASCE/SEI is not responsible for any modifications or changes that SBCNC has made to the provisions to accommodate local conditions.

The development process of SBC 301 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made and the most important one was adding the seismic contour maps for Saudi Arabia and some parts and items relating to seismic design outside the intensity of the seismic belt of the Kingdom have been deleted. Only SI Units were used through out the Code.

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CHAPTER 1
GENERAL

SECTION 1.1
SCOPE

1.1.0 The Saudi Building Code for Loading referred to as SBC 301 provides minimum load requirements for the design of buildings and other structures. Loads and appropriate load combinations, which have been developed to be used together, are set forth for strength design and allowable stress design. For design strengths and allowable stress limits, design specifications for conventional structural materials used in buildings and modifications contained in SBC 301 shall be followed.

SECTION 1.2
DEFINITIONS

1.2.0 The following definitions apply to the provisions of the entire SBC 301.

Allowable Stress Design. A method of proportioning structural members such that elastically computed stresses produced in the members by nominal loads do not exceed specified allowable stresses (also called working stress design).

Authority Having Jurisdiction. The organization, office, or individual charged with the responsibility of administering and enforcing the provisions of this code.

Buildings. Structures, usually enclosed by walls and a roof, constructed to provide support or shelter for an intended occupancy.

Design Strength. The product of the nominal strength and a resistance factor.

Essential Facilities. Buildings and other structures that are intended to remain operational in the event of extreme environmental loading from wind, or earthquakes.

Factored Load. The product of the nominal load and a load factor.

Hazardous Material. Chemicals or substances classified as a physical or health hazard whether the chemicals or substances are in a usable or waste condition.

Health Hazard. Chemicals or substances classified by the authority having jurisdiction as toxic, highly toxic, or corrosive.

Limit State. A condition beyond which a structure or member becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

Load Effects. Forces and deformations produced in structural members by the applied loads.

Load Factor. A factor that accounts for deviations of the actual load from the nominal load, for uncertainties in the analysis that transforms the load into a load effect, and for the probability that more than one extreme load will occur simultaneously.

Loads. Forces or other actions that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude. All other loads are variable loads. (see also nominal loads.)

Nominal Loads. The magnitudes of the loads specified in Chapter 3 through 13 (dead, live, soil, wind, rain, flood, and earthquake) of SBC 301.

Nominal Strength. The capacity of a structure or member to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Occupancy. The purpose for which a building or other structure, or part thereof, is used or intended to be used.

Other Structures. Structures, other than buildings, for which loads are specified in this code.

P-Delta Effect. The second-order effect on shears and moments of frame members induced by axial loads on a laterally displaced building frame.

Physical Hazard. Chemicals or substances in a liquid, solid, or gaseous form that are classified by the authority having jurisdiction as combustible, flammable, explosive, oxidizer, pyrophoric, unstable (reactive), or water reactive.

Resistance Factor. A factor that accounts for deviations of the actual strength from the nominal strength and the manner and consequences of failure (also called strength reduction factor).

Strength Design. A method of proportioning structural members such that the computed forces produced in the members by the factored loads do not exceed the member design strength (also called load and resistance factor design).

Temporary facilities. Buildings or other structures that are to be in service for a limited time and have a limited exposure period for environmental loadings.

SECTION 1.3
CONSTRUCTION DOCUMENTS

1.3.1 **General.** Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets fully dimensioned. The design loads and other information pertinent to the structural design required by Sections 1.3.1.1 through 1.3.1.7 shall be clearly indicated on the construction documents for parts of the building or structure.

1.3.1.1 **Floor Live Load.** The uniformly distributed, concentrated and impact floor live load used in the design shall be indicated for floor areas. Live load reduction of the uniformly distributed floor live loads, if used in the design, shall be indicated.

1.3.1.2 **Roof Live Load.** The roof live load used in the design shall be indicated for roof areas (Section 4.9).

1.3.1.3 **Wind Design Data.** The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral-force-resisting system of the building:

1. Basic wind speed (3-second gust), km/hr.
2. Wind importance factor, I, and building category.
3. Wind exposure, if more than one wind exposure is utilized, the wind exposure and applicable wind direction shall be indicated.
4. The applicable internal pressure coefficient.
5. Components and cladding. The design wind pressures in terms of kN/m^2 to be used for the design of exterior component and cladding materials not specifically designed by the registered design professional.

1.3.1.4 **Earthquake Design Data.** The following information related to seismic loads shall be shown, regardless of whether seismic loads govern the design of the lateral-force-resisting system of the building:

1. Seismic Occupancy importance factor, I.
2. Mapped spectral response accelerations S_S and S_1.
3. Site class.
4. Design spectral response accelerations S_{DS} and S_{D1}.
5. Seismic design category.
6. Basic seismic-force-resisting system(s).
7. Design base shear.
8. Seismic response coefficient(s), C_S.
9. Response modification factor(s), R.
10. Analysis procedure used.

1.3.1.5 **Flood Load.** For buildings located in flood hazard areas as established in Section 5.3, the following information shall be shown, regardless of whether flood loads govern the design of the building:

1. In flood hazard areas not subject to high-velocity wave action, the elevation of proposed lowest floor, including basement.
2. In flood hazard areas not subject to high-velocity wave action, the elevation to which any nonresidential building will be dry flood-proofed.
3. In flood hazard areas subject to high-velocity wave action, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including basement.

1.3.1.6 **Special Loads.** Special loads that are applicable to the design of the building, structure or portions thereof shall be indicated along with the specified section of this code that addresses the special loading condition.

1.3.1.7 System and Components Requiring Special Inspections for Seismic Resistance. Construction documents or specifications shall be prepared for those systems and components requiring special inspection for seismic resistance as specified in SBC 302 by the registered design professional responsible for their design and shall be submitted for approval in accordance with SBC administrative code. Reference to seismic provisions in lieu of detailed drawings is acceptable.

1.3.2 Restrictions on Loading. It shall be unlawful to place, or cause or permit to be placed, on any floor or roof of a building, structure or portion thereof, a load greater than is permitted by these requirements.

1.3.3 Live Loads Posted. Where the live loads for which each floor or portion thereof of a commercial or industrial building is or has been designed to exceed 2.50 kN/m^2, such design live loads shall be conspicuously posted by the owner in that part of each story in which they apply, using durable signs. It shall be unlawful to remove or deface such notices.

1.3.4 Occupancy Permits for Changed Loads. Construction documents for other than residential buildings filed with the building official with applications for permits shall show on each drawing the live loads per square meter (m^2) of area covered for which the building is designed. Occupancy permits for buildings hereafter erected shall not be issued until the floor load signs, required by Section 1.3.3, have been installed.

SECTION 1.4
BASIC REQUIREMENTS

1.4.1 General. Building, structures and parts thereof shall be designed and constructed in accordance with strength design, load and resistance factor design, allowable stress design, empirical design or conventional construction methods, as permitted by the applicable material chapters.

1.4.2 Strength. Buildings and other structures, and all parts thereof, shall be designed and constructed to support safely the factored loads in load combinations defined in this document without exceeding the appropriate strength limit states for the materials of construction. Alternatively, buildings and other structures, and all parts thereof, shall be designed and constructed to support safely the nominal loads in load combinations defined in this document without exceeding the appropriate specified allowable stresses for the materials of construction.

Loads and forces for occupancies or uses not covered in this chapter shall be subject to the approval of the building official.

1.4.3 Serviceability. Structural systems, and members thereof, shall be designed to have adequate stiffness to limit deflections, lateral drift, vibration, or any other deformations that adversely affect the intended use and performance of buildings and other structures. See Section 10.12 for drift limits applicable to earthquake loading.

1.4.3.1 Deflections. The deflections of structural members shall not exceed the more restrictive of the limitations of Sections 1.4.3.2 through 1.4.3.4 or that permitted by Table 1.4-1.

TABLE 1.4-1: DEFLECTION LIMITS[a, b, c, g,]

Construction	L	W^e	$D+L^f$
Roof members[d] Supporting plaster ceiling Supporting nonplaster ceiling Not supporting ceiling	 $l/360$ $l/240$ $l/180$	 $l/360$ $l/240$ $l/180$	 $l/240$ $l/180$ $l/120$
Floor members	$l/360$	–	$l/240$
Exterior walls and interior partitions: With brittle finishes With flexible finishes	 – –	 $l/240$ $l/120$	 – –
Farm buildings	–	–	$l/180$
Greenhouses	–	–	$l/120$

a. For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed $l/60$. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed $l/150$. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed $l/90$. For roofs, this exception only applies when the metal sheets have no roof covering.
b. Interior partitions not exceeding 1.8 m in height and flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 4.11.
c. See SBC 201 for glass supports.
d. The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate drainage shall be investigated for ponding. See Chapter 8 for rain and ponding requirements and SBC 201 for roof drainage requirements.
e. The wind load is permitted to be taken as 0.7 times the “component and cladding” loads for the purpose of determining deflection limits herein.
f. For steel structural members, the dead load shall be taken as zero.
g. For cantilever members, l shall be taken as twice the length of the cantilever.

1.4.3.2 **Reinforced Concrete.** The deflection of reinforced concrete structural members shall not exceed that permitted by SBC 304.

1.4.3.3 **Steel.** The deflection of steel structural members shall not exceed that permitted by SBC 306.

1.4.3.4 **Masonry.** The deflection of masonry structural members shall not exceed that permitted by SBC 305.

1.4.4 **Analysis.** Load effects on individual structural members shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility, and both short- and long-term material properties. Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the added eccentricities expected to occur during their service life.

The total lateral force shall be distributed to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements that are assumed not to be a part of the lateral-force-resisting system shall be permitted to be incorporated into buildings provided that their effect on the action of the system is considered and provided for in design. Provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral-force-resisting system.

1.4.5 **Counteracting Structural Actions.** All structural members and systems, and all components and cladding in a building or other structure, shall be designed to resist forces due to earthquake, wind, soil and hydrostatic pressure and flood loads, with consideration of overturning, sliding, and uplift, and continuous load paths shall be provided for transmitting these forces to the foundation. Where sliding is used to isolate the elements, the effects of friction between sliding elements shall be included as a force. Where all or a portion of the resistance to these forces is provided by dead load, the dead load shall be taken as the minimum dead load likely to be in place during the event causing the considered forces. Consideration shall be given to the effects of vertical and horizontal deflections resulting from such forces.

1.4.6 **Self-straining Forces.** Provision shall be made for anticipated self-straining forces arising from differential settlements of foundations and from restrained dimensional changes due to temperature, moisture, shrinkage, creep, and similar effects.

SECTION 1.5
GENERAL STRUCTURAL INTEGRITY

1.5.0 Buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage. This shall be achieved through an arrangement of the structural elements that provides stability to the entire structural system by transferring loads from any locally damaged region to adjacent regions capable of resisting those loads without collapse. This shall be accomplished by providing sufficient continuity, redundancy, or energy-dissipating capacity (ductility), or a combination thereof, in the members of the structure.

SECTION 1.6
CLASSIFICATION OF BUILDINGS AND OTHER STRUCTURES

1.6.1 **Nature of Occupancy.** Buildings and other structures shall be classified, based on the nature of occupancy, according to Table 1.6-1 for the purposes of applying flood, wind, and earthquake provisions. The categories range from I to IV, where Category I represents buildings and other structures with a low hazard to human life in the event of failure and Category IV represents essential facilities. Each building or other structure shall be assigned to the highest applicable category or categories. Assignment of the same structure to multiple categories based on use and the type of load condition being evaluated (e.g., wind, seismic, etc.) shall be permissible.

When buildings or other structures have multiple uses (occupancies), the relationship between the uses of various parts of the building or other structure and the independence of the structural systems for those various parts shall be examined. The classification for each independent structural system of a multiple use building or other structure shall be that of the highest usage group in any part of the building or other structure that is dependent on that basic structural system.

1.6.2 **Hazardous Materials and Extremely Hazardous Materials.** Buildings and other structures containing hazardous materials or extremely hazardous materials are permitted to be classified as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as part of an overall risk management plan (RMP) that a release of the hazardous material or extremely hazardous material does not pose a threat to the public.

In order to qualify for this reduced classification, the owner or operator of the buildings or other structures containing the hazardous materials or extremely hazardous materials shall have a risk management plan that incorporates three elements as a minimum: a hazard assessment, a prevention program, and an emergency response plan.

As a minimum, the hazard assessment shall include the preparation and reporting of worst-case release scenarios for each structure under consideration, showing the potential effect on the public for each. As a minimum, the worst-case event shall include the complete failure (instantaneous release of entire contents) of a vessel, piping system, or other storage structure. A worst-case event includes (but is not limited to) a release during the design wind or design seismic event. In this assessment, the evaluation of the effectiveness of subsequent measures for accident mitigation shall be based on the assumption that the complete failure of the primary storage structure has occurred. The off-site impact must be defined in terms of population within the potentially affected area. In order to qualify for the reduced classification, the hazard assessment shall demonstrate that a release of the hazardous material from a worst-case event does not pose a threat to the public outside the property boundary of the facility.

As a minimum, the prevention program shall consist of the comprehensive elements of process safety management, which is based on accident prevention through the application of management controls in the key areas of design, construction, operation, and maintenance. Secondary containment of the hazardous materials or extremely hazardous materials (including, but not limited to, double wall tank, dike of sufficient size to contain a spill, or other means to contain a release of the hazardous materials or extremely hazardous material within the property boundary of the facility and prevent release of harmful quantities of contaminants to the air, soil, ground water, or surface water) are permitted to be used to mitigate the risk of release. When secondary containment is provided, it shall be designed for all environmental loads and is not eligible for this reduced classification.

As a minimum, the emergency response plan shall address public notification, emergency medical treatment for accidental exposure to humans, and procedures for emergency response to releases that have consequences beyond the property boundary of the facility. The emergency response plan shall address the potential that resources for response could be compromised by the event that has caused the emergency.

SECTION 1.7
ADDITIONS AND ALTERATIONS TO EXISTING STRUCTURES

1.7.0 When an existing building or other structure is enlarged or otherwise altered, structural members affected shall be strengthened if necessary so that the factored loads defined in this document will be supported without exceeding the specified design strength for the materials of construction. When using allowable stress design, strengthening is required when the stresses due to nominal loads exceed the specified allowable stresses for the materials of construction.

SECTION 1.8
LOAD TESTS

1.8.1 **In-situ Load Tests.** The building official is authorized to require an engineering analysis or a load test, or both, of any construction whenever there is reason to question the safety of the construction for the intended occupancy or use. Engineering analysis and load tests shall be conducted in accordance with SBC 302.

1.8.2 **Preconstruction Load Tests.** Materials and methods of construction that are not capable of being designed by approved engineering analysis or that do not comply with the specified material design standards or alternative test procedures in accordance with SBC 302, shall be load tested in accordance with SBC 302.

SECTION 1.9
ANCHORAGE

1.9.1 **General.** Anchorage of the roof to walls and columns, and of walls and columns to foundations, shall be provided to resist the uplift and sliding forces that result from the application of the prescribed loads.

1.9.2 **Concrete and Masonry Walls.** Concrete and masonry walls shall be anchored to floors, roofs and other structural elements that provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this chapter but not less than a minimum strength design horizontal force of 4.0 kN/m of wall, substituted for "*E*" in the load combinations of Section 2.3 or 2.4. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1.2 m. Required anchors in masonry-walls of hollow-units or cavity-walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 7.1 and 10.11 for wind and earthquake design requirements.

TABLE 1.6-1:
CLASSIFICATION OF BUILDINGS AND OTHER STRUCTURES FOR FLOOD, WIND AND EARTHQUAKE LOADS

Nature of Occupancy	Category
1) Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: a) Agricultural facilities b) Certain temporary facilities c) Minor storage facilities	I
All buildings and other structures except those listed in Categories I, III, and IV	II
1) Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: a) Buildings and other structures where more than 300 people congregate in one area b) Buildings and other structures with day care facilities with capacity greater than 150 c) Buildings and other structures with elementary school or secondary school facilities with capacity greater than 250 d) Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities e) Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities f) Jails and detention facilities g) Power generating stations and other public utility facilities not included in Category IV 2) Buildings and other structures not included in Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of hazardous materials to be dangerous to the public if released. 3) Buildings and other structures containing hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.6.2 that a release of the hazardous material does not pose a threat to the public.	III
1) Buildings and other structures designated as essential facilities including, but not limited to: a) Hospitals and other health care facilities having surgery or emergency treatment facilities b) Fire, rescue, ambulance, and police stations and emergency vehicle garages c) Designated earthquake, hurricane, or other emergency shelters d) Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response e) Power generating stations and other public utility facilities required in an emergency f) Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Category IV structures during an emergency g) Aviation control towers, air traffic control centers, and emergency aircraft hangars h) Water storage facilities and pump structures required to maintain water pressure for fire suppression i) Buildings and other structures having critical national defense functions 2) Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing extremely hazardous materials where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. 3) Buildings and other structures containing extremely hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.6.2 that a release of the extremely hazardous material does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	IV

CHAPTER 2
COMBINATION OF LOADS

SECTION 2.1
GENERAL

Buildings and other structures shall be designed using the provisions of either Section 2.3 or 2.4. Either Section 2.3 or 2.4 shall be used exclusively for proportioning elements of a particular construction material throughout the structure.

SECTION 2.2
SYMBOLS AND NOTATIONS

D = dead load;
E = earthquake load;
F = load due to fluids with well-defined pressures and maximum heights;
F_a = flood load;
H = load due to lateral earth pressure, ground water pressure, or pressure of bulk materials;
L = live load;
L_r = roof live load;
P = ponding load;
R = rain load;
T = self-straining force;
W = wind load;

SECTION 2.3
COMBINING FACTORED LOADS USING STRENGTH DESIGN

2.3.1 **Applicability.** The load combinations and load factors given in Section 2.3.2 shall be used only in those cases in which they are specifically authorized by the applicable material design standard.

2.3.2 **Basic Combinations.** Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

$1.4\ (D + F)$ **(Eq. 2.3.2-1)**

$1.2\ (D + F + T) + 1.6\ (L + H) + 0.5\ (L_r \text{ or } R)$ **(Eq. 2.3.2-2)**

$1.2\ D + 1.6\ (L_r \text{ or } R) + (f_1 L \text{ or } 0.8\ W)$ **(Eq. 2.3.2-3)**

$1.2D + 1.6W + f_1 L + 0.5\ (L_r \text{ or } R)$ **(Eq. 2.3.2-4)**

$1.2D + 1.0\ E + f_1 L$ **(Eq. 2.3.2-5)**

$0.9D + 1.6W + 1.6H$ **(Eq. 2.3.2-6)**

$0.9D + 1.0E + 1.6H$ **(Eq. 2.3.2-7)**

where

f_1 = 1.0 for areas occupied as places of public assembly, for live loads in excess of 5.0 kN/m^2, and for parking garage live load.

f_1 = 0.5 for other live loads.

Exceptions:

1. The load factor on H shall be set equal to zero in (Eq. 2.3.2-6) and (Eq. 2.3.2-7) if the structural action due to H counteracts that due to W or E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.
2. For Concrete structures and Masonry construction (Eq. 2.3.2-2) shall be

$$1.4\ (D + F + T) + 1.7\ (L + H) + 0.5\ (L_r \text{ or } R)$$

Each relevant strength limit state shall be investigated. Effects of one or more loads not acting shall be investigated. The most unfavorable effects from both wind and earthquake loads shall be investigated, where appropriate, but they need not be considered to act simultaneously. Refer to Section 9.3 for specific definition of the earthquake load effect E.

2.3.3 **Load Combinations Including Flood Load.** When a structure is located in a flood zone (Section 5.3.1), the following load combinations shall be considered:

1. In V-Zones or Coastal A-Zones, 1.6W in (Eq. 2.3.2-4) and (Eq. 2.3.2-6) shall be replaced by 1.6W + 2.0 F_a.
2. In noncoastal A-Zones, 1.6W in (Eq. 2.3.2-4) and (Eq. 2.3.2-6) shall be replaced by 0.8W + 1.0 F_a.

SECTION 2.4
COMBINING NOMINAL LOADS USING ALLOWABLE STRESS DESIGN

2.4.1 **Basic Combinations.** Loads listed herein shall be considered to act in the following combinations; whichever produces the most unfavorable effect in the building, foundation, or structural member being considered. Effects of one or more loads not acting shall be considered.

$D + F$ **(Eq. 2.4.1-1)**

$D + H + F + L + T$ **(Eq. 2.4.1-2)**

$D + H + F + (L_r \text{ or } R)$ **(Eq. 2.4.1-3)**

$D + H + F + 0.75\ (L + T) + 0.75\ (L_r \text{ or } R)$ **(Eq. 2.4.1-4)**

$D + H + F + (W \text{ or } 0.7E)$ **(Eq. 2.4.1-5)**

$D + H + F + 0.75\ (W \text{ or } 0.7\ E) + 0.75\ L + 0.75\ (L_r \text{ or } R)$ **(Eq. 2.4.1-6)**

$0.6D + W + H$ **(Eq. 2.4.1-7)**

$0.6D + 0.7E + H$ **(Eq. 2.4.1-8)**

The most unfavorable effects from both wind and earthquake loads shall be considered, where appropriate, but they need not be assumed to act simultaneously. Refer to Section 9.3 for the specific definition of the earthquake load effect E.

Increases in allowable stress shall not be used with the loads or load combinations given in this standard unless it can be demonstrated that such an increase is justified by structural behavior caused by rate or duration of load.

2.4.1.1 **Load Combinations Including Flood Load.** When a structure is located in a flood zone, the following load combinations shall be considered:

1. In V-Zones or Coastal A-Zones (Section 5.3.1), 1.5 F_a shall be added to other loads in (Eq. 2.4.1-5), (Eq. 2.4.1-6) and (Eq. 2.4.1-7) and E shall be set equal to zero in (Eq. 2.4.1-5) and (Eq. 2.4.1-6).
2. In noncoastal A-Zones, $0.75F_a$ shall be added to combinations (Eq. 2.4.1-5), (Eq. 2.4.1-6) and (Eq. 2.4.1-7) and E shall be set equal to zero in (Eq. 2.4.1-5) and (Eq. 2.4.1-6).

2.4.2 **Alternative basic load combinations.** In lieu of the basic load combinations specified in Section 2.4.1, and as required by SBC 303, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternate basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced, where permitted by the material section of this code or referenced standard.

$D + L + (L_r \text{ or } R)$ **(Eq. 2.4.2-1)**

$D + L + 1.3\ W$ **(Eq. 2.4.2-2)**

$D + L + 1.3\ W$ **(Eq. 2.4.2-3)**

$D + L + 1.3\ W/2$ **(Eq. 2.4.2-4)**

$D + L + E/1.4$ **(Eq. 2.4.2-5)**

$0.9D + E/1.4$ **(Eq. 2.4.2-6)**

Exceptions:
Crane hook loads need not be combined with roof live load or one-half of the wind load.

2.4.2.1 **Other loads.** Where *F*, *H*, *P* or *T* are to be considered in design, 1.0 times each applicable load shall be added to the combinations specified in Section 2.4.2.

SECTION 2.5
SPECIAL SEISMIC LOAD COMBINATIONS

For both allowable stress design and strength design methods, where specifically required by Chapters 9 through 16 or by SBC 303, SBC 304, SBC 305, and SBC 306 or Ref. 11.1-1, elements and components shall be designed to resist the forces calculated using Equation 2.5-1 when the effects of the seismic ground motion are additive to gravity forces and those calculated using Equation 2.5-2 when the effects of the seismic ground motion counteract gravity forces.

$1.2D + f_1L + E$ **(Eq. 2.5-1)**

$0.9D + E$ **(Eq. 2.5-2)**

where:

E = The maximum effect of horizontal and vertical forces as per section 10.4.1.

f_1 = 1.0 for areas occupied as places of public assembly, for live loads in excess of 5.0 kN/m^2 and for parking garage live load.

f_1 = 0.5 for other live loads.

SECTION 2.6
LOAD COMBINATIONS FOR EXTRAORDINARY EVENTS

Where required by the applicable code, standard, or the authority having jurisdiction, strength and stability shall be checked to ensure that structures are capable of withstanding the effects of extraordinary (i.e., low-probability) events such as fires, explosions, and vehicular impact.

CHAPTER 3
DEAD LOADS

SECTION 3.1
DEFINITION

Dead loads consist of the weight of all materials of construction incorporated into the building including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items, and fixed service equipment including the weight of cranes.

SECTION 3.2
WEIGHTS OF MATERIALS AND CONSTRUCTIONS

In determining dead loads for purposes of design, the actual weights of materials and constructions shall be used provided that in the absence of definite information, values approved by the authority having jurisdiction shall be used. The minimum design dead loads are shown in Table 3-1 and Table 3-1(a) and the minimum densities for design loads from materials are shown in Table 3-2.

SECTION 3.3
WEIGHT OF FIXED SERVICE EQUIPMENT

In determining dead loads for purposes of design, the weight of fixed service equipment such as plumbing stacks and risers, electrical feeders, heating, ventilating and air conditioning systems (HVAC) and fire sprinkler systems shall be included.

TABLE 3-1: MINIMUM DESIGN DEAD LOADS

Component	Load (kN/m^2)	Component	Load (kN/m^2)
COVERINGS, ROOF, AND WALL		**FLOORS AND FLOOR FINISHES**	
Asbestos-cement shingles	0.20	Asphalt block (50 mm), 13 mm mortar	1.45
Asphalt shingles	0.10	Cement finish (25 mm) on stone-concrete fill	1.55
Cement tile	0.80	Ceramic or quarry tile (20 mm) on 13 mm mortar bed	0.80
Clay tile (for mortar add 0.50 kN/m^2)			
Book tile, 50 mm	0.60	Ceramic or quarry tile (20 mm) on 25 mm mortar bed	1.10
Book tile, 75 mm	1.00		
Ludowici	0.50	Concrete fill finish (per mm thickness)	0.023
Roman	0.60	Hardwood flooring, 20 mm	0.20
Spanish	0.90	Linoleum or asphalt tile, 6 mm	0.05
Composition:		Marble and mortar on stone-concrete fill	1.60
Three-ply ready roofing	0.05	Slate (per mm thickness)	0.03
Four-ply felt and gravel	0.25	Solid flat tile on 25 mm mortar base	1.10
Five-ply felt and gravel	0.30	Subflooring, 20 mm	0.15
Copper or tin	0.05	Terrazzo (38 mm) directly on slab	0.90
Corrugated asbestos-cement roofing	0.20	Terrazzo (25 mm) on stone-concrete fill	1.55
Deck, metal, 20 gage	0.10	Terrazzo (25 mm), on 50 mm stone concrete	1.55
Deck, metal, 18 gage	0.15	Wood block (75 mm) on mastic, no fill	0.50
Decking, 50 mm wood (Douglas fir)	0.25	Wood block (75 mm) on 13 mm mortar base	0.80
Decking, 75 mm wood (Douglas fir)	0.40	**FRAME PARTITIONS**	
Fiberboard, 13 mm	0.05	Movable steel partitions	0.20
Gypsum sheathing, 13 mm	0.10	Wood or steel studs, 13 mm gypsum board each side	0.40
Insulation, roof boards (per mm thickness)		Wood studs, 50 x 100, unplastered	0.20
Cellular glass	0.0015	Wood studs, 50 x 100, plastered one side	0.60
Fibrous glass	0.002	Wood studs, 50 x 100, plastered two sides	1.00
Fiberboard	0.003	**FRAME WALLS**	
Perlite	0.0015	Exterior stud walls with brick veneer	2.30
Polystyrene foam	0.0005	Windows, glass, frame and sash	0.40
Urethane foam with skin	0.001	Clay brick wythes:	
Plywood (per mm thickness)	0.006	100 mm	1.90
Rigid insulation, 13 mm	0.05	200 mm	3.80
Skylight, metal frame, 10 mm wire glass	0.40	300 mm	5.50
		400 mm	7.50
Slate, 5 mm	0.35	**CEILINGS**	
Slate, 6 mm	0.50	Acoustical fiberboard	0.05
Waterproofing membranes:		Gypsum board (per mm thickness)	0.01
Bituminous, gravel-covered	0.30	Mechanical duct allowance	0.20
Bituminous, smooth surface	0.10	Plaster on tile or concrete	0.25
Liquid applied	0.05	Plaster on wood lath	0.40
Single-ply, sheet	0.03	Suspended steel channel system	0.10
Wood sheathing (per mm thickness)	0.006	Suspended metal lath and cement plaster	0.75
Wood shingles	0.15	Suspended metal lath and gypsum plaster	0.50
FLOOR FILL		Wood furring suspension system	0.15
Cinder concrete, per mm	0.017		
Lightweight concrete, per mm	0.015		
Sand, per mm	0.015		
Stone concrete, per mm	0.023		

TABLE 3-1(a):
MINIMUM DESIGN DEAD LOADS*FOR
DIFFERENT THICKNESS OF MASONRY WALLS, (kN/m^2)

Component thickness		100 mm	150 mm	200 mm	250 mm	300 mm
Hollow concrete masonry unit wythes:						
Density of unit (16.5 kN/m^3)						
No grout		1.05	1.30	1.70	2.00	2.35
1200 mm	grout		1.50	1.95	2.35	2.80
1000 mm			1.60	2.05	2.55	3.00
800 mm	spacing		1.65	2.15	2.70	3.15
600 mm			1.80	2.35	2.95	3.45
400 mm			2.00	2.70	3.35	4.00
Full grout			2.75	3.70	4.70	5.70
Density of unit (19.5 kN/m^3):						
No grout		1.25	1.35	1.70	2.10	2.40
1200 mm	grout		1.60	2.10	2.60	3.00
1000 mm			1.65	2.15	2.70	3.10
800 mm	spacing		1.70	2.25	2.80	3.25
600 mm			1.90	2.45	3.00	3.60
400 mm			2.10	2.80	3.50	4.20
Full grout			2.80	3.90	4.90	5.90
Density of unit (21.0 kN/m^3)						
No grout		1.40	1.70	2.15	2.60	3.00
1200 mm	grout		1.60	2.40	2.90	3.45
1000 mm			1.70	2.55	3.10	3.70
800 mm	spacing		1.80	2.65	3.25	3.85
600 mm			2.00	2.80	3.50	4.10
400 mm			2.25	3.15	3.95	4.70
Full grout			3.05	4.15	5.25	6.40
Solid concrete masonry unit wythes (incl. concrete brick):						
Density of unit (16.5 kN/m^3):		1.55	2.35	3.20	4.00	4.90
Density of unit (19.5 kN/m^3):		1.85	2.85	3.80	4.80	5.80
Density of unit (21.0 kN/m^3):		2.00	3.00	4.15	5.15	6.25

* Weights of masonry include mortar but not plaster. For plaster, add 0.25 kN/m^2 for each face plastered. Values given represent averages. In some cases, there is a considerable range of weight for the same construction.

TABLE 3-2:
MINIMUM DENSITIES FOR DESIGN LOADS FROM MATERIALS

Material	Density (kN/m^3)	Material	Density (kN/m^3)
Aluminum	26.5	Earth (submerged)	
Bituminous products		Clay	12.5
Asphaltum	13.0	Soil	11.0
Graphite	21.0	River mud	14.0
Paraffin	9.0	Sand or gravel	9.5
Petroleum, crude	8.5	Sand or gravel and clay	10.0
Petroleum, refined	8.0	Glass	25.0
Petroleum, benzine	7.0	Gravel, dry	16.5
Petroleum, gasoline	6.5	Gypsum, loose	11.0
Pitch	11.0	Gypsum, wallboard	8.0
Tar	12.0	Ice	9.0
Brass	82.5	Iron	
Bronze	87.0	Cast	71.0
Cast-stone masonry (cement, stone, sand)	23.0	Wrought	75.5
Cement, portland, loose	14.0	Lead	111.5
Ceramic tile	23.5	Lime	
Charcoal	2.0	Hydrated, loose	5.0
Cinder fill	9.0	Hydrated, compacted	7.0
Cinders, dry, in bulk	7.0	Masonry, Ashlar stone	
Coal		Granite	26.0
Anthracite, piled	8.0	Limestone, crystalline	26.0
Bituminous, piled	7.5	Limestone, oolitic	21.0
Lignite, piled	7.5	Marble	27.0
Peat, dry, piled	3.5	Sandstone	23.0
Concrete, plain		Masonry, brick	
Cinder	17.0	Hard (low absorption)	20.5
Expanded-slag aggregate	16.0	Medium (medium absorption)	18.0
Haydite (burned-clay aggregate)	14.0	Soft (high absorption)	16.0
Slag	21.0	Masonry, concrete*	
Stone (including gravel)	23.0	Lightweight units	16.5
Vermiculite and perlite aggregate, non-load-bearing	4.0-8.0	Medium weight units	19.5
Other light aggregate, load-bearing	11.0-16.5	Normal weight units	21.0
Concrete, reinforced		Masonry grout	22.0
Cinder	17.5	Masonry, rubble stone	
Slag	22.0	Granite	24.0
Stone (including gravel)	24.0	Limestone, crystalline	23.0
Copper	87.5	Limestone, oolitic	22.0
Cork, compressed	2.0	Marble	24.5
Earth (not submerged)		Sandstone	21.5
Clay, dry	10.0	Mortar, cement or lime	20.5
Clay, damp	17.5	Particleboard	7.0
Clay and gravel, dry	16.0	Plywood	6.0
Silt, moist, loose	12.5	Riprap (Not submerged)	
Silt, moist, packed	15.0	Limestone	13.0
Silt, flowing	17.0	Sandstone	14.0
Sand and gravel, dry, loose	16.0	Sand	
Sand and gravel, dry, packed	17.5	Clean and dry	14.0
Sand and gravel, wet	19.0	River, dry	17.0

(continued)

TABLE 3-2:
MINIMUM DENSITIES FOR DESIGN LOADS FROM MATERIALS - continued

Material	Density (kN/m^3)	Material	Density (kN/m^3)
Slag		Tin	72.0
Bank	11.0	Water	
Bank screenings	17.0	Fresh	10.0
Machine	15.0	Sea	10.0
Sand	8.0	Wood, seasoned	
Slate	27.0	Ash, commercial white	6.5
Steel, cold-drawn	77.5	Cypress, southern	5.5
Stone, quarried, piled		Fir, Douglas, coast region	5.5
Basalt, granite, gneiss	15.0	Hem fir	4.5
Limestone, marble, quartz	15.0	Oak, commercial reds and whites	7.5
Sandstone	13.0	Pine, southern yellow	6.0
Shale	14.5	Redwood	4.5
Greenstone, hornblende	17.0	Spruce, red, white, and Stika	4.5
Terra cotta, architectural		Western hemlock	5.0
Voids filled	19.0	Zinc, rolled sheet	70.5
Voids unfilled	11.5		

*Tabulated values apply to solid masonry and to the solid portion of hollow masonry.

CHAPTER 4
LIVE LOADS

SECTION 4.1
DEFINITION

Live loads are those loads produced by the use and occupancy of the building or other structure and do not include construction or environmental loads such as wind load, rain load, earthquake load, flood load, or dead load. Live loads on a roof are those produced (1) during maintenance by workers, equipment, and materials, and (2) during the life of the structure by movable objects such as planters and by people.

SECTION 4.2
UNIFORMLY DISTRIBUTED LOADS

4.2.1 **Required Live Loads.** The live loads used in the design of buildings and other structures shall be the maximum loads expected by the intended use or occupancy but shall in no case be less than the minimum uniformly distributed unit loads required by Table 4-1 and Table 4-2.

4.2.2 **Provision for Partitions.** In office buildings or other buildings where partitions will be erected or rearranged partition weight shall be considered, whether or not partitions are shown on the plans, unless the specified live load exceeds (4 kN/m^2).

SECTION 4.3
CONCENTRATED LOADS

Floors and other similar surfaces shall be designed to support safely the uniformly distributed live loads prescribed in Section 4.2 or the concentrated load, in kilonewtons (kN), given in Table 4-1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area (750 mm x 750 mm) and shall be located so as to produce the maximum load effects in the structural members.

Any single panel point of the lower chord of exposed roof trusses or any point along the primary structural members supporting roofs over manufacturing, commercial storage and warehousing, and commercial garage floors shall be capable of carrying safely a suspended concentrated load of not less than 9.0 kN in addition to dead load. For all other occupancies, a load of 0.9 kN shall be used instead of 9.0 kN.

SECTION 4.4
LOADS ON HANDRAILS, GUARDRAIL SYSTEMS, GRAB BAR SYSTEMS, VEHICLE BARRIER SYSTEMS, AND FIXED LADDERS

4.4.1 **Definitions.**

Handrail. A rail grasped by hand for guidance and support. A handrail assembly includes the handrail, supporting attachments, and structures.

Fixed Ladder. A ladder that is permanently attached to a structure, building, or equipment.

Guardrail System. A system of building components near open sides of an elevated surface for the purpose of minimizing the possibility of a fall from the elevated surface by people, equipment, or material.

Grab Bar System. A bar provided to support body weight in locations such as toilets, showers, and tub enclosures.

Vehicle Barrier System. A system of building components near open sides of a garage floor or ramp, or building walls that act as restraints for vehicles.

4.4.2 Loads.

(a) Handrail assemblies and guardrail systems shall be designed to resist a load of 0.75 kN/m applied in any direction at the top and to transfer this load through the supports to the structure. For one- and two-family dwellings, the minimum load shall be 0.3 kN/m.

Further, all handrail assemblies and guardrail systems shall be able to resist a single concentrated load of 0.9 kN applied in any direction at any point along the top, and have attachment devices and supporting structure to transfer this loading to appropriate structural elements of the building. This load need not be assumed to act concurrently with the loads specified in the preceding paragraph.

Intermediate rails (all those except the handrail), balusters, and panel fillers shall be designed to withstand a horizontally applied normal load of (0.2 kN) on an area not to exceed (300 mm x 300 mm) including openings and space between rails. Reactions due to this loading are not required to be superimposed with those of either preceding paragraph.

(b) Grab bar systems shall be designed to resist a single concentrated load of (1.1 kN) applied in any direction at any point.

(c) Vehicle barrier systems for passenger cars shall be designed to resist a single load of (27.0 kN) applied horizontally in any direction to the barrier system, and shall have anchorages or attachments capable of transferring this load to the structure. For design of the system, the load shall be assumed to act at a minimum height of (450 mm) above the floor or ramp surface on an area not to exceed (300 mm x 300 mm), and is not required to be assumed to act concurrently with any handrail or guardrail loadings specified in the preceding paragraphs of Section 4.4.2. Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provision for traffic railings.

(d) The minimum design live load on fixed ladders with rungs shall be a single concentrated load of 1.5 kN, and shall be applied at any point to produce the maximum load effect on the element being considered. The number and position of additional concentrated live load units shall be a minimum of 1 unit of 1.5 kN for every 3 m of ladder height.

(e) Where rails of fixed ladders extend above a floor or platform at the top of the ladder, each side rail extension shall be designed to resist a concentrated live load of 0.5 kN in any direction at any height up to the top of the side rail extension. Ship ladders with treads instead of rungs shall have minimum design loads as stairs, defined in Table 4-1.

SECTION 4.5
LOADS NOT SPECIFIED

For occupancies or uses not designated in Section 4.2 or 4.3, the live load shall be determined in accordance with a method approved by the building official.

SECTION 4.6
PARTIAL LOADING

The full intensity of the appropriately reduced live load applied only to a portion of a structure or member shall be accounted for if it produces a more unfavorable effect than the same intensity applied over the full structure or member.

SECTION 4.7
IMPACT LOADS

The live loads specified in Sections 4.2.1 and 4.4.2 shall be assumed to include adequate allowance for ordinary impact conditions. Provision shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

4.7.1 **Elevators.** All elevator loads shall be increased by 100% for impact and the structural supports shall be designed within the limits of deflection prescribed by Refs. 4-1 and 4-2.

4.7.2 **Machinery.** For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact:

(1) elevator machinery, 100%;

(2) light machinery, shaft- or motor-driven, 20%;

(3) reciprocating machinery or power-driven units, 50%;

(4) hangers for floors or balconies, 33%. All percentages shall be increased where specified by the manufacturer.

SECTION 4.8
REDUCTION IN LIVE LOADS

The minimum uniformly distributed live loads, L_o in Table 4-1, may be reduced according to the following provisions.

4.8.1 **General.** Subject to the limitations of Sections 4.8.2 through 4.8.5, members for which a value of $K_{LL}A_T$ is 37.0 m^2 or more are permitted to be designed for a reduced live load in accordance with the following formula:

$$L = L_o \left(0.25 + \frac{4.57}{\sqrt{K_{LL} A_T}} \right) \qquad \textbf{(Eq. 4-1)}$$

where

L = reduced design live load per square m of area supported by the member.

L_o = unreduced design live load per square m of area supported by the member (see Table 4-1)

K_{LL} = live load element factor (see Table 4-3).

A_T = tributary area m^2

L = shall not be less than $0.50L_0$ for members supporting one floor and L shall not be less than $0.40L_0$ for members supporting two or more floors.

4.8.2 **Heavy Live Loads.** Live loads that exceed 5 kN/m^2 shall not be reduced except the live loads for members supporting two or more floors may be reduced by 20%.

4.8.3 **Passenger Car Garages.** The live loads shall not be reduced in passenger car garages except the live loads for members supporting two or more floors may be reduced by 20%.

4.8.4 **Special Occupancies.** Live loads of 5 kN/m^2 or less shall not be reduced in public assembly occupancies.

4.8.5 **Limitations on One-Way Slabs.** The tributary area, A_T, for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

SECTION 4.9
MINIMUM ROOF LIVE LOADS

4.9.1 **Flat, Pitched, and Curved Roofs.** Ordinary flat, pitched, and curved roofs shall be designed for the live loads specified in Eq. 4-2 or other controlling combinations of loads as discussed in Chapter 2, whichever produces the greater load. In structures such as greenhouses, where special scaffolding is used as a work surface for workmen and materials during maintenance and repair operations, a lower roof load than specified in Eq. 4-2 shall not be used unless approved by the authority having jurisdiction.

$$L_r = 1.0\, R_1 R_2 \text{ where } 0.6 \le L_r \le 1.0 \qquad \textbf{(Eq. 4-2)}$$

Where:

L_r = roof live load/m^2 of horizontal projection for a non-accessible roof, kN/m^2.

The reduction factors R_1 and R_2 shall be determined as

$$R_1 = \begin{cases} 1 & \text{for } A_t \le 18.0\ m^2 \\ 1.2 - 0.0111\, A_t & \text{for } 18.0\ m^2 < A_t < 54\ m^2 \\ 0.6 & \text{for } A_t \ge 54\ m^2 \end{cases}$$

where A_t = tributary area m^2 supported by any structural member and

$$R_2 = \begin{cases} 1 & \text{for } F \le 4 \\ 1.2 - 0.05\, F & \text{for } 4 < F < 12 \\ 0.6 & \text{for } F \ge 12 \end{cases}$$

where, for a pitched roof, F = 0.12 x slope, with slope expressed in percentage points and, for an arch or dome, *F* = rise-to-span ratio multiplied by 32.

4.9.2 **Special-Purpose Roofs.** Roofs used for promenade purposes shall be designed for a minimum live load of 3.0 kN/m^2. Roofs used for roof gardens or assembly purposes shall be designed for a minimum live load of 5 kN/m^2. Roofs used for other special purposes shall be designed for appropriate loads as approved by the authority having jurisdiction.

4.9.3 **Special Structural Elements.** Live loads of 5 kN/m^2 or less shall not be reduced for roof members except as specified in Section 4.9.

SECTION 4.10
CRANE LOADS

The crane live load shall be the rated capacity of the crane. Design loads for the runway beams, including connections and support brackets, of moving bridge cranes and monorail cranes shall include the maximum wheel loads of the crane and the vertical impact, lateral, and longitudinal forces induced by the moving crane.

4.10.1 **Maximum Wheel Load.** The maximum wheel loads shall be the wheel loads produced by the weight of the bridge, as applicable, plus the sum of the rated capacity and the weight of the trolley with the trolley positioned on its runway at the location where the resulting load effect is maximum.

4.10.2 **Vertical Impact Force.** The maximum wheel loads of the crane shall be increased by the percentages shown below to determine the induced vertical impact or vibration force:

Monorail cranes (powered)	25
Cab-operated or remotely operated bridge cranes (powered)	25
Pendant-operated bridge cranes (powered)	10
Bridge cranes or monorail cranes with hand-geared bridge, trolley, and hoist	0

4.10.3 **Lateral Force.** The lateral force on crane runway beams with electrically powered trolleys shall be calculated as 20% of the sum of the rated capacity of the crane and the weight of the hoist and trolley. The lateral force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction perpendicular to the beam, and shall be distributed with due regard to the lateral stiffness of the runway beam and supporting structure.

4.10.4 **Longitudinal Force.** The longitudinal force on crane runway beams, except for bridge cranes with hand-geared bridges, shall be calculated as 10% of the maximum wheel loads of the crane. The longitudinal force shall be assumed to act horizontally at the traction surface of a runway beam in either direction parallel to the beam.

TABLE 4-1:
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS

Occupancy or Use	Uniform kN/m^2	Conc. kN
Apartments (see residential)		
Access floor systems		
Office use	2.5	9
Computer use	5	9
Armories and drill rooms	7.5	
Assembly areas and theaters		
• Fixed seats (fastened to floor)	3	
• Lobbies	5	
• Movable seats	5	
• Platforms (assembly)	5	
• Stage floors	7.5	
Balconies (exterior)	5	
On one- and two-family residences only, and not exceeding 10 m^2	3	
Bowling alleys, poolrooms, and similar recreational areas	4	
Catwalks for maintenance access	2	1.5
Corridors		
First floor	5	
Other floors, same as occupancy served except as indicated		
Mosques	5	
Decks (patio and roof)		
Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	5	
Dwellings (see residential)		
Elevator machine room grating (on area of 2500 mm^2)		1.5
Finish light floor plate construction (on area of 650 mm^2)		1
Fire escapes	5	
Fixed ladders	See Section 4.4	
Garages (passenger vehicles only)	2	Note (1)
Trucks and buses	Note (2)	Note (2)
Grandstands (see stadium and arena bleachers)		
Gymnasiums, main floors, and balconies	5 Note (4)	
Handrails, guardrails, and grab bars	See Section 4.4	
Hospitals		
• Operating rooms, laboratories	3	4.5
• Private rooms	2	4.5
• Wards	2	4.5
• Corridors above first floor	4	4.5
Hotels (see residential)		
Libraries		
• Reading rooms	3	4.5
• Stack rooms	7.5 Note (3)	4.5
• Corridors above first floor	4	4.5
Manufacturing		
• Light	6	9
• Heavy	12	13.5
Marquees and canopies	4	
Office buildings		
• File and computer rooms shall be designed for heavier loads based on anticipated occupancy:		
• Lobbies and first floor corridors	5	9
• Offices	2.5	9
• Corridors above first floor	4.0	9
Penal institutions		
Cell blocks	2	
Corridors	5	

TABLE 4-1:
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o,
AND MINIMUM CONCENTRATED LIVE LOADS – continued

Occupancy or Use	Uniform kN/m²	Conc. kN
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	0.5	
Uninhabitable attics with storage	1.0	
Habitable attics and sleeping areas	1.5	
All other areas except stairs and balconies	2.0	
Hotels and multifamily houses		
Private rooms and corridors serving them	2.0	
Public rooms and corridors serving them	5.0	
Reviewing stands, grandstands, and bleachers	5.0 Note (4)	
Roofs	See Sections 4.3 and 4.9	
Schools		
Classrooms	3	4.5
Corridors above first floor	4	4.5
First floor corridors	5	4.5
Scuttles, skylight ribs, and accessible ceilings		10
Sidewalks, vehicular driveways, and yards subject to trucking	12 Note (5)	36 Note (6)
Stadiums and arenas		
Bleachers	5 Note (4)	
Fixed Seats (fastened to floor)	3 Note (4)	
Stairs and exit-ways	5	Note (7)
One- and two-family residences only	2	
Storage areas above ceilings	1	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	6	
Heavy	12	
Stores		
Retail		
First floor	5	4.5
Upper floors	4	4.5
Wholesale, all floors	6	4.5
Vehicle barriers	See Section 4.4	
Walkways and elevated platforms (other than exit-ways)	3	
Yards and terraces, pedestrians	5	

Notes

(1) Floors in garages or portions of building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 4-1 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 13.0 kN acting on an area of 100 mm by 100 mm, footprint of a jack; (2) for mechanical parking structures without slab or deck which are used for storing passenger car only, 10 kN per wheel.

(2) Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provisions for truck and bus loadings.

(3) The loading applies to stack room floors that support nonmobile, double-faced library bookstacks subject to the following limitations:
   a. The nominal bookstack unit height shall not exceed 2300 mm;
   b. The nominal shelf depth shall not exceed 300 mm for each face; and
   c. Parallel rows of double-faced bookstacks shall be separated by aisles not less than 900 mm wide.

(4) In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 0.4 kN/linear m of seat applied in a direction parallel to each row of seats and 0.15 kN/linear m of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

(5) Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

(6) The concentrated wheel load shall be applied on an area 100 mm by 100 mm, footprint of a jack.

(7) Minimum concentrated load on stair treads on area of 2500 mm^2 is 1.5 kN.

TABLE 4-2: MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS

Occupancy or use	Live Load kN/m^2	Occupancy or use	Live Load kN/m^2
Air-conditioning (machine space)	10*	Laboratories, scientific	5
Amusement park structure	5*	Laundries	7.2*
Attic, Nonresidential		Libraries, corridors	4*
Nonstorage	1.2	Manufacturing, ice	14.5
Storage	4*	Morgue	6.0
Bakery	7.2	Office Buildings	
Exterior	5	Business machine equipment	5*
Interior (fixed seats)	3	Files (see file room)	
Interior (movable seats)	5	Printing Plants	
Boathouse, floors	5*	Composing rooms	5
Boiler room, framed	14.5	Linotype rooms	5
Broadcasting studio	5	Paper storage	**
Catwalks	1.2	Press rooms	7.2*
Ceiling, accessible furred	0.5#	Public rooms	5
Cold Storage		Railroad tracks	++
No overhead system	12+	Ramps	
Overhead system		Driveway (see garages)	
Floor	7.2	Pedestrian (see sidewalks and corridors in Table 4-1)	
Roof	12	Seaplane (see hangars)	
Computer equipment	7.2*	Rest rooms	3
Courtrooms	2.5 – 5.0	Rinks	
Dormitories		Ice skating	12
Nonpartitioned	4	Roller skating	5
Partitioned	2	Storage, hay or grain	14.5*
Elevator machine room	7.2*	Telephone exchange	7.2*
Fan room	7.2*	Theaters	
File room		Dressing rooms	2
Duplicating equipment	7.2*	Grid-iron floor or fly gallery:	
Card	6*	Grating	3
Letter	4*	Well beams, 3.7 kN/m per pair	
Foundries	30*	Header beams, 15 kN/m	
Fuel rooms, framed	20	Pin rail, 3.7 kN/m	
Garages -trucks	℘	Projection room	5
Greenhouses	7.2	Toilet rooms	3
Hangars	7.2 ℘	Transformer rooms	10*
Incinerator charging floor	5	Vaults, in offices	12*
Kitchens, other than domestic	7.2*		

* Use weight of actual equipment or stored material when greater.

\+ Plus 7.2 kN/m^2 for trucks.

℘ Use Ministry of Transportation Lane Load for Highway Bridges. Also subject to not less than 100% maximum axle load.

** Paper storage 2.5 kN/m^2/m of clear story height.

++ As required by Saudi Railway Organization.

\# Accessible ceilings normally are not designed to support persons. The value in this table is intended to account for occasional light storage or suspension of items. If it may be necessary to support the weight of maintenance personnel, this shall be provided for.

SECTION 4.11
INTERIOR WALLS AND PARTITIONS

4.11.1 Interior walls and partitions that exceed 1.8 m in height, including their finish materials, shall have adequate strength to resist the loads to which they are subjected but not less than a horizontal load of $0.25 kN/m^2$.

TABLE 4-3:
LIVE LOAD ELEMENT FACTOR, K_{LL}

Element	K_{LL}
Interior Columns Exterior Columns without cantilever slabs	4 4
Edge Columns with cantilever slabs	3
Corner Columns with cantilever slabs Edge Beams without cantilever slabs Interior Beams	2 2 2
All Other Members Not Identified Above including: Edge Beams with cantilever slabs Cantilever Beams One-way Slabs Two-way Slabs Members without provisions for continuous shear transfer normal to their span	1

CHAPTER 5
SOIL AND HYDROSTATIC PRESSURE AND FLOOD LOADS

SECTION 5.1
PRESSURE ON BASEMENT WALLS

In the design of basement walls and similar approximately vertical structures below grade, provision shall be made for the lateral pressure of adjacent soil. Due allowance shall be made for possible surcharge from fixed or moving loads. When a portion or the whole of the adjacent soil is below a free-water surface, computations shall be based on the weight of the soil diminished by buoyancy, plus full hydrostatic pressure.

Basement walls shall be designed to resist lateral soil loads. Soil loads specified in Table 5-1 shall be used as the minimum design lateral soil loads unless specified otherwise in a soil investigation report approved by the authority having jurisdiction. The lateral pressure from surcharge loads shall be added to the lateral earth pressure load. The lateral pressure shall be increased if soils with expansion potential are present at the site as determined by a geotechnical investigation.

SECTION 5.2
UPLIFT ON FLOORS AND FOUNDATIONS

In the design of basement floors and similar approximately horizontal elements below grade, the upward pressure of water, where applicable, shall be taken as the full hydrostatic pressure applied over the entire area. The hydrostatic head shall be measured from the underside of the construction. Any other upward loads shall be included in the design.

Where expansive soils are present under foundations or slabs-on-ground, the foundations, slabs, and other components shall be designed to tolerate the movement or resist the upward pressures caused by the expansive soils, or the expansive soil shall be removed or stabilized around and beneath the structure according to SBC 303.

SECTION 5.3
FLOOD LOADS

The provisions of this section apply to buildings and other structures located in areas prone to flooding as defined on a flood hazard map as shown in Figure 5-1.

5.3.1 **Definitions.** The following definitions apply to the provisions of Section 5.3.

Approved. Acceptable to the authority having jurisdiction.

Base Flood. The flood having a 1% chance of being equaled or exceeded in any given year.

Base Flood Elevation (BFE). The elevation of flooding, including wave height, having a 1% chance of being equaled or exceeded in any given year.

Breakaway Wall. Any type of wall subject to flooding that is not required to provide structural support to a building or other structure, and that is designed and constructed such that, under base flood or lesser flood conditions, it will collapse in such a way that: (1) it allows the free passage of floodwaters, and (2) it does not damage the structure or supporting foundation system.

Coastal A-Zone. An area within a Special Flood Hazard Area, landward of a V-Zone or landward of an open coast without mapped V-Zones. To be classified as a Coastal A-Zone, the principal source of flooding must be astronomical tides, storm surges, seiches, or tsunamis, not riverine flooding.

Coastal High-Hazard Area (V-ZONE). An area within a Special Flood Hazard Area, extending from offshore to the inland limit of a primary frontal dune along an open coast, and any other area that is subject to high-velocity wave action from storms or seismic sources.

Design Flood. The greater of the following two flood events: (1) the Base Flood, affecting those areas identified as Special Flood Hazard Areas or (2) the flood corresponding to the area designated as a Flood Hazard Area or otherwise legally designated.

Design Flood Elevation (DFE). The elevation of the Design Flood, including wave height, relative to the datum.

Flood Hazard Area. The area subject to flooding during the Design Flood.

Flood Hazard Map. The map delineating flood hazard areas adopted by the authority having jurisdiction.

Special Flood Hazard Area (Area of Special Flood Hazard). The land in the floodplain subject to a 1% or greater chance of flooding in any given year. These areas are designated as A-Zones or V-Zones.

5.3.2 Design Requirements.

5.3.2.1 Design Loads. Structural systems of buildings or other structures shall be designed, constructed, connected, and anchored to resist flotation, collapse, and permanent lateral displacement due to action of flood loads associated with the design flood (see Section 5.3.3) and other loads in accordance with the load combinations of Chapter 2.

5.3.2.2 Erosion and Scour. The effects of erosion and scour shall be included in the calculation of loads on buildings and other structures in flood hazard areas.

5.3.2.3 Loads on Breakaway Walls. Walls and partitions required by Ref. 5-1, to break away, including their connections to the structure, shall be designed for the largest of the following loads acting perpendicular to the plane of the wall:

1. The wind load specified in Chapters 6 and 7.
2. The earthquake load specified in Chapters 9 through 16.
3. 0.5 kN/m^2.

The loading at which breakaway walls are intended to collapse shall not exceed 1.0 kN/m^2 unless the design meets the following conditions:

1. Breakaway wall collapse is designed to result from a flood load less than that which occurs during the base flood; and

2. The supporting foundation and the elevated portion of the building shall be designed against collapse, permanent lateral displacement, and other structural damage due to the effects of flood loads in combination with other loads as specified in Chapter 2.

5.3.3 Loads during Flooding.

5.3.3.1 **Load Basis.** In flood hazard areas, the structural design shall be based on the design flood. The local flood elevation shall be determined by the authority having jurisdiction.

5.3.3.2 **Hydrostatic Loads.** Hydrostatic loads caused by a depth of water to the level of the design flood elevation shall be applied over all surfaces involved, both above and below ground level, except that for surfaces exposed to free water, the design depth shall be increased by 0.30 m.

Reduced uplift and lateral loads on surfaces of enclosed spaces below the design flood elevation shall apply only if provision is made for entry and exit of floodwater.

5.3.3.3 **Hydrodynamic Loads.** Dynamic effects of moving water shall be determined by a detailed analysis utilizing basic concepts of fluid mechanics.

Regression equations for flood magnitude are presented below for the three hydrologic regions of Saudi Arabia, as shown in Figure 5-2. The average velocity of water V can be obtained by dividing the flood magnitude by the vertical cross sectional area at where the flood is passing.

For Hydrologic Region 1:

$$Q_{50} = 14.40\ A^{0.49472} \qquad \textbf{(Eq. 5-1)}$$

For Hydrologic Region 2:

$$Q_{50} = 0.0594\ A^{0.617}\ E^{-1.22}\ P^{0.933} \qquad \textbf{(Eq. 5-2)}$$

For Hydrologic Region 3:

$$Q_{50} = 0.499\ A^{0.383}\ E^{-5.60} \qquad \textbf{(Eq. 5-3)}$$

where

Q_{50} = Flood magnitude in cubic metres per second for 50 years recurrence interval.

A = Drainage area in square kilometers.

E = Mean basin elevation in thousands of metres above mean sea level.

P = Mean annual precipitation in millimetres as per Table 5-2.

Exception: Where water velocities do not exceed 3.0 m/s, dynamic effects of moving water shall be permitted to be converted into equivalent hydrostatic loads by increasing the design flood elevation for design purposes by an equivalent surcharge depth, d_h, on the headwater side and above the ground level only, equal to:

$$d_h = \frac{aV^2}{2g} \qquad \textbf{(Eq. 5-4)}$$

where

V = average velocity of water in m/s

g = acceleration due to gravity, 9.81 m/s^2

a = coefficient of drag or shape factor (not less than 1.25)

The equivalent surcharge depth shall be added to the design flood elevation design depth and the resultant hydrostatic pressures applied to, and uniformly distributed across, the vertical projected area of the building or structure which is perpendicular to the flow. Surfaces parallel to the flow or surfaces wetted by the tailwater shall be subject to the hydro-static pressures for depths to the design flood elevation only.

5.3.3.4 **Wave Loads.** Wave loads shall be determined by one of the following three methods: (1) using the analytical procedures outlined in this section, (2) by more advanced numerical modeling procedures or, (3) by laboratory test procedures (physical modeling).

Wave loads are those loads that result from water waves propagating over the water surface and striking a building or other structure. Design and construction of buildings and other structures subject to wave loads shall account for the following loads: waves breaking on any portion of the building or structure; uplift forces caused by shoaling waves beneath a building or structure, or portion thereof; wave run-up striking any portion of the building or structure; wave-induced drag and inertia forces; wave-induced scour at the base of a building or structure, or its foundation. Wave loads shall be included for both V-Zones and A-Zones. In V-Zones, waves are (0.90 m) high, or higher; in coastal floodplains landward of the V-Zone, waves are less than high (0.90 m).

Nonbreaking and broken wave loads shall be calculated using the procedures described in Sections 5.3.3.2 and 5.3.3.3 to calculate hydrostatic and hydrodynamic loads.

Breaking waves loads shall be calculated using the procedures described in Sections 5.3.3.4.1 through 5.3.3.4.4. Breaking wave heights used in the procedures described in Sections 5.3.3.4.1 through 5.3.3.4.4 shall be calculated for V Zones and Coastal A Zones using Eqs. 5-5 and 5-6.

$$H_b = 0.78\ d_s \qquad \textbf{(Eq. 5-5)}$$

where

H_b = breaking wave height in m

d_s = local stillwater depth in m

The local stillwater depth shall be calculated using Eq. 5-5, unless more advanced procedures or laboratory tests permitted by this section are used.

$$d_s = 0.65(\mathrm{BFE}-\mathrm{G}) \qquad \textbf{(Eq. 5-6)}$$

where
BFE = Base Flood Elevation in m
G = Ground elevation in m

5.3.3.4.1 Breaking Wave Loads on Vertical Pilings and Columns. The net force resulting from a breaking wave acting on a rigid vertical pile or column shall be assumed to act at the stillwater elevation and shall be calculated by the following:

$$F_D = 0.5\ \gamma_w C_D D H_b^2 \qquad \textbf{(Eq. 5-7)}$$

where

F_D = net wave force, in kN.

γ_w = unit weight of water, in kN/m^3, = 9.80 kN/m^3 for fresh water and 10.05 kN/m^3 for salt water.

C_D = coefficient of drag for breaking waves, = 1.75 for round piles or columns, and = 2.25 for square piles or columns.

D = pile or column diameter, in m for circular sections, or for a square pile or column, 1.4 times the width of the pile or column in m.

H_b = breaking wave height, in m.

5.3.3.4.2 Breaking Wave Loads on Vertical Walls. Maximum pressures and net forces resulting from a normally incident breaking wave (depth-limited in size, with $H_b = 0.78$ d_s acting on a rigid vertical wall shall be calculated by the following:

$$P_{max} = C_p\ \gamma_w\ d_s + 1.2\ \gamma_w\ d_s \qquad \textbf{(Eq. 5-8)}$$

and

$$F_t = 1.1 C_p \gamma_w\ d_s^2\ + 2.4\ \gamma_w\ d_s^2 \qquad \textbf{(Eq. 5-9)}$$

where

P_{max}= maximum combined dynamic $(C_p \gamma_w d_s)$ and static $(1.2\ \gamma_w d_s)$ wave pressures, also referred to as shock pressures in kN/m^2

F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse or wave impact force in kN/m, acting near the still-water elevation

C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 5-3)

γ_w = unit weight of water, in kN/m^3, = 9.80 kN/m^3 for fresh water and 10.05 kN/m^3 for salt water

d_s = stillwater depth in m at base of building or other structure where the wave breaks.

This procedure assumes the vertical wall causes a reflected or standing wave against the waterward side of the wall with the crest of the wave at a height of 1.2 d_s above the stillwater level. Thus, the dynamic static and total pressure distributions against the wall are as shown in Figure 5-3.

This procedure also assumes the space behind the vertical wall is dry, with no fluid balancing the static component of the wave force on the outside of the wall. If free water exists behind the wall, a portion of the hydrostatic component of the wave pressure and force disappears (see Figure 5-4) and the net force shall be computed by Eq. 5-10 (the maximum combined wave pressure is still computed with Eq. 5-8).

$$F_t = 1.1\ C_p\ \gamma_w\ d_s^2\ + 1.9\gamma_w\ d_s^2 \qquad \textbf{(Eq. 5-10)}$$

where

F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse, of wave impact force in kN/m, acting near the stillwater elevation

C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 5-3)

γ_w = unit weight of water, in kN/m^3, = 9.80 kN/m^3 for fresh water and 10.00 kN/m^3 for salt water

d_s = stillwater depth in m at base of building or other structure where the wave breaks

5.3.3.4.3 **Breaking Wave Loads on Nonvertical Walls.** Breaking wave forces given by Eq. 5-9 and Eq. 5-10 shall be modified in instances where the walls or surfaces upon which the breaking waves act are nonvertical. The horizontal component of breaking wave force shall be given by:

$$F_{nv} = F_t \sin^2\ \alpha \qquad \textbf{(Eq. 5-11)}$$

where

F_{nv} = horizontal component of breaking wave force in kN/m

F_t = net breaking wave force acting on a vertical surface in kN/m

α = vertical angle between nonvertical surface and the horizontal

5.3.3.4.4 **Breaking Wave Loads from Obliquely Incident Waves.** Breaking wave forces given by Eq. 5-9 and Eq. 5-10 shall be modified in instances where waves are obliquely incident. Breaking wave forces from non-normally incident waves shall be given by:

$$F_{oi} = F_t\ \sin^2\ \alpha \qquad \textbf{(Eq. 5-12)}$$

where

F_{oi} = horizontal component of obliquely incident breaking wave force in kN/m

F_t = net breaking wave force (normally incident waves) acting on a vertical surface in kN/m

α = horizontal angle between the direction of wave approach and the vertical surface

5.3.3.4.5 **Impact Loads.** Impact loads are those that result from debris, and any object transported by floodwaters striking against buildings and structures, or parts thereof. Impact loads shall be determined using a rational approach as concentrated loads acting horizontally at the most critical location at or below the design flood elevation.

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(Map will be provided when available)

FIGURE 5-1
FLOOD HAZARD MAP

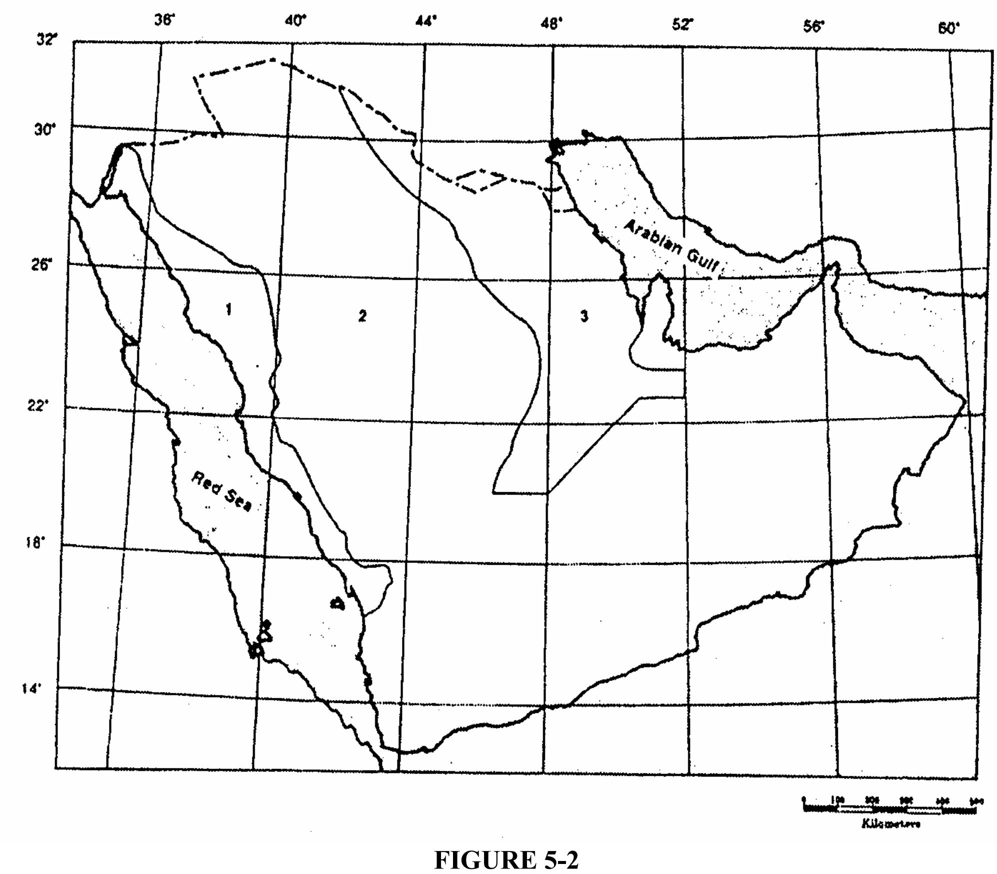


FIGURE 5-2
HYDROLOGIC REGIONS IN SAUDI ARABIA

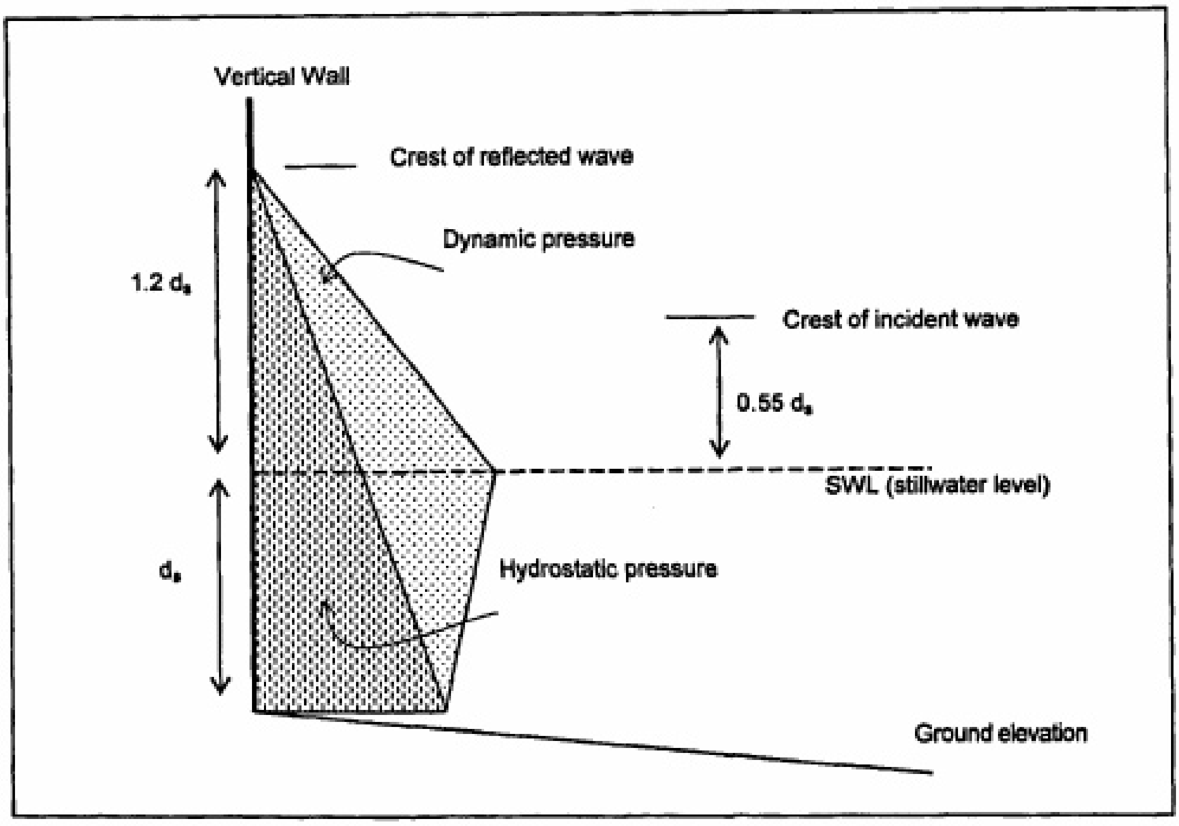


FIGURE 5-3
NORMALLY INCIDENT BREAKING WAVE PRESSURES AGAINST A VERTICAL WALL (SPACE BEHIND VERTICAL WALL IS DRY)

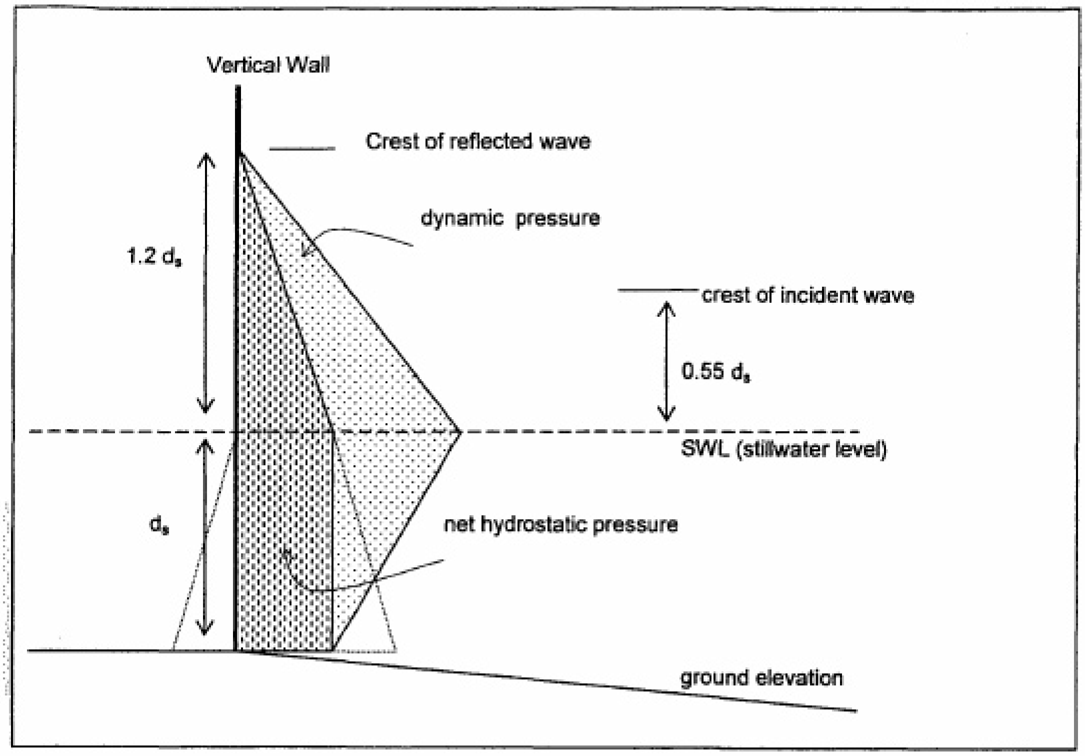


FIGURE 5-4
NORMALLY INCIDENT BREAKING WAVE PRESSURES AGAINST A VERTICAL WALL (STILLWATER LEVEL EQUAL ON BOTH SIDES OF WALL)

TABLE 5-1:
DESIGN LATERAL SOIL LOAD

Description of Backfill Material	Unified Soil Classification	Design Lateral Soil Load (Note A) $kN/m^2/m$ of depth
Well-graded, clean gravels; gravel-sand mixes	GW	5.50 Note C
Poorly graded clean gravels; gravel-sand mixes	GP	5.50 Note C
Silty gravels, poorly graded gravel-sand mixes	GM	5.50 Note C
Clayey gravels, poorly graded gravel-and-clay mixes	GC	7.0 Note C
Well-graded, clean sands; gravelly-sand mixes	SW	5.50 Note C
Poorly graded clean sands; sand-gravel mixes	SP	5.50 Note C
Silty sands, poorly graded sand-silt mixes	SM	7.0 Note C
Sand-silt clay mix with plastic fines	SM-SC	13.5 Note D
Clayey sands, poorly graded sand-clay mixes	SC	13.5 Note D
Inorganic silts and clayey silts	ML	13.5 Note D
Mixture of inorganic silt and clay	ML-CL	13.5 Note D
Inorganic clays of low to medium plasticity	CL	16
Organic silts and silt-clays, low plasticity	OL	Note B
Inorganic clayey silts, elastic silts	MH	Note B
Inorganic clays of high plasticity	CH	Note B
Organic clays and silty clays	OH	Note B

Note A. Design lateral soil loads are given for moist conditions for the specified soils at their optimum densities. Actual field conditions shall govern. Submerged or saturated soil pressures shall include the weight of the buoyant soil plus the hydrostatic loads.

Note B. Unsuitable as backfill material.

Note C. For relatively rigid walls, as when braced by floors, the design lateral soil load shall be increased for sand and gravel type soils to 9.5 kN/m^2 per metre of depth. Basement walls extending not more than 2.5 m below grade and supporting light floor systems are not considered as being relatively rigid walls.

Note D. For relatively rigid walls, as when braced by floors, the design lateral load shall be increased for silt and clay type soils to 16 kN/m^2 per metre of depth. Basement walls extending not more than 2.5 m below grade and supporting light floor systems are not considered as being relatively rigid walls.

TABLE 5-2:
RAINFALL PRECIPITATION

City (Region)	Mean Annual Precipitation 'P' (mm)
Abha	330
Abqaiq	81
Abu Sa'fah	**
Al-Baha	154
Al-Jauf	59
Ar'Ar' (Badana)	73
Berri	90
Dhahran	83
Duba (as Al-Wajih)	28
Hail	130
Rarad	40
Hawta	112
Hofuf	92
Jeddah	54
Jizan	120
Ju'aymah	89
Khamis Mushayt	220
Khurais	81
Medina	**
Marjan	**
Najran	64
Qasim	135
Qaisumah	121
Qatif	87
Rabigh	60
Ras Tanura	89
Riyadh	116
Safaniya	98
Shaybah	24
Shedgum	81
Tanajib	98
Tabuk	48
Turaif	78
Udhailiyah	**
Uthmaniyah	**
Yanbu	36

** Data Not Available

TABLE 5-3:
VALUE OF DYNAMIC PRESSURE COEFFICIENT, C_p

Building Category	C_p
I	1.6
II	2.8
III	3.2
IV	3.5

CHAPTER 6
WIND LOADS

SECTION 6.1
GENERAL

6.1.1 **Scope.** Buildings and other structures, including the main wind force-resisting system and all components and cladding thereof, shall be designed and constructed to resist wind loads as specified herein.

6.1.2 **Allowed Procedures.** The design wind loads for buildings and other structures, including the main wind force-resisting system and component and cladding elements thereof, shall be determined using one of the following procedures: (1) Method 1 – Simplified Procedure as specified in Section 7.1 for buildings meeting the requirements specified therein; (2) Method 2 – Analytical Procedure as specified in Section 7.2 for buildings meeting the requirements specified therein; (3) Method 3 – Wind Tunnel Procedure according to Section 7.3.

6.1.3 **Wind Pressures Acting on Opposite Faces of Each Building Surface.** In the calculation of design wind loads for the main wind force-resisting system and for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be taken into account.

6.1.4 **Minimum Design Wind Loading.** The design wind load, determined by any one of the procedures specified in Section 6.1.2, shall be not less than specified in this Section.

6.1.4.1 **Main Wind Force-Resisting System**

The wind load to be used in the design of the main wind force-resisting system for an enclosed or partially enclosed building or other structure shall not be less than 0.5 kN/m^2 multiplied by the area of the building or structure projected onto a vertical plane normal to the assumed wind direction. The design wind force for open buildings and other structures shall be not less than 0.5 kN/m^2 multiplied by the area A_f, as defined in section 6.3.

6.1.4.2 **Components and Cladding.**

The design wind pressure for components and cladding of buildings shall not be less than a net pressure of 0.5 kN/m^2 acting in either direction normal to the surface.

SECTION 6.2
DEFINITIONS

The following definitions apply only to the provisions of Chapters 6 and 7:

Approved. Acceptable to the authority having jurisdiction.

Basic Wind Speed, V. 3-second gust speed at 10 m above the ground in Exposure C (see Section 6.4.2) as determined in accordance with Section 6.4.1.

Building, Enclosed. A building that does not comply with the requirements for open or partially enclosed buildings.

Building Envelope. Cladding, roofing, exterior walls, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.

Building and Other Structure, Flexible. Slender buildings and other structures that have a fundamental natural frequency less than 1 Hz.

Building, Low-Rise. Enclosed or partially enclosed buildings that comply with the following conditions:

1. Mean roof height h is less than or equal to 18 m; and

2. Mean roof height h does not exceed least horizontal dimension.

Building, Open. A building having each wall at least 80% open. This condition is expressed for each wall by the equation $A_o > 0.8A_g$ where

A_o = total area of openings in a wall that receives positive external pressure, in m^2

A_g = the gross area of that wall in which A_o is identified, in m^2

Building, Partially Enclosed. A building that complies with the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10%, and

2. The total area of openings in a wall that receives positive external pressure exceeds 0.4 m^2 or 1% of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20%.

These conditions are expressed by the following equations:

1. $A_o > 1.10\ A_{oi}$

2. $A_o > 0.4\ m^2$ or $> 0.01\ A_g$, whichever is smaller, and $A_{oi}/A_{gi} \leq 0.20$

where

A_o, A_g are as defined for Open Building

A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o, in m^2.

A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in m^2.

Building or Other Structure, Regular Shaped. A building or other structure having no unusual geometrical irregularity in spatial form.

Building or Other Structures, Rigid. A building or other structure whose fundamental frequency is greater than or equal to 1 Hz.

Building, Simple Diaphragm. An enclosed or partially enclosed building in which wind loads are transmitted through floor and roof diaphragms to the vertical main wind force-resisting system.

Components and Cladding. Elements of the building envelope that do not qualify as part of the main wind force-resisting system.

Design Force, F. Equivalent static force to be used in the determination of wind loads for open buildings and other structures.

Design Pressure, p. Equivalent static pressure to be used in the determination of wind loads for buildings.

Effective Wind Area. The area used to determine GC_p. For component and cladding elements, the effective wind area in Figures 7.2-7 through 7.2-13 is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

Escarpment. Also known as scarp, with respect to topographic effects in Section 6.4.3, a cliff or steep slope generally separating two levels or gently sloping areas (see Figure 6.4-2).

Glazing. Glass or transparent or translucent plastic sheet used in windows, doors, skylights, or curtain walls.

Hill. With respect to topographic effects in Section 6.4.3, a land surface characterized by strong relief in any horizontal direction (see Figure 6.4-2).

Importance Factor, I. A factor that accounts for the degree of hazard to human life and damage to property.

Main Wind Force-Resisting System. An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.

Mean Roof Height, h. The average of the roof eave height and the height to the highest point on the roof surface, except that, for roof angles of less than or equal to 10 degrees, the mean roof height shall be the roof eave height.

Openings. Apertures or holes in the building envelope that allow air to flow through the building envelope and that are designed as "open" during design winds as defined by these provisions.

Recognized Literature. Published research findings and technical papers that are approved.

Ridge. With respect to topographic effects in Section 6.4.3, an elongated crest of a hill characterized by strong relief in two directions (see Figure 6.4-2).

SECTION 6.3
SYMBOLS AND NOTATIONS

The following symbols and notations apply only to the provisions of Chapters 6 and 7:

A	=	effective wind area, in m^2;
A_f	=	area of open buildings and other structures either normal to the wind direction or projected on a plane normal to the wind direction, in m^2;
A_g	=	the gross area of that wall in which A_o is identified, in m^2;
A_{gi}	=	the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in m^2;
A_o	=	total area of openings in a wall that receives positive external pressure, in m^2;
A_{oi}	=	the sum of the areas of openings in the building envelope (walls and roof) not including A_o, in m^2;
A_{og}	=	total area of openings in the building envelope in m^2;
a	=	width of pressure coefficient zone, in m;
B	=	horizontal dimension of building measured normal to wind direction, in m;
$\bar{b}$	=	mean hourly wind speed factor in Eq. 7.2-11 from Table 7.2-1;
$\hat{b}$	=	3-second gust speed factor from Table 7.2-1;
C_f	=	force coefficient to be used in determination of wind loads for other structures;
C_p	=	external pressure coefficient to be used in determination of wind loads for buildings;
c	=	turbulence intensity factor in Eq. 7.2-2 from Table 7.2-1;
D	=	diameter of a circular structure or member, in m;
D'	=	depth of protruding elements such as ribs and spoilers, in m;
G	=	gust effect factor;
G_f	=	gust effect factor for main wind force-resisting systems of flexible buildings and other structures;
GC_{pn}	=	combined net pressure coefficient for a parapet;
GC_p	=	product of external pressure coefficient and gust effect factor to be used in determination of wind loads for buildings;
GC_{pf}	=	product of the equivalent external pressure coefficient and gust effect factor to be used in determination of wind loads for main wind force-resisting system of low-rise buildings;
GC_{pi}	=	product of internal pressure coefficient and gust effect factor to be used in determination of wind loads for buildings;
g_Q	=	peak factor for background response in Eqs. 7.2-1 and 7.2-5;
g_R	=	peak factor for resonant response in Eq. 7.2-5;
g_v	=	peak factor for wind response in Eqs. 7.2-1 and 7.2-5;
H	=	height of hill or escarpment in Figure 6.4-2, in m;
h	=	mean roof height of a building or height of other structure, except that eave height shall be used for roof angle θ of less than or equal to 10 degrees, in m;
I	=	importance factor;
$I_{\bar{z}}$	=	intensity of turbulence from Eq. 7.2-2;
K_1, K_2, K_3	=	multipliers in Figure 6.4-2 to obtain K_{zt};
K_d	=	wind directionality factor in Table 6.4-1;
K_h	=	velocity pressure exposure coefficient evaluated at height $z = h$;

K_Z	=	velocity pressure exposure coefficient evaluated at height z;
K_{zt}	=	topographic factor;
L	=	horizontal dimension of a building measured parallel to the wind direction, in m;
L_h	=	distance upwind of crest of hill or escarpment in Figure 6.4-2 to where the difference in ground elevation is half the height of hill or escarpment, in m;
$L_{\bar{z}}$	=	integral length scale of turbulence, in m;
ℓ	=	integral length scale factor from Table 7.2-1, m;
M	=	larger dimension of sign, in m;
N	=	smaller dimension of sign, in m;
N_I	=	reduced frequency from Eq. 7.2-9;
n_1	=	building natural frequency, Hz;
p	=	design pressure to be used in determination of wind loads for buildings, in kN/m^2;
p_L	=	wind pressure acting on leeward face in Figure 7.2-5;
p_{net10}	=	net design wind pressure for exposure B at h = 10 m and I = 1.0 from Figure 7.1-2;
p_p	=	combined net pressure on a parapet from Eq. 7.2-17;
p_{s10}	=	simplified design wind pressure for exposure B at h = 10 m and I = 1.0 from Figure 7.1-1;
p_w	=	wind pressure acting on windward face in Figure 7.2-5;
Q	=	background response factor from Eq. 7.2-3;
q	=	velocity pressure, in kN/m^2;
q_h	=	velocity pressure evaluated at height z = h, kN/m^2;
q_i	=	velocity pressure for internal pressure determination;
q_p	=	velocity pressure at top of parapet;
q_z	=	velocity pressure evaluated at height z above ground, kN/m^2;
R	=	resonant response from Eq. 7.2-7;
R_B, R_h, R_L	=	values from Eq. 7.2-10a and 7.2-10b;
R_i	=	reduction factor from Eq. 7.2-13;
R_n	=	value from Eq. 7.2-8;
r	=	rise-to-span ratio for arched roofs;
V	=	basic wind speed in km/h. The basic wind speed corresponds to a 3-second gust speed at 10 m above ground in Exposure Category C;
V_i	=	unpartitioned internal volume m^3;
$\overline{V}_{\bar{z}}$	=	mean hourly wind speed at height $\bar{z}$, m/s;
W	=	width of building in Figures 7.2-8, and 7.2-10 A and B and width of span in Figures 7.2-9 and 7.2-11, in m;
X	=	distance to center of pressure from windward edge in Figure 7.2-14, in m;
x	=	distance upwind or downwind of crest in Figure 6.4-2, in m;
z	=	height above ground level, in m;
$\bar{z}$	=	equivalent height of structure, in m;
z_g	=	nominal height of the atmospheric boundary layer used in this standard. Values appear in Table 7.2-1;
z_{min}	=	exposure constant from Table 7.2-1;
α	=	3-sec gust speed power law exponent from Table 7.2-1
$\hat{\alpha}$	=	reciprocal of α from Table 7.2-1;
$\bar{\alpha}$	=	mean hourly wind speed power law exponent in Eq. 7.2-11 from Table 7.2-1;

β	=	damping ratio, percent critical for buildings or other structures;
ε	=	ratio of solid area to gross area for open sign, face of a trussed tower, or lattice structure;
λ	=	adjustment factor for building height and exposure from Figures 7.1-1 and 7.1-2;
$\bar{\varepsilon}$	=	integral length scale power law exponent in Eq. 7.2-4 from Table 7.2-1;
η	=	value used in Eq. 7.2-10a and 7.2-10b (see Section 7.2.7.2);
θ	=	angle of plane of roof from horizontal, in degrees;
υ	=	height-to-width ratio for solid sign.

SECTION 6.4
BASIC WIND PARAMETERS

6.4.1 **Basic Wind Speed.** The basic wind speed, V, used in the determination of design wind loads on buildings and other structures shall be taken as shown in Figure 6.4-1 except as provided in Sections 6.4.1.1 and 6.4.1.2. The wind shall be assumed to come from any horizontal direction.

6.4.1.1 **Special Wind Regions.** The basic wind speed shall be increased where records or experience indicate that the wind speeds are higher than those reflected in Figure 6.4-1. Mountainous terrain, gorges, and special regions shall be examined for unusual wind conditions. The authority having jurisdiction shall adjust basic wind speed to account for higher local wind speeds. Such adjustment shall be based on meteorological information and an estimate of the basic wind speed obtained in accordance with the provisions of Section 6.4.1.2.

6.4.1.2 **Estimation of Basic Wind Speeds from Regional Climatic Data.** Regional climatic data shall only be used in lieu of the basic wind speeds given in Figure 6.4-1 when: (1) approved extreme-value statistical-analysis procedures have been employed in reducing the data; and (2) the length of record, sampling error, averaging time, anemometer height, data quality, and terrain exposure of the anemometer have been taken into account.

6.4.1.3 **Wind Directionality Factor.** The wind directionality factor, K_d, shall be determined from Table 6.4-1. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4.

6.4.2 **Exposure.** For each wind direction considered, an exposure category that adequately reflects the characteristics of ground roughness and surface irregularities shall be determined for the site at which the building or structure is to be constructed. Account shall be taken of variations in ground surface roughness that arises from natural topography and vegetation as well as constructed features.

6.4.2.1 **Wind Directions and Sectors.** For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building or structure shall be determined for the two upwind sectors extending 45 degrees either side of the selected wind direction. The exposures in these two sectors shall be determined in accordance with Sections 6.4.2.2 and 6.4.2.3 and the exposure resulting in the highest wind loads shall be used to represent the winds from that direction.

TABLE 6.4-1:
WIND DIRECTIONALITY FACTOR, K_d

Structure Type	Directionality Factor K_d*
Buildings Main Wind Force Resisting System Components and Cladding	 0.85 0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures Square Hexagonal Round	 0.90 0.95 0.95
Solid Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers Triangular, square, rectangular All other cross sections	 0.85 0.95

* Directionality Factor K_d has been calibrated with combinations of loads specified in Chapter 2. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4.

6.4.2.2 **Surface Roughness Categories.** A ground surface roughness within each 45-degree sector shall be determined for a distance upwind of the site as defined in Section 6.4.2.3 from the categories defined below, for the purpose of assigning an exposure category as defined in Section 6.4.2.3.

Surface Roughness B: Urban and suburban areas, wooded areas or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C: Open terrain with scattered obstructions having heights generally less than 10 m.

Surface Roughness D: Flat, unobstructed areas and water surfaces.

6.4.2.3 **Exposure Categories.** Exposure B: Exposure B shall apply where the ground surface roughness condition, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 800 m or 10 times the height of the building, whichever is greater.

Exception: For buildings whose mean roof height is less than or equal to 10 m, the upwind distance may be reduced to 450 m.

Exposure C: Exposure C shall apply for all cases where exposures B or D do not apply.

Exposure D: Exposure D shall apply where the ground surface roughness, as defined by surface roughness D, prevails in the upwind direction for a distance at least 1500 m or 10 times the building height, whichever is greater. Exposure D shall extend inland from the shoreline for a distance of 200 m or 10 times the height of the building, whichever is greater.

For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used.

6.4.3 Topographic Effects.

6.4.3.1 Wind Speed-Up over Hills, Ridges, and Escarpments. Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography, located in any exposure category, shall be included in the design when buildings and other site conditions and locations of structures meet all of the following conditions:

1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (100 H) or 3 km, whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined;
2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 3 km radius in any quadrant by a factor of two or more;
3. The structure is located as shown in Figure 6.4-2 in the upper half of a hill or ridge or near the crest of an escarpment;
4. $H/L_h \geq 0.2$; and
5. H is greater than or equal to 4.5m for Exposures C and D and 18 m for Exposure B.

6.4.3.2 Topographic Factor. The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt}:

$$K_{zt} = (1 + K_1K_2K_3)^2 \qquad \textbf{(Eq. 6.4-1)}$$

where K_1, K_2, and K_3 are given in Figure 6.4-2.

SECTION 6.5
IMPORTANCE FACTOR

An importance factor, I, for the building or other structure shall be determined from Table 6.5-1 based on building and structure categories defined in Table 1.6-1.

SECTION 6.6
ENCLOSURE CLASSIFICATIONS

6.6.1 General. For the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open as defined in Section 6.2.

6.6.2 Openings.

A determination shall be made of the amount of openings in the building envelope in order to determine the enclosure classification as defined in Section 6.6.1.

6.6.3 Multiple Classifications.

If a building by definition complies with both the “open” and “partially enclosed” definitions, it shall be classified as an “open” building. A building that does not comply with either the “open” or “partially enclosed” definitions shall be classified as an “enclosed” building.

TABLE 6.5-1: IMPORTANCE FACTOR, I

Category[1]	Regions with V = 136-160 km/h	Regions with V > 160 km/h
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

1. The building and structure classification categories are defined in Table 1.6-1.

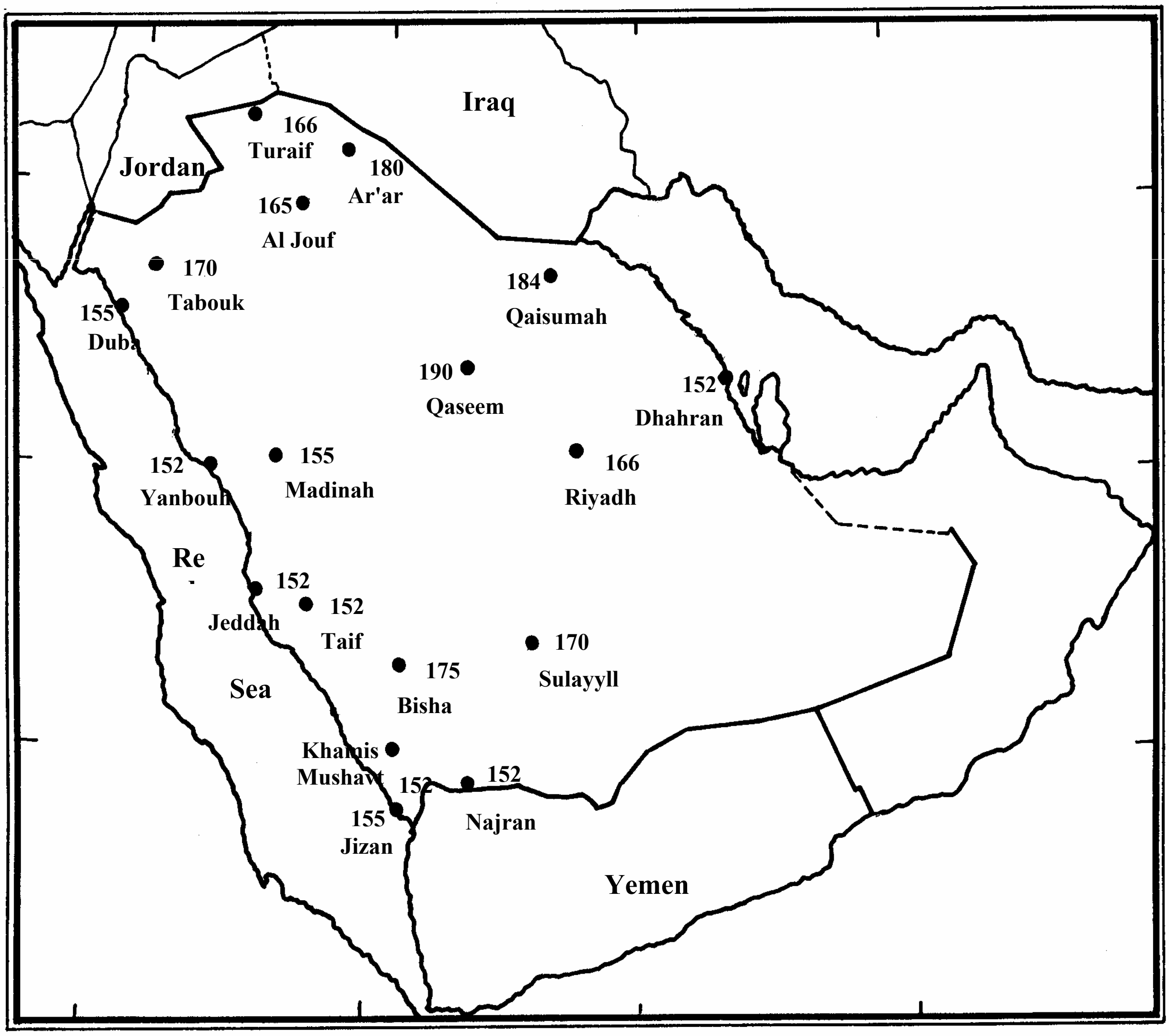
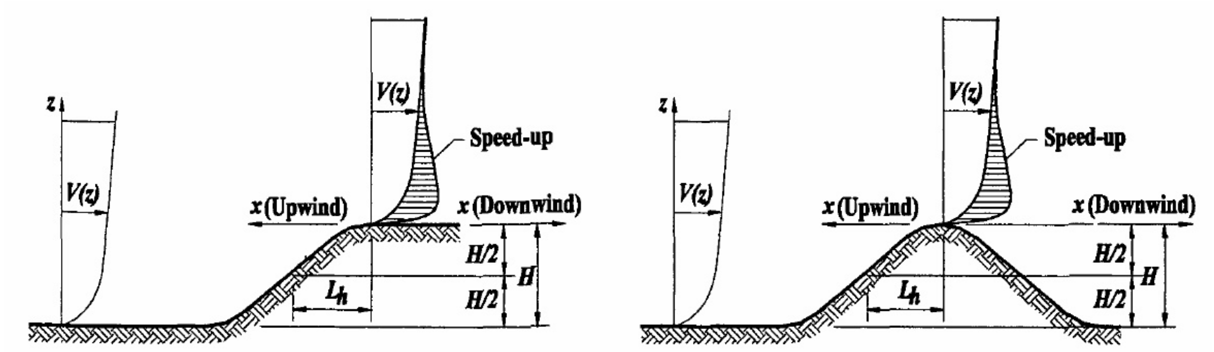


FIGURE 6.4-1
BASIC 3-SECOND GUST WIND SPEED IN km/h FOR SELECTED CITIES OF SAUDI ARABIA. ADOPTED FROM SAUDI ARAMCO DATA SAES A-112.



ESCARPMENT **2-D RIDGE OR 3-D AXISYMMETRICAL HILL**

Topographic Multipliers for Exposure C										
H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2-D Ridge	2-D Escarp.	3-D Axisym. Hill		2-D Escarp.	All Other Cases		2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute 2H for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:
   - H: Height of hill or escarpment relative to the upwind terrain, in metres.
   - L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in metres.
   - K_1: Factor to account for shape of topographic feature and maximum speed-up effect.
   - K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.
   - K_3: Factor to account for reduction in speed-up with height above local terrain.
   - x: Distance (upwind or downwind) from the crest to the building site, in metres.
   - z: Height above local ground level, in metres.
   - μ : Horizontal attenuation factor.
   - γ : Height attenuation factor.

FIGURE 6.4-2
TOPOGRAPHIC FACTOR, K_{zt}

Equations:

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

K_1 is determined from table below

$$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$$

$$K_3 = e^{-\gamma z / L_h}$$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	$K_1/(H/L_h)$			γ	μ	
	Exposure				Upwind of Crest	Downwind of Crest
	B	C	D			
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. hill	0.95	1.05	1.15	4	1.5	1.5

FIGURE 6.4-2 – cont'd
TOPOGRAPHIC FACTOR, K_{zt}

CHAPTER 7
DESIGN WIND LOAD PROCEDURES

SECTION 7.1
METHOD 1 - SIMPLIFIED PROCEDURE

7.1.1 **Scope.** A building whose design wind loads are determined in accordance with this Section shall meet all the conditions of Section 7.1.1.1 or 7.1.1.2. If a building qualifies only under Section 7.1.1.2 for design of its components and cladding, then its main wind force-resisting system shall be designed by Method 2.

7.1.1.1 **Main Wind Force-Resisting Systems.**

For the design of main wind force-resisting systems, the building must meet all of the following conditions:

1. The building is a simple diaphragm building as defined in Section 6.2,
2. The building is a low-rise building as defined in Section 6.2,
3. The building is enclosed as defined in Section 6.2,
4. The building is a regular shaped building or structure as defined in Section 6.2, and has an approximately symmetrical cross section in each direction with either a flat roof, or a gable or hip roof with $\theta \leq 45$ degrees,
5. The building is not classified as a flexible building as defined in Section 6.2,
6. The building does not have response characteristics making it subject to across-wind loading, vortex shedding, instability due to galloping or flutter; and does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration,
7. The building structure has no expansion joints or separations,
8. The building is not subject to the topographic effects of Section 6.4.3 (i.e., $K_{zt} = 1.0$).

7.1.1.2 **Components and Cladding.** For the design of components and cladding the building must meet all the following conditions:

1. The mean roof height $h < 18$ m,
2. The building is enclosed as defined in Section 6.2,
3. The building is a regular shaped building or structure as defined in Section 6.2,
4. The building does not have response characteristics making it subject to across-wind loading, vortex shedding, instability due to galloping or flutter; and does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration,
5. The building is not subject to the topographic effects of Section 6.4.3 (i.e., $K_{zt} = 1.0$),
6. The building has either a flat roof, or a gable roof with $\theta \leq 45$ degrees, or a hip roof with $\theta \leq 27$ degrees.

7.1.2 Design Procedure.

1. The basic wind speed V shall be determined in accordance with Section 6.4.1. The wind shall be assumed to come from any horizontal direction.
2. An exposure category shall be determined in accordance with Section 6.4.2.
3. An importance factor I shall be determined in accordance with Section 6.5.
4. A height and exposure adjustment coefficient, λ, shall be determined from Figure 7.1-1.

7.1.2.1 Main Wind Force-Resisting System. Simplified design wind pressures, p_s, for the main wind force-resisting systems of low-rise simple diaphragm buildings represent the net pressures (sum of internal and external) to be applied to the horizontal and vertical projections of building surfaces as shown in Figure 7.1-1. For the horizontal pressures (Zones A, B, C, D), p_s is the combination of the windward and leeward net pressures. p_s shall be determined by the following equation

$$p_s = \lambda \, I \, p_{s10} \qquad \textbf{(Eq. 7.1-1)}$$

Where

λ = adjustment factor for building height and exposure from Figure 7.1-1.

I = importance factor as defined in Section 6.5.

p_{s10} = simplified design wind pressure for exposure B, at h = 10 m, and for I = 1.0, from Figure 7.1-1.

7.1.2.1.1 Minimum Pressures. The load effects of the design wind pressures from Section 7.1.2.1 shall not be less than the minimum load case from Section 6.1.4.1 assuming the pressures, p_s, for Zones A, B, C, and D all equal to +0.5 kN/m^2, while assuming Zones E, F, G, and H all equal to 0 kN/m^2.

7.1.2.1.2 Pressures in Concrete Buildings. In reinforced concrete buildings the additional edge pressure in Zones A, B, E, and F may be neglected and a uniform pressure as computed for Zones C, D, G, and H can be taken across the entire width.

7.1.2.2 Components and Cladding. Net design wind pressures p_{net}, for the components and cladding of buildings designed using Method 1 represent the net pressures (sum of internal and external) to be applied normal to each building surface as shown in Figure 7.1-2. p_{net} shall be determined by the following equation:

$$p_{net} = \lambda \, I \, p_{net10} \qquad \textbf{(Eq. 7.1-2)}$$

Where

λ = adjustment factor for building height and exposure from Figure 7.1-2.

I = importance factor as defined in Section 6.5.

p_{net10} = net design wind pressure for exposure B, at h = 10 m and for I = 1.0, from Figure 7.1-2.

7.1.2.2.1 Minimum Pressures. The positive design wind pressures, p_{net}, from Section 7.1.2.2 shall not be less than +0.5 kN/m^2, and the negative design wind pressures, p_{net}, from 7.1.2.2 shall not be less than -0.5 kN/m^2.

7.1.3 Air-Permeable Cladding. Design wind loads determined from Figure 7.1-2 shall be used for all air-permeable cladding unless approved test data or recognized literature demonstrate lower loads for the type of air-permeable cladding being considered.

SECTION 7.2
METHOD 2 – ANALYTICAL PROCEDURE

7.2.1 **Scope.** A building or other structure whose design wind loads are determined in accordance with this Section shall meet all of the following conditions:

1. The building or other structure is a regular shaped building or structure as defined in Section 6.2, and
2. The building or other structure does not have response characteristics making it subject to across-wind loading, vortex shedding, instability due to galloping or flutter; or does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

7.2.2 **Limitations.** The provisions of Section 7.2 take into consideration the load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings or other structures. Buildings or other structures not meeting the requirements of Section 7.2.1, or having unusual shapes or response characteristics shall be designed using recognized literature documenting such wind load effects or shall use the wind tunnel procedure according to Section 7.3.

7.2.2.1 **Shielding.** There shall be no reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

7.2.2.2 **Air-Permeable Cladding.** Design wind loads determined from Section 7.2 shall be used for air-permeable cladding.

7.2.3 **Design Procedure.**

1. The basic wind speed V and wind directionality factor K_d shall be determined in accordance with Section 6.4.1.
2. An exposure category or exposure categories shall be determined for each wind direction in accordance with Section 6.4.2.
3. A topographic factor K_{zt} shall be determined in accordance with Section 6.4.3.
4. An importance factor I shall be determined in accordance with Section 6.5.
5. An enclosure classification shall be determined in accordance with Section 6.6.
6. Velocity pressure exposure coefficient K_z or K_h, as applicable, shall be determined for each wind direction in accordance with Section 7.2.6.
7. A gust effect factor G or G_f, as applicable, shall be determined in accordance with Section 7.2.7.
8. Velocity pressure q_z or q_h, as applicable, shall be determined in accordance with Section 7.2.8.

9. Internal pressure coefficient GC_{pi} shall be determined in accordance with Section 7.2.9.1.

10. External pressure coefficients C_p or GC_{pf}, or force coefficients C_f, as applicable, shall be determined in accordance with Section 7.2.9.2 or 7.2.9.3, respectively.

11. Design wind load p or F shall be determined in accordance with Sections 7.2.10 and 7.2.11, as applicable.

7.2.4 Exposure Category for Main Wind Force-Resisting Systems.

7.2.4.1 Buildings and Other Structures. For each wind direction considered, wind loads for the design of the main wind force-resisting system (MWFRS) determined from Figure 7.2-2 shall be based on the exposure categories defined in Section 6.4.2.3.

7.2.4.2 Low-Rise Buildings. Wind loads for the design of the main wind force-resisting systems for low-rise buildings shall be determined using a velocity pressure q_h based on the exposure resulting in the highest wind loads for any wind direction at the site when external pressure coefficients GC_{pf} given in Figure 7.2-6 are used.

7.2.5 Exposure Category for Components and Cladding.

7.2.5.1 Buildings with Mean Roof Height h Less Than or Equal to 18 m. Components and cladding for buildings with a mean roof height h of 18 m or less shall be designed using a velocity pressure q_h based on the exposure resulting in the highest wind loads for any wind direction at the site.

7.2.5.2 Buildings with Mean Roof Height h Greater Than 18 m and Other Structures. Components and cladding for buildings with a mean roof height h in excess of 18 m and for other structures shall be designed using the exposure resulting in the highest wind loads for any wind direction at the site.

7.2.6 Velocity Pressure Exposure Coefficient. Based on the exposure category determined in Section 6.4.2.3, a velocity pressure exposure coefficient K_z or K_h, as applicable, shall be determined from Table 7.2-2.

7.2.7 Gust Effect Factor.

7.2.7.1 Rigid Structures. For rigid structures as defined in Section 6.2, the gust effect factor shall be taken as 0.85 or calculated by the formula:

$$G = 0.925\left(\frac{\left(1+1.7 g_Q I_{\bar{z}} Q\right)}{1+1.7 g_v I_{\bar{z}}}\right) \qquad \textbf{(Eq. 7.2-1)}$$

$$I_{\bar{z}} = c\left(10/\bar{z}\right)^{1/6} \qquad \textbf{(Eq. 7.2-2)}$$

where $I_{\bar{z}}$ = the intensity of turbulence at height $\bar{z}$ where $\bar{z}$ = the equivalent height of the structure defined as 0.6 h but not less than z_{min} for all building heights h. z_{min} and c are listed for each exposure in Table 7.2-1; g_Q and g_v shall be taken as 3.4. The background response Q is given by

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \qquad \textbf{(Eq. 7.2-3)}$$

where *B, h* are defined in Section 6.3; and $L_{\bar{z}}$ = the integral length scale of turbulence at the equivalent height given by

$$L_{\bar{z}} = \ell(\bar{z}/10)^{\bar{\varepsilon}} \qquad \textbf{(Eq. 7.2-4)}$$

in which ℓ and $\bar{\varepsilon}$ are constants listed in Table 7.2-1.

7.2.7.2 Flexible or Dynamically Sensitive Structures. For flexible or dynamically sensitive structures as defined in Section 6.2, the gust effect factor shall be calculated by:

$$G_f = 0.925\left(\frac{\left(1 + 1.7\, I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}\right)}{1 + 1.7 g_v I_{\bar{z}}}\right) \qquad \textbf{(Eq. 7.2-5)}$$

g_Q and g_v shall be taken as 3.4 and g_R is given by

$$g_R = \sqrt{2\, ln\,(3600\, n_1)} + \frac{0.577}{\sqrt{2\, ln\,(3600\, n_1)}} \qquad \textbf{(Eq. 7.2-6)}$$

R, the resonant response factor, is given by

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B\,(0.53 + 0.47\, R_\ell)} \qquad \textbf{(Eq. 7.2-7)}$$

$$R_n = \frac{7.47\, N_1}{(1 + 10.3\, N_1)^{5/3}} \qquad \textbf{(Eq. 7.2-8)}$$

$$N_1 = \frac{n_1\, L_{\bar{z}}}{\bar{V}_{\bar{z}}} \qquad \textbf{(Eq. 7.2-9)}$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) \quad for \;\; \eta > 0 \qquad \textbf{(Eq. 7.2-10a)}$$

$$R_\ell = 1 \quad for \;\; \eta = 0 \qquad \textbf{(Eq. 7.2-10b)}$$

where the subscript ℓ in Eq. 7.2-10a and Eq. 7.2-10b shall be taken as h, B, and L respectively.

n_1 = building natural frequency;

R_ℓ = R_h setting $\eta = 4.6\, n_1\, h/\bar{V}_{\bar{z}}$;

R_ℓ = R_B setting $\eta = 4.6\, n_1\, B/\bar{V}_{\bar{z}}$;

R_ℓ = R_L setting $\eta = 15.4\, n_1\, L/\overline{V}_{\bar{z}}$;

β = damping ratio, percent of critical h, B, L are defined in Section 6.3; and

$\overline{V}_{\bar{z}}$ = mean hourly wind speed (m/sec) at height $\bar{z}$ determined from Eq.7.2-11.

$$\overline{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{\alpha}} V\left(\frac{1}{3.6}\right) \qquad \textbf{(Eq. 7.2-11)}$$

where $\bar{b}$ and $\bar{\alpha}$ are constants listed in Table 7.2-1 and V is the basic wind speed in km/h.

7.2.7.3 **Limitations** Where combined gust effect factors and pressure coefficients (GC_p, GC_{pi}, GC_{pf}) are given in figures and tables, the gust effect factor shall not be determined separately.

7.2.8 Velocity Pressure.

Velocity pressure, q_z evaluated at height z shall be calculated by the following equation:

$$q_z = 0.0473 \times 10^{-3}\, K_z K_{zt} K_d V^2 I \quad (\text{kN/m}^2);\ V \text{ in km/h} \qquad \textbf{(Eq. 7.2-12)}$$

where
K_d is the wind directionality factor defined in Section 6.4.1, K_z is the velocity pressure exposure coefficient defined in Section 7.2.6 and K_{zt} is the topographic factor defined in Section 6.4.3, and q_h is the velocity pressure calculated using Eq. 7.2-12 at mean roof height h.

7.2.9 Pressure and Force Coefficients.

7.2.9.1 **Internal Pressure Coefficient.** Internal pressure coefficients, GC_{pi} shall be determined from Figure 7.2-1 based on building enclosure classifications determined from Section 6.6.

7.2.9.1.1 **Reduction Factor for Large Volume Buildings.** For a partially enclosed building containing a single, unpartitioned large volume, the internal pressure coefficient, GC_{pi}, shall be multiplied by the following reduction factor, R_i:

$R_i = 1.0$
or

$$R_i = 0.5\left(1 + \frac{1}{\sqrt{1 + \dfrac{V_i}{6950 A_{og}}}}\right) \le 1.0 \qquad \textbf{(Eq. 7.2-13)}$$

where

A_{og} = total area of openings in the building envelope (walls and roof, in m^2)

V_i = unpartitioned internal volume, in m^3

7.2.9.2 External Pressure Coefficients.

7.2.9.2.1 Main Wind Force-Resisting Systems. External pressure coefficients for main wind force resisting systems C_p, are given in Figures 7.2-2, 7.2-3, and 7.2-4. Combined gust effect factor and external pressure coefficients, GC_{pf}, are given in Figure 7.2-6 for low-rise buildings. The pressure coefficient values and gust effect factor in Figure 7.2-6 shall not be separated.

7.2.9.2.2 Components and Cladding. Combined gust effect factor and external pressure coefficients for components and cladding GC_p, are given in Figures 7.2-7 through 7.2-13. The pressure coefficient values and gust effect factor shall not be separated.

7.2.9.3 Force Coefficients.

Force coefficients C_f are given in Figures 7.2-14 through 7.2-18.

7.2.9.4 Roof Overhangs.

7.2.9.4.1 Main Wind Force-Resisting System. Roof overhangs shall be designed for a positive pressure on the bottom surface of windward roof overhangs corresponding to $C_p = 0.8$ in combination with the pressures determined from using Figures 7.2-2 and 7.2-6.

7.2.9.4.2 Components and Cladding. For all buildings, roof overhangs shall be designed for pressures determined from pressure coefficients given in Figure 7.2-7 B, C, and D.

7.2.9.5 Parapets.

7.2.9.5.1 Main Wind Force-Resisting System. The pressure coefficients for the effect of parapets on the MWFRS loads are given in Section 7.2.10.2.4

7.2.9.5.2 Components and Cladding. The pressure coefficients for the design of parapet component and cladding elements are taken from the wall and roof pressure coefficients as specified in Section 7.2.10.4.4.

7.2.10 Design Wind Loads on Enclosed and Partially Enclosed Buildings.

7.2.10.1 General.

7.2.10.1.1 Sign Convention. Positive pressure acts toward the surface and negative pressure acts away from the surface.

7.2.10.1.2 Critical Load Condition. Values of external and internal pressures shall be combined algebraically to determine the most critical load.

7.2.10.1.3 Tributary Areas Greater than 65 m^2. Component and cladding elements with tributary areas greater than 65 m^2 shall be permitted to be designed using the provisions for main wind force resisting systems.

7.2.10.2 Main Wind Force-Resisting Systems.

7.2.10.2.1 Rigid Buildings of All Height. Design wind pressures for the main wind force-resisting system of buildings of all heights shall be determined by the following equation:

$$p = qGC_p - q_i(GC_{pi}) \quad (kN/m^2) \qquad \textbf{(Eq. 7.2-14)}$$

where

q = q_z for windward walls evaluated at height z above the ground;

q = q_h for leeward walls, side walls, and roofs, evaluated at height h;

q_i = q_h for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings;

q_i = q_z for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$);

G = gust effect factor from Section 7.2.7;

C_p = external pressure coefficient from Figure 7.2-2 or 7.2-4;

(GC_{pi}) = internal pressure coefficient from Figure 7.2-1

q and q_i = shall be evaluated using exposure defined in Section 6.4.2.3. Pressure shall be applied simultaneously on windward and leeward walls and on roof surfaces as defined in Figures 7.2-2 and 7.2-5.

7.2.10.2.2 Low-Rise Building. Alternatively, design wind pressures for the main wind force-resisting system of low-rise buildings shall be determined by the following equation:

$$p = q_h [(GC_{pf}) - (GCp_i)] \quad (kN/m^2) \qquad \textbf{(Eq. 7.2-15)}$$

where

q_h = velocity pressure evaluated at mean roof height h using exposure defined in Section 6.4.2.3;

(GC_{pf}) = external pressure coefficient from Figure 7.2-6;

(GC_{pi}) = internal pressure coefficient from Figure 7.2-1.

7.2.10.2.3 Flexible Buildings. Design wind pressures for the main wind force-resisting system of flexible buildings shall be determined from the following equation:

$$p = qG_f\, C_p – q_i\, (GC_{pi}) \quad (kN/m^2) \qquad \textbf{(Eq. 7.2-16)}$$

where q, q_i, C_p and (GC_{pi}) are as defined in Section 7.2.10.2.1 and, G_f = gust effect factor defined in Section 7.2.7.2

7.2.10.2.4 Parapets. The design wind pressure for the effect of parapets on main wind force-resisting systems of rigid, low-rise or flexible buildings with flat, gable, or hip roofs shall be determined by the following equation:

$$p_p = q_pGC_{pn} \quad (kN/m^2) \qquad \textbf{(Eq. 7.2-17)}$$

where

p_p = combined net pressure on the parapet due to the combination of the net pressures from the front and back parapet surfaces. Plus (and minus) signs signify net pressure acting toward (and away from) the front (exterior) side of the parapet.

q_p = velocity pressure evaluated at the top of the parapet.

GC_{pn} = combined net pressure coefficient

= +1.8 for windward parapet

= -1.1 for leeward parapet

7.2.10.3 **Design Wind Load Cases.** The main wind force-resisting system of buildings of all heights, whose wind loads have been determined under the provisions of Sections 7.2.10.2.1 and 7.2.10.2.3, shall be designed for the wind load cases as defined in Figure 7.2-5. The eccentricity e for rigid structures shall be measured from the geometric center of the building face and shall be considered for each principal axis (e_x, e_y). The eccentricity e for flexible structures shall be determined from the following equation and shall be considered for each principal axis (e_x, e_y):

$$e = \frac{e_Q + 1.7\, I_{\bar{z}} \sqrt{\left(g_Q Q e_Q\right)^2 + \left(g_R\, R e_R\right)^2}}{1.7\, I_{\bar{z}} \sqrt{\left(g_Q Q\right)^2 + \left(g_R R\right)^2}} \qquad \textbf{(Eq. 7.2-18)}$$

where

e_Q = eccentricity e as determined for rigid structures in Figure 7.2-5

e_R = distance between the elastic shear center and center of mass of each floor

$I_{\bar{z}}$, g_Q, Q, g_R, R shall be as defined in Section 7.2.7

The sign of the eccentricity e shall be plus or minus, whichever causes the more severe load effect.

Exception: One-story buildings with h less than or equal to 10 m, buildings two stories or less framed with light-framed construction and buildings two stories or less designed with flexible diaphragms need only be designed for Load Case 1 and Load Case 3 in Figure 7.2-5.

7.2.10.4 **Components and Cladding.**

7.2.10.4.1 **Low-Rise Buildings and Buildings with h < 18 m.** Design wind pressures on component and cladding elements of low-rise buildings and buildings with h ≤ 18 m shall be determined from the following equation:

$$p = q_h[(GC_p) - (GC_{pi})] \quad (\text{kN/m}^2) \qquad \textbf{(Eq. 7.2-19)}$$

where

q_h = velocity pressure evaluated at mean roof height h using exposure defined in Section 6.4.2.3;

(GC_p) = external pressure coefficients given in Figures 7.2-7 through 7.2-12; and

(GC_{pi}) = internal pressure coefficient given in Figure 7.2-1.

7.2.10.4.2 **Buildings with h > 18 m.** Design wind pressures on components and cladding for all buildings with h > 18 m shall be determined from the following equation:

$$p = q(GC_p) - q_i(GC_{pi}) \;\; (kN/m^2) \qquad \textbf{(Eq. 7.2-20)}$$

where

q = q_z for windward walls calculated at height z above the ground;

q = q_h for leeward walls, side walls, and roofs, evaluated at height h;

q_i = q_h for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings; and

q_i = q_z for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$);

(GCp) = external pressure coefficient from Figure 7.2-13;

(GCpi) = internal pressure coefficient from Figure 7.2-1.

q and q_i = shall be evaluated using exposure defined in Section 6.4.2.3.

7.2.10.4.3 Alternative Design Wind Pressure for Components and Cladding in Buildings with 18 m < h < 27 m.

Alternative to the requirements of Section 7.2.10.4.2, the design of components and claddings for building with a mean roof height greater than 18 m and less than 27 m, values from Figures 7.2-7 through 7.2-12 shall be used only if the height to width ratio is one or less (except as permitted by Note 6 of Figure 7.2-13) and Eq. 7.2-19 is used.

7.2.10.4.4 Parapets. The design wind pressure on the components and cladding elements of parapets shall be designed by the following equation:

$$p = q_p(GC_p - GC_{pi}) \qquad \textbf{(Eq. 7.2-21)}$$

where

q_p = velocity pressure evaluated at the top of the parapet;

GC_p = external pressure coefficient from Figures 7.2-7 through 7.2-13; and

GC_{pi} = internal pressure coefficient from Figure 7.2-1, based on the porosity of the parapet envelope

Two load cases shall be considered. Load Case A shall consist of applying the applicable positive wall pressure from Figure 7.2-7A or 7.2-13 to the front surface of the parapet while applying the applicable negative edge or corner zone roof pressure from Figure 7.2-7B through 7.2-13 to the back surface. Load Case B shall consist of applying the applicable positive wall pressure from Figure 7.2-7A or 7.2-13 to the back of the parapet surface, and applying the applicable negative wall pressure from Figure 7.2-7A or 7.2-13 to the front surface. Edge and corner zones shall be arranged as shown in Figures 7.2-7 through 7.2-13. GC_p shall be determined for appropriate roof angle and effective wind area from Figures 7.2-7 through 7.2-13. If internal pressure is present, both load cases should be evaluated under positive and negative internal pressure.

7.2.11 **Design Wind Loads on Open Buildings and Other Structures.** The design wind force for open buildings and other structures shall be determined by the following formula:

$$F = q_z G C_f A_f \quad \text{(kN)} \qquad \textbf{(Eq. 7.2-22)}$$

where

q_z = velocity pressure evaluated at height z of the centroid of area A_f using exposure defined in Section 6.4.2.3;

G = gust effect factor from Section 7.2.7;

C_f = net force coefficients from Figures 7.2-14 through 7.2-18; and

A_f = projected area normal to the wind except where C_f is specified for the actual surface area, m^2.

SECTION 7.3
METHOD 3 – WIND-TUNNEL PROCEDURE

7.3.1 **Scope.** Wind-tunnel tests shall be used where required by Section 7.2.2. Wind-tunnel testing shall be permitted in lieu of Methods 1 and 2 for any building or structure.

7.3.2 **Test Conditions.** Wind-tunnel tests, or similar tests employing fluids other than air, used for the determination of design wind loads for any building or other structure, shall be conducted in accordance with this section. Tests for the determination of mean and fluctuating forces and pressures shall meet all of the following conditions:

1. the natural atmospheric boundary layer has been modeled to account for the variation of wind speed with height;

2. the relevant macro (integral) length and micro length scales of the longitudinal component of atmospheric turbulence are modeled to approximately the same scale as that used to model the building or structure;

3. the modeled building or other structure and surrounding structures and topography are geometrically similar to their full-scale counterparts, except that, for low-rise buildings meeting the requirements of Section 7.2.1, tests shall be permitted for the modeled building in a single exposure site as defined in Section 6.4.2.1;

4. the projected area of the modeled building or other structure and surroundings is less than 8% of the test section cross-sectional area unless correction is made for blockage;

5. the longitudinal pressure gradient in the wind-tunnel test section is accounted for;

6. Reynolds number effects on pressures and forces are minimized; and

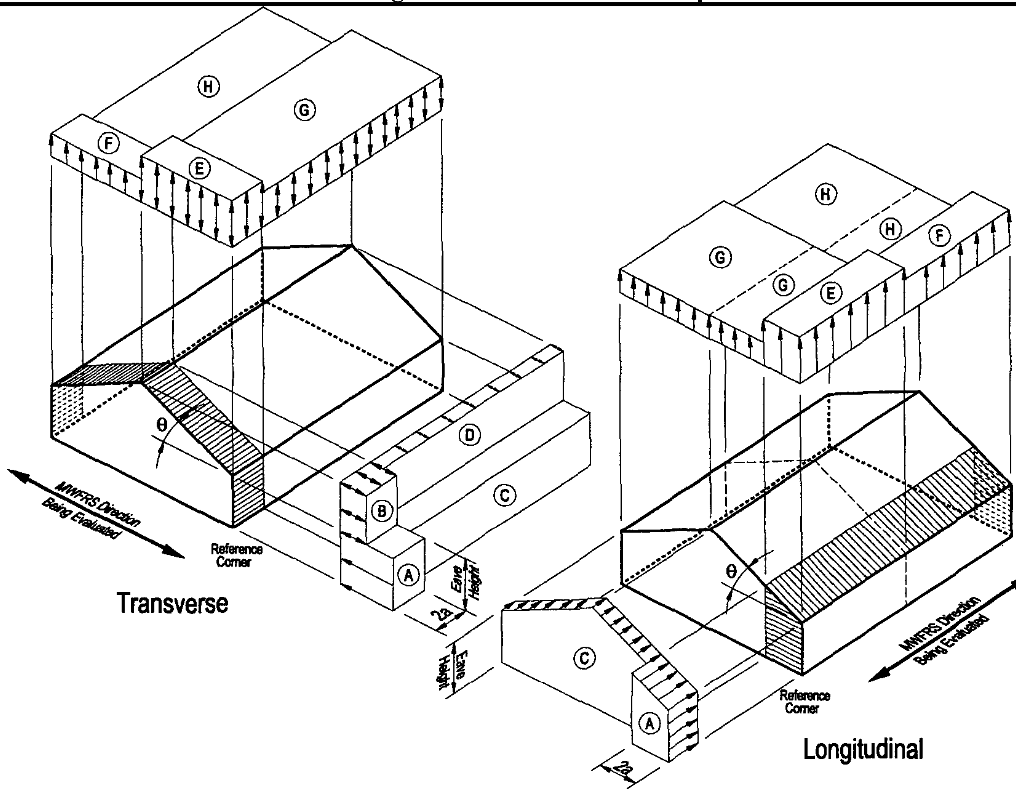
7. response characteristics of the wind-tunnel instrumentation are consistent with the required measurements.

7.3.3 **Dynamic Response.** Tests for the purpose of determining the dynamic response of a building or other structure shall be in accordance with Section 7.3.2. The structural model and associated analysis shall account for mass distribution, stiffness, and damping.

7.3.4 **Limitations.**

7.3.4.1 **Limitations on Wind Speeds.** Variation of basic wind speeds with direction shall not be permitted unless the analysis for wind speeds conforms to the requirements of Section 6.4.1.2.

Main Wind Force Resisting System - Method 1		$h \leq 18m$
Figure 7.1-1	**Design Wind Pressures**	**Walls & Roofs**
Enclosed Buildings		



Notes:

1. Pressures shown are applied to the horizontal and vertical projections, for exposure B, at h= 10 m, for I=1.0. Adjust to other exposures and heights with adjustment factor λ .
2. The load patterns shown shall be applied to each corner of the building in turn as the reference corner. (See Figure 7.2-6)
3. For the design of the longitudinal MWFRS use $\theta = 0°$, and locate the zone E/F, G/H boundary at the mid-length of the building.
4. Load cases 1 and 2 must be checked for 25°< $\theta \leq$ 45°. Load case 2 at 25° is provided only for interpolation between 25° to 30°.
5. Plus and minus signs signify pressures acting toward and away from the projected surfaces, respectively.
6. For roof slopes other than those shown, linear interpolation is permitted.
7. The total horizontal load shall not be less than that determined by assuming p_s= 0 in zones B & D.
8. The zone pressures represent the following:
   Horizontal pressure zones - Sum of the windward and leeward net (sum *of* internal and external) pressures on vertical projection of:
   - A - End zone of wall — C - Interior zone of wall
   - B - End zone of roof — D - Interior zone of roof

   Vertical pressure zones - Net (sum of internal and external) pressures on horizontal projection of:
   - E - End zone of windward roof — G - Interior zone of windward roof
   - F - End zone of leeward roof — H - Interior zone of leeward roof
9. Where zone E or G falls on a roof overhang on the windward side of the building, use E_{OH} and G_{OH} for the pressure on the horizontal projection of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied.
10. Notation:
    - a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
    - h: Mean roof height, in metres, except that eave height shall be used for roof angles <10°.
    - θ : Angle of plane of roof from horizontal, in degrees.

Main Wind Force Resisting System - Method 1		$h \leq 18m$
Figure 7.1-1 (Cont'd)	Design Wind Pressures	**Walls & Roofs**
Enclosed Buildings		

Simplified Design Wind Pressure , p_{s10} (kN/m^2) (Exposure B at h = 10 m with I = 1.0)

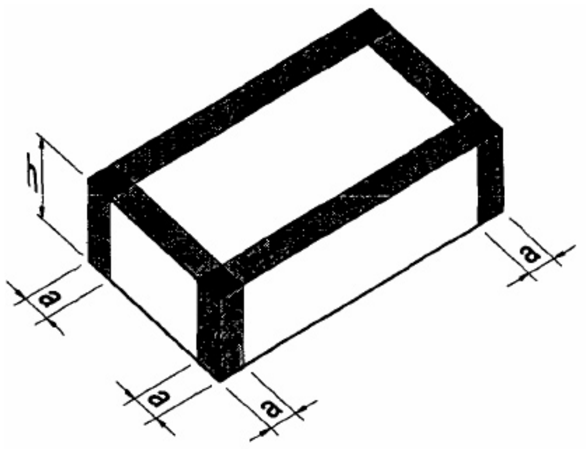
Basic Wind Speed (km/h)	Roof Angle (deg)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	E_{OH}	G_{OH}
135	0 to 5°	1	0.55	-0.28	0.36	-0.17	-0.66	-0.37	-0.46	-0.29	-0.92	-0.72
	10°	1	0.62	-0.26	0.41	-0.15	-0.66	-0.40	-0.46	-0.31	-0.92	-0.72
	15°	1	0.69	-0.23	0.46	-0.13	-0.66	-0.43	-0.46	-0.33	-0.92	-0.72
	20°	1	0.76	-0.20	0.51	-0.11	-0.66	-0.46	-0.46	-0.35	-0.92	-0.72
	25°	1	0.69	0.11	0.50	0.11	-0.31	-0.42	-0.22	-0.34	-0.57	-0.48
		2	—	—	—	—	-0.11	-0.23	-0.03	-0.14	—	—
	30° to 45°	1	0.62	0.42	0.49	0.34	0.05	-0.37	0.01	-0.32	-0.22	-0.25
		2	0.62	0.42	0.49	0.34	0.24	-0.19	0.21	-0.13	-0.22	-0.25
145	0 to 5°	1	0.61	-0.32	0.41	-0.19	-0.74	-0.42	-0.51	-0.33	-1.03	-0.81
	10°	1	0.69	-0.29	0.46	-0.17	-0.74	-0.45	-0.51	-0.34	-1.03	-0.81
	15°	1	0.77	-0.26	0.51	-0.14	-0.74	-0.48	-0.51	-0.37	-1.03	-0.81
	20°	1	0.85	-0.23	0.57	-0.12	-0.74	-0.51	-0.51	-0.39	-1.03	-0.81
	25°	1	0.77	0.12	0.56	0.13	-0.34	-0.47	-0.25	-0.37	-0.64	-0.55
		2	—	—	—	—	-0.13	-0.25	-0.03	-0.16	—	—
	30° to 45°	1	0.69	0.47	0.55	0.38	0.05	-0.42	0.02	-0.36	-0.24	-0.28
		2	0.69	0.47	0.55	0.38	0.27	-0.21	0.23	-0.15	-0.24	-0.28
160	0 to 5°	1	0.76	-0.39	0.50	-0.23	-0.91	-0.52	-0.64	-0.40	-1.28	-1.00
	10°	1	0.86	-0.35	0.57	-0.21	-0.91	-0.56	-0.64	-0.43	-1.28	-1.00
	15°	1	0.95	-0.32	0.64	-0.18	-0.91	-0.59	-0.64	-0.45	-1.28	-1.00
	20°	1	1.05	-0.28	0.70	-0.15	-0.91	-0.64	-0.64	-0.48	-1.28	-1.00
	25°	1	0.95	0.15	0.69	0.16	-0.42	-0.57	-0.31	-0.46	-0.79	-0.67
		2	—	—	—	—	-0.16	-0.32	-0.04	-0.20	—	—
	30° to 45°	1	0.85	0.58	0.68	0.47	0.07	-0.52	0.02	-0.45	-0.30	-0.34
		2	0.85	0.58	0.68	0.47	0.33	-0.25	0.28	-0.18	-0.30	-0.34
175	0 to 5°	1	0.92	-0.48	0.61	-0.28	-1.11	-0.63	-0.77	-0.48	-1.55	-1.21
	10°	1	1.03	-0.43	0.69	-0.25	-1.11	-0.68	-0.77	-0.52	-1.55	-1.21
	15°	1	1.15	-0.38	0.77	-0.22	-1.11	-0.72	-0.77	-0.55	-1.55	-1.21
	20°	1	1.27	-0.34	0.85	-0.19	-1.11	-0.77	-0.77	-0.58	-1.55	-1.21
	25°	1	1.15	0.19	0.83	0.19	-0.51	-0.70	-0.37	-0.56	-0.95	-0.81
		2	—	—	—	—	-0.20	-0.38	-0.05	-0.24	—	—
	30° to 45°	1	1.03	0.71	0.82	0.56	0.08	-0.63	0.03	-0.54	-0.36	-0.42
		2	1.03	0.71	0.82	0.56	0.40	-0.31	0.34	-0.22	-0.36	-0.42
190	0 to 5°	1	1.09	-0.57	0.72	-0.34	-1.31	-0.75	-0.91	-0.58	-1.84	-1.44
	10°	1	1.24	-0.51	0.82	-0.30	-1.31	-0.80	-0.91	-0.62	-1.84	-1.44
	15°	1	1.37	-0.45	0.91	-0.26	-1.31	-0.86	-0.91	-0.66	-1.84	-1.44
	20°	1	1.51	-0.40	1.01	-0.22	-1.31	-0.91	-0.91	-0.69	-1.84	-1.44
	25°	1	1.37	0.22	0.99	0.23	-0.61	-0.83	-0.44	-0.67	-1.13	-0.97
		2	—	—	—	—	-0.23	-0.45	-0.06	-0.29	—	—
	30° to 45°	1	1.23	0.84	0.98	0.67	0.10	-0.75	0.03	-0.64	-0.43	-0.49
		2	1.23	0.84	0.98	0.67	0.47	-0.37	0.41	-0.26	-0.43	-0.49
205	0 to 5°	1	1.28	-0.67	0.85	-0.39	-1.54	-0.88	-1.07	-0.68	-2.16	-1.69
	10°	1	1.45	-0.60	0.96	-0.35	-1.54	-0.94	-1.07	-0.72	-2.16	-1.69
	15°	1	1.61	-0.54	1.07	-0.31	-1.54	-1.01	-1.07	-0.77	-2.16	-1.69
	20°	1	1.78	-0.47	1.18	-0.26	-1.54	-1.07	-1.07	-0.81	-2.16	-1.69
	25°	1	1.61	0.26	1.16	0.26	-0.71	-0.98	-0.52	-0.79	-1.33	-1.13
		2	—	—	—	—	-0.27	-0.53	-0.07	-0.34	—	—
	30° to 45°	1	1.44		1.15	0.79	0.11	-0.88	0.04	-0.75	-0.51	-0.58
		2	1.44	0.99	1.15	0.79	0.56	-0.43	0.48	-0.31	-0.51	-0.58

Main Wind Force Resisting System - Method 1		$h \leq 18m$
Figure 7.1-1 (Cont'd)	**Design Wind Pressures**	**Walls & Roofs**
Enclosed Buildings		

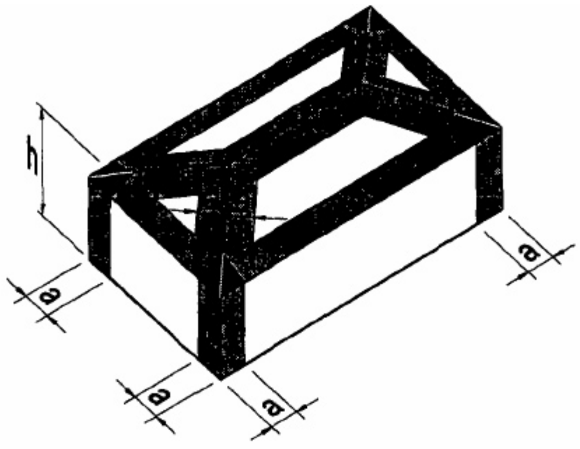
ADJUSTMENT FACTOR
FOR BUILDING HEIGHT AND EXPOSURE, λ

Mean roof height (m)	Exposure		
	B	C	D
4.5	1	1.21	1.47
6.0	1	1.29	1.55
7.5	1	1.35	1.61
9.0	1	1.4	1.66
10.5	1.05	1.45	1.7
12.0	1.09	1.49	1.74
13.5	1.12	1.53	1.78
15.0	1.16	1.56	1.81
16.5	1.19	1.59	1.84
18.0	1.22	1.62	1.87

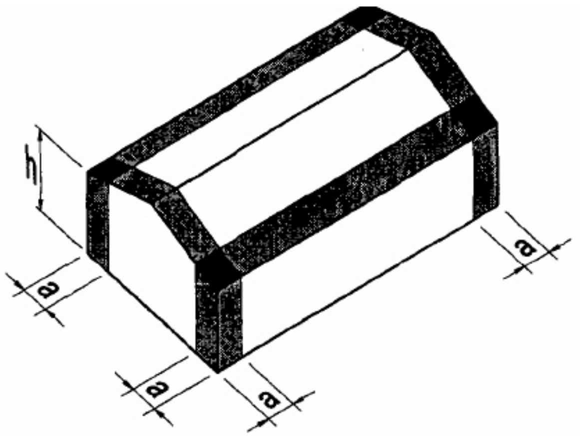
Components and Cladding - Method 1		$h \leq 18m$
Figure 7.1-2	**Design Wind Pressures**	**Walls & Roofs**
Enclosed Buildings		



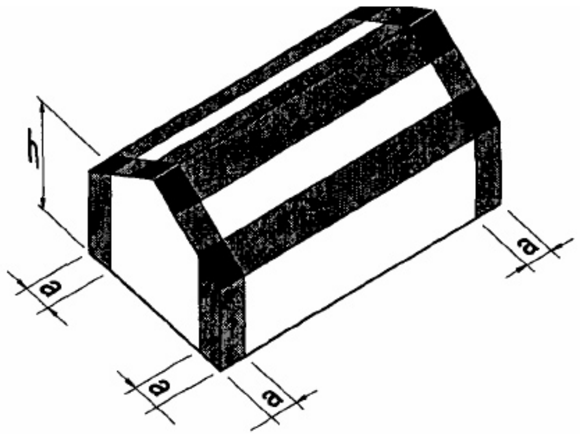
Flat Roof



Hip Roof ($7^o < \theta \leq 27^o$)



Gable Roof ($\theta \leq 7^o$)



Gable Roof ($7^o < \theta \leq 45^o$)

☐ **Interior Zones** — **Roofs-Zone 1 / Walls-Zone 4**

■ **End Zones** — **Roofs –Zone 2/ Walls – Zone 5**

■ **Corner Zones** — **Roofs - Zone 3**

Notes:

1. Pressures shown are applied normal to the surface, for exposure B, at h = 10 m, for I=1.0. Adjust to other exposures and heights with adjustment factor λ
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For hip roofs with $\theta \leq 25°$, Zone 3 shall be treated as Zone 2.
4. For effective wind areas between those given, value may be interpolated, otherwise use the value associated with the lower effective wind area.
5. Notation:
   - a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
   - h: Mean roof height, in metres, except that eave height shall be used for roof angles <10°.
   - θ : Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 1		$h \le 18m$
Figure 7.1-2 (Cont'd)	Design Wind Pressures	**Walls & Roofs**
Enclosed Buildings		

Net Design Wind Pressure , p_{net10} (kN/m^2) (Exposure B at h = 10 m with I = 1.0)

	Zone	Effective Wind area (m^2)	Basic Wind Speed V (km/h)											
			135		145		160		175		190		205	
Roof 0 to 7 degrees	1	1	0.25	-0.62	0.28	-0.70	0.35	-0.86	0.43	-1.04	0.50	-1.24	0.59	-1.46
	1	2	0.24	-0.61	0.27	-0.68	0.33	-0.84	0.40	-1.02	0.47	-1.21	0.56	-1.42
	1	5	0.22	-0.58	0.24	-0.66	0.30	-0.81	0.36	-0.98	0.43	-1.17	0.51	-1.37
	1	10	0.20	-0.57	0.23	-0.64	0.28	-0.79	0.34	-0.95	0.40	-1.13	0.47	-1.33
	2	1	0.25	-1.04	0.28	-1.17	0.35	-1.45	0.43	-1.75	0.50	-2.08	0.59	-2.44
	2	2	0.24	-0.93	0.27	-1.04	0.33	-1.29	0.40	-1.56	0.47	-1.86	0.56	-2.18
	2	5	0.22	-0.79	0.24	-0.88	0.30	-1.09	0.36	-1.32	0.43	-1.57	0.51	-1.84
	2	10	0.20	-0.68	0.23	-0.76	0.28	-0.93	0.34	-1.13	0.40	-1.35	0.47	-1.58
	3	1	0.25	-1.57	0.28	-1.76	0.35	-2.17	0.43	-2.63	0.50	-3.13	0.59	-3.68
	3	2	0.24	-1.30	0.27	-1.46	0.33	-1.80	0.40	-2.18	0.47	-2.60	0.56	-3.05
	3	5	0.22	-0.94	0.24	-1.06	0.30	-1.31	0.36	-1.58	0.43	-1.88	0.51	-2.21
	3	10	0.20	-0.68	0.23	-0.76	0.28	-0.93	0.34	-1.13	0.40	-1.35	0.47	-1.58
Roof > 7 to 27 degrees	1	1	0.36	-0.57	0.40	-0.64	0.50	-0.79	0.60	-0.95	0.71	-1.13	0.84	-1.33
	1	2	0.33	-0.56	0.37	-0.62	0.45	-0.77	0.55	-0.93	0.65	-1.10	0.77	-1.29
	1	5	0.29	-0.53	0.32	-0.60	0.39	-0.74	0.48	-0.89	0.57	-1.06	0.67	-1.24
	1	10	0.25	-0.52	0.28	-0.58	0.35	-0.71	0.43	-0.87	0.50	-1.03	0.59	-1.21
	2	1	0.36	-0.99	0.40	-1.11	0.50	-1.37	0.60	-1.66	0.71	-1.98	0.84	-2.32
	2	2	0.33	-0.91	0.37	-1.02	0.45	-1.26	0.55	-1.53	0.65	-1.82	0.77	-2.14
	2	5	0.29	-0.81	0.32	-0.90	0.39	-1.12	0.48	-1.35	0.57	-1.61	0.67	-1.89
	2	10	0.25	-0.73	0.28	-0.81	0.35	-1.01	0.43	-1.22	0.50	-1.45	0.59	-1.70
	3	1	0.36	-1.47	0.40	-1.64	0.50	-2.03	0.60	-2.46	0.71	-2.92	0.84	-3.43
	3	2	0.33	-1.37	0.37	-1.54	0.45	-1.90	0.55	-2.29	0.65	-2.73	0.77	-3.21
	3	5	0.29	-1.24	0.32	-1.39	0.39	-1.72	0.48	-2.08	0.57	-2.48	0.67	-2.91
	3	10	0.25	-1.15	0.28	-1.29	0.35	-1.59	0.43	-1.92	0.50	-2.29	0.59	-2.69
Roof > 27 to 45 degrees	1	1	0.57	-0.62	0.64	-0.70	0.79	-0.86	0.95	-1.04	1.13	-1.24	1.33	-1.46
	1	2	0.56	-0.59	0.62	-0.66	0.77	-0.82	0.93	-0.99	1.10	-1.18	1.29	-1.38
	1	5	0.53	-0.55	0.60	-0.61	0.74	-0.76	0.89	-0.92	1.06	-1.09	1.24	-1.28
	1	10	0.52	-0.52	0.58	-0.58	0.71	-0.71	0.87	-0.87	1.03	-1.03	1.21	-1.21
	2	1	0.57	-0.73	0.64	-0.81	0.79	-1.01	0.95	-1.22	1.13	-1.45	1.33	-1.70
	2	2	0.56	-0.69	0.62	-0.78	0.77	-0.96	0.93	-1.16	1.10	-1.39	1.29	-1.63
	2	5	0.53	-0.66	0.60	-0.73	0.74	-0.90	0.89	-1.10	1.06	-1.30	1.24	-1.53
	2	10	0.52	-0.62	0.58	-0.70	0.71	-0.86	0.87	-1.04	1.03	-1.24	1.21	-1.46
	3	1	0.57	-0.73	0.64	-0.81	0.79	-1.01	0.95	-1.22	1.13	-1.45	1.33	-1.70
	3	2	0.56	-0.69	0.62	-0.78	0.77	-0.96	0.93	-1.16	1.10	-1.39	1.29	-1.63
	3	5	0.53	-0.66	0.60	-0.73	0.74	-0.90	0.89	-1.10	1.06	-1.30	1.24	-1.53
	3	10	0.52	-0.62	0.58	-0.70	0.71	-0.86	0.87	-1.04	1.03	-1.24	1.21	-1.46
Wall	4	1	0.62	-0.68	0.70	-0.76	0.86	-0.93	1.04	-1.13	1.24	-1.35	1.46	-1.58
	4	2	0.59	-0.65	0.67	-0.72	0.82	-0.90	1.00	-1.08	1.18	-1.29	1.39	-1.51
	4	5	0.56	-0.61	0.62	-0.68	0.77	-0.84	0.93	-1.02	1.11	-1.22	1.30	-1.43
	4	10	0.53	-0.58	0.59	-0.65	0.73	-0.80	0.89	-0.98	1.05	-1.16	1.24	-1.36
	4	50	0.46	-0.52	0.52	-0.58	0.64	-0.71	0.78	-0.87	0.92	-1.03	1.09	-1.21
	5	1	0.62	-0.83	0.70	-0.93	0.86	-1.15	1.04	-1.39	1.24	-1.66	1.46	-1.95
	5	2	0.59	-0.78	0.67	-0.87	0.82	-1.08	1.00	-1.30	1.18	-1.55	1.39	-1.82
	5	5	0.56	-0.70	0.62	-0.79	0.77	-0.97	0.93	-1.18	1.11	-1.40	1.30	-1.64
	5	10	0.53	-0.65	0.59	-0.72	0.73	-0.90	0.89	-1.08	1.05	-1.29	1.24	-1.51
	5	50	0.46	-0.52	0.52	-0.58	0.64	-0.71	0.78	-0.87	0.92	-1.03	1.09	-1.21

Components and Cladding - Method 1		$h \leq 18m$
Figure 7.1-2 (Cont'd)	Design Wind Pressures	**Walls & Roofs**
Enclosed Buildings		

Roof Overhang Net Design Wind Pressure , p_{net10} (kN/m^2) (Exposure B at h = 10 m with I = 1.0)

	Zone	Effective Wind area (m^2)	Basic Wind Speed V (km/h)				
			145	160	175	190	205
Roof 0 to 7 degrees	2	1	-1.01	-1.24	-1.50	-1.79	-2.10
	2	2	-0.99	-1.22	-1.47	-1.76	-2.06
	2	5	-0.96	-1.19	-1.44	-1.71	-2.01
	2	10	-0.95	-1.17	-1.41	-1.68	-1.97
	3	1	-1.66	-2.04	-2.47	-2.94	-3.45
	3	2	-1.30	-1.60	-1.94	-2.31	-2.71
	3	5	-0.83	-1.02	-1.24	-1.47	-1.73
	3	10	-0.48	-0.58	-0.71	-0.84	-0.99
Roof > 7 to 27 degrees	2	1	-1.30	-1.60	-1.94	-2.31	-2.71
	2	2	-1.30	-1.60	-1.94	-2.31	-2.71
	2	5	-1.30	-1.60	-1.94	-2.31	-2.71
	2	10	-1.30	-1.60	-1.94	-2.31	-2.71
	3	1	-2.19	-2.70	-3.27	-3.89	-4.56
	3	2	-1.97	-2.44	-2.95	-3.51	-4.12
	3	5	-1.69	-2.09	-2.53	-3.01	-3.53
	3	10	-1.48	-1.82	-2.21	-2.63	-3.08
Roof > 27 to 45 degrees	2	1	-1.18	-1.46	-1.77	-2.10	-2.47
	2	2	-1.15	-1.42	-1.71	-2.04	-2.39
	2	5	-1.10	-1.36	-1.64	-1.95	-2.29
	2	10	-1.06	-1.31	-1.59	-1.89	-2.22
	3	1	-1.18	-1.46	-1.77	-2.10	-2.47
	3	2	-1.15	-1.42	-1.71	-2.04	-2.39
	3	5	-1.10	-1.36	-1.64	-1.95	-2.29
	3	10	-1.06	-1.31	-1.59	-1.89	-2.22

ADJUSTMENT FACTOR
FOR BUILDING HEIGHT AND EXPOSURE, λ

Mean roof height (m)	Exposure		
	B	C	D
4.5	1	1.21	1.47
6.0	1	1.29	1.55
7.5	1	1.35	1.61
9.0	1	1.4	1.66
10.5	1.05	1.45	1.7
12.0	1.09	1.49	1.74
13.5	1.12	1.53	1.78
15.0	1.16	1.56	1.81
16.5	1.19	1.59	1.84
18.0	1.22	1.62	1.87

Main Wind Force Resisting System / Components and Cladding - Method 2		All Heights
Figure 7.2-1	**Internal Pressure Coefficient, GC_{pi}**	**Walls & Roofs**
Enclosed, Partially Enclosed, and Open Buildings		

Enclosure Classification	GC_{pi}
Open Buildings	0.00
Partially Enclosed Buildings	+0.55 -0.55
Enclosed Buildings	+0.18 -0.18

Notes:

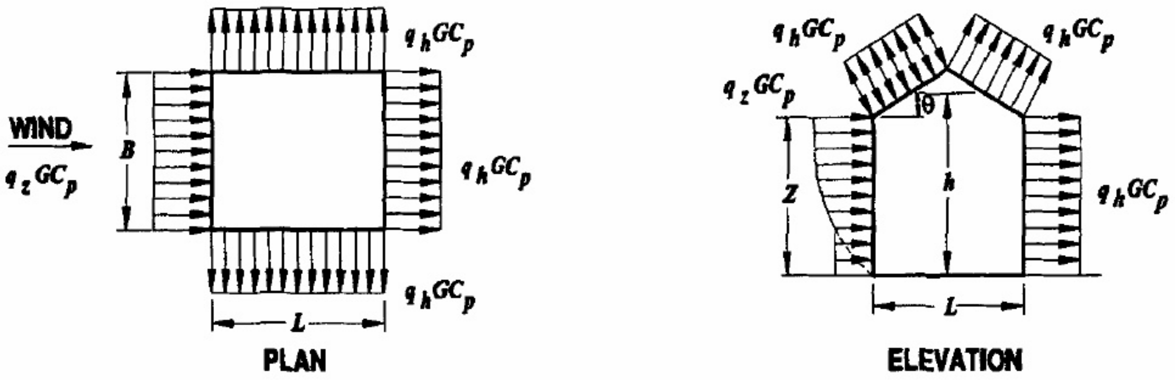
Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.

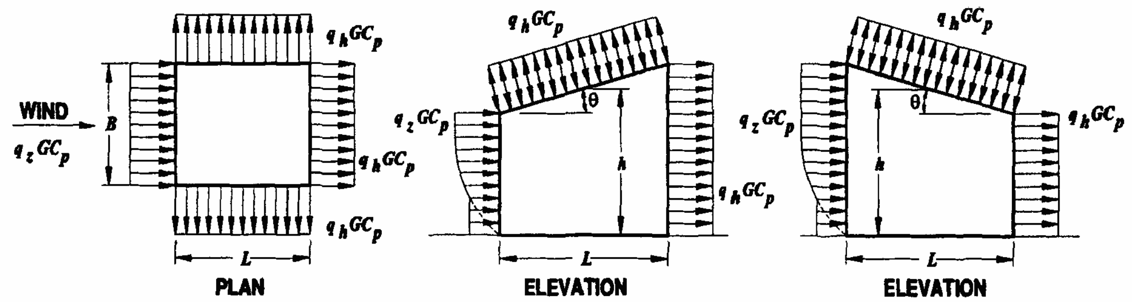
Values of GC_{pi} shall be used with q_Z or q_h, as specified in 7.2.10.

Two cases shall be considered to determine the critical load requirements for the appropriate condition:

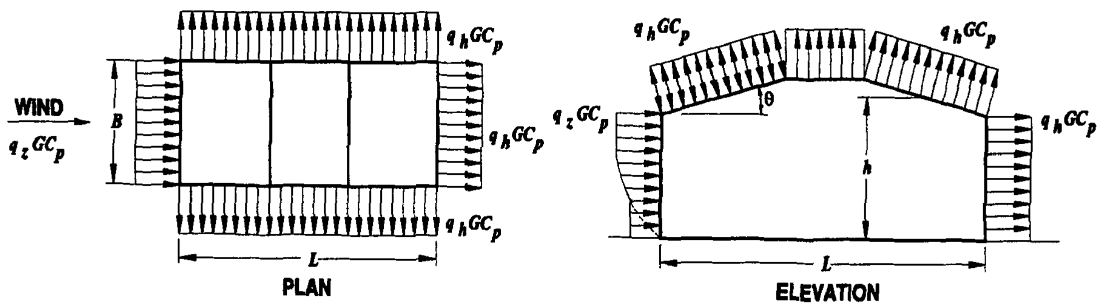
(i) a positive value of GC_{pi} applied to all internal surfaces
(ii) a negative value of GC_{pi} applied to all internal surfaces

Main Wind Force Resisting System - Method 2		All Heights
Figure 7.2-2	External Pressure Coefficient, C_p	**Walls & Roofs**
Enclosed, Partially Enclosed Buildings		





MONOSLOPE ROOF (NOTE 4)



MANSARD ROOF (NOTE 8)

Main Wind Force Resisting System Method 2		All Heights
Figure 7.2-2 (Cont'd)	External Pressure Coefficients, C_p	**Walls & Roofs**
Enclosed, Partially Enclosed Buildings		

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction		Windward								Leeward		
		Angle, θ (degrees)								Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal to ridge for $\theta \geq 10^0$	≤ 0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horizontal distance from windward edge	Cp
Normal to ridge for $\theta < 10^o$ and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3,-0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7,-0.18

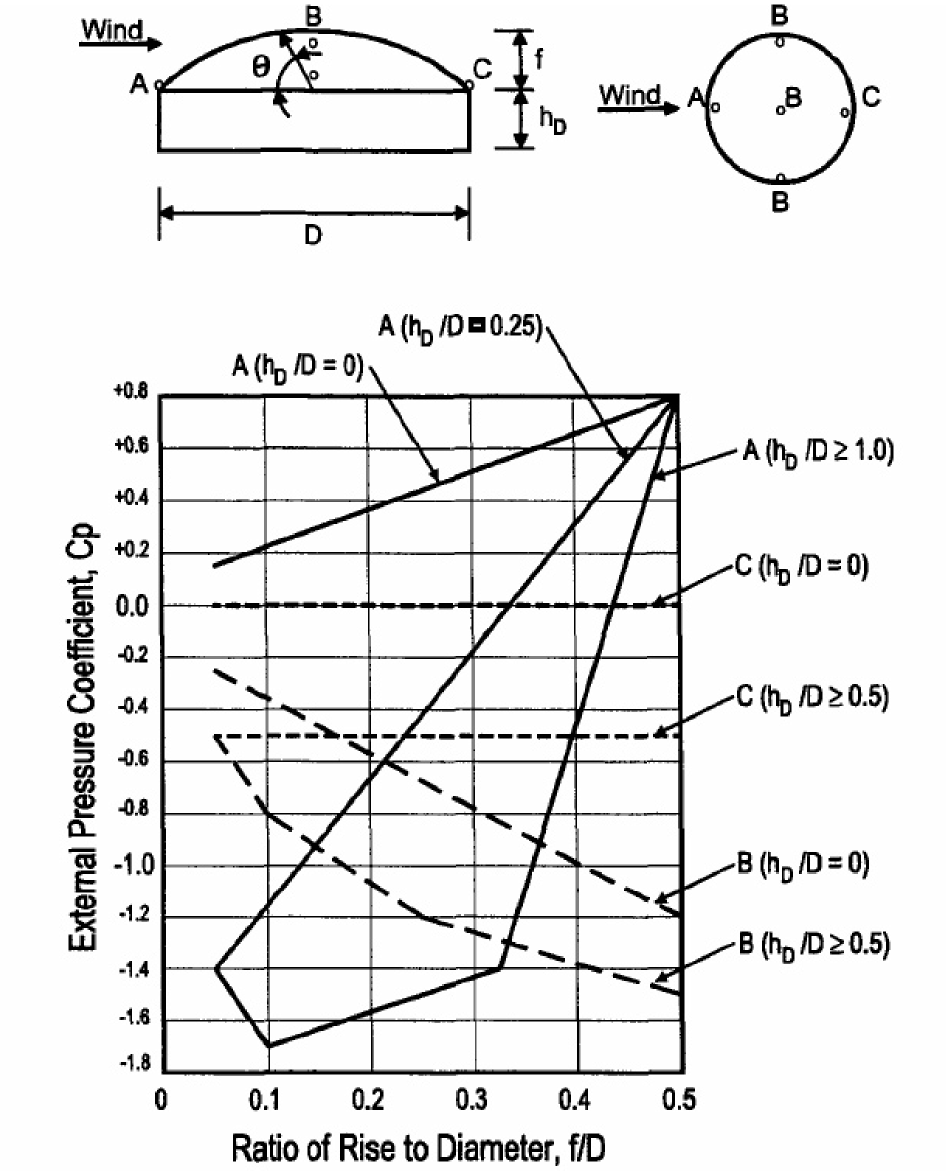
* Value is provided for interpolation purposes.
**Value can be reduced linearly with area over which it is applicable as follows

Area (sq m)	Reduction Factor
≤ 10 sq m	1.0
20 sq m	0.9
≥ 100 sq m	0.8

Notes
1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of *L/B, h/L* and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f *as* determined by Section 7.2.7.2.
6. Refer to Figure 7.2-3 for domes and Figure 7.2-4 for arched roofs.
7. Notation:
   - B: Horizontal dimension of building, in metre, measured normal to wind direction.
   - L: Horizontal dimension of building, in metre, measured parallel to wind direction.
   - h: Mean roof height in metres, except that eave height shall be used for $\theta \leq 10$ degrees.
   - z: Height above ground, in metres.
   - G: Gust effect factor.
   - q_z, q_h: Velocity pressure, in kN/m^2, evaluated at respective height.
   - θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

\# For roof slopes greater than 80°, use C_p = 0.8

Main Wind Force Resisting System - Method 2		All Heights
Figure 7.2-3	External Pressure Coefficient, C_p	**Domed Roofs**
Enclosed, Partially Enclosed Buildings and Structures		



External Pressure Coefficients for Domes with a Circular Base.
(Adapted from Eurocode, 1995)

1. Two load cases shall be considered:
   Case A. C_p values between A and B and between B and C shall be determined by linear interpolation along arcs on the dome parallel to the wind direction;
   Case B. C_p shall be the constant value of A for $\theta \leq 25$ degrees, and shall be determined by linear interpolation from 25 degrees to B and from B to C.
2. Values denote C_p to be used with q_{h_D+f} where h_D + f is the height at the top of the dome.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. C_p is constant on the dome surface for arcs of circles perpendicular to the wind direction; for example, the arc passing through B-B-B and all arcs parallel to B-B-B.
5. For values of h_D/D between those listed on the graph curves, linear interpolation shall be permitted.
6. $\theta = 0$ degrees on dome spring line, $\theta = 90$ degrees at dome center top point. f is measured from spring line to top.
7. The total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
8. For f/D values less than 0.05, use Figure 7.2-2.

Main Wind Force Resisting System - Method 2		All Heights
Figure 7.2-4	**External Pressure Coefficients, C_p**	**Arched Roofs**
Enclosed, Partially Enclosed Buildings and Structures		

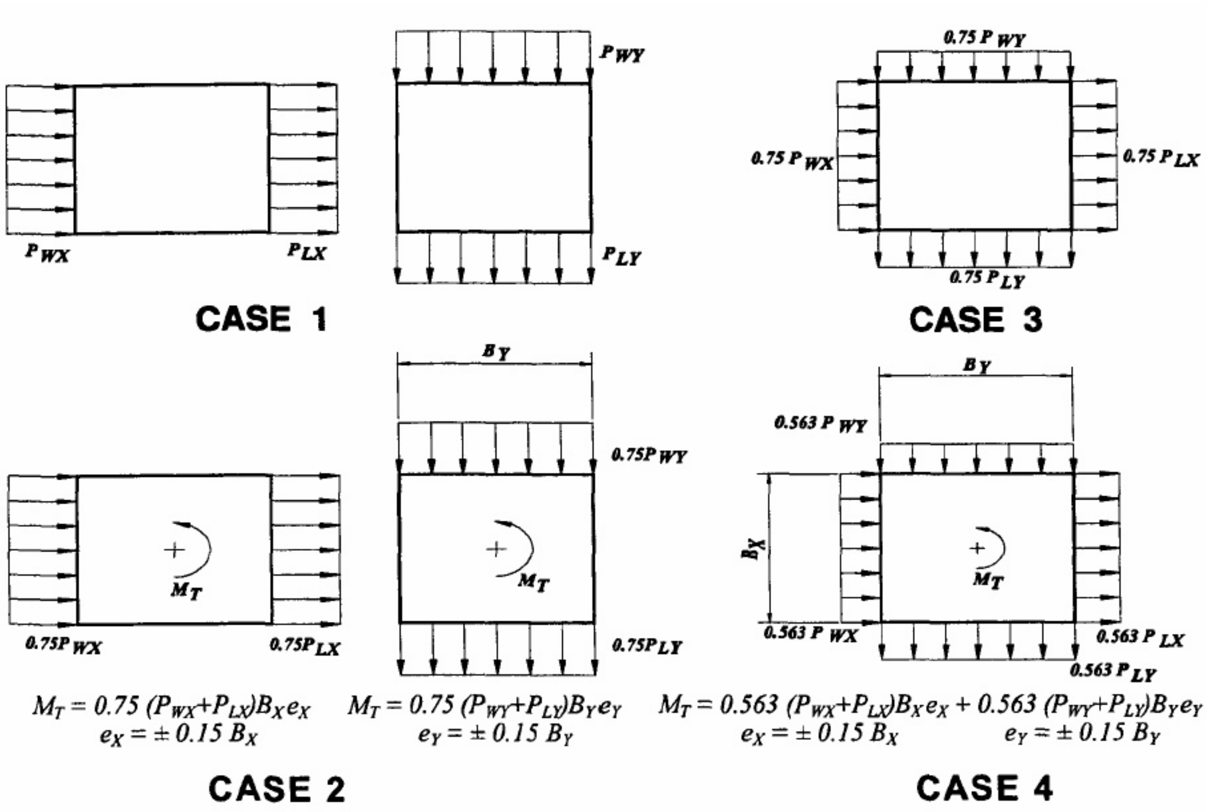
Conditions	**Rise-to-span ratio, r**	**C_P**		
		Windward quarter	**Center half**	**Leeward quarter**
Roof on elevated structure	$0 < r < 0.2$	-0.9	$-0.7 - r$	-0.5
	$0.2 \le r < 0.3$*	$1.5r - 0.3$	$-0.7 - r$	-0.5
	$0.3 \le r \le 0.6$	$2.75r - 0.7$	$-0.7 - r$	-0.5
Roof springing from ground level	$0 < r \le 0.6$	$1.4r$	$-0.7 - r$	-0.5

* When the rise-to-span ratio is $0.2 \le r \le 0.3$, alternate coefficients given by $6r - 2.1$ shall also be used for the windward quarter.

Notes:

1. Values listed are for the determination of average loads on main wind force resisting systems.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For wind directed parallel to the axis of the arch, use pressure coefficients from Fig. 7.2-2 with wind directed parallel to ridge.
4. For components and cladding: (1) At roof perimeter, use the external pressure coefficients in Fig. 7.2-7 with θ based on spring-line slope and (2) for remaining roof areas, use external pressure coefficients of this table multiplied by 0.87.

Main Wind Force Resisting System - Method 2		All Heights
Figure 7.2-5	**Design Wind Load Cases**	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

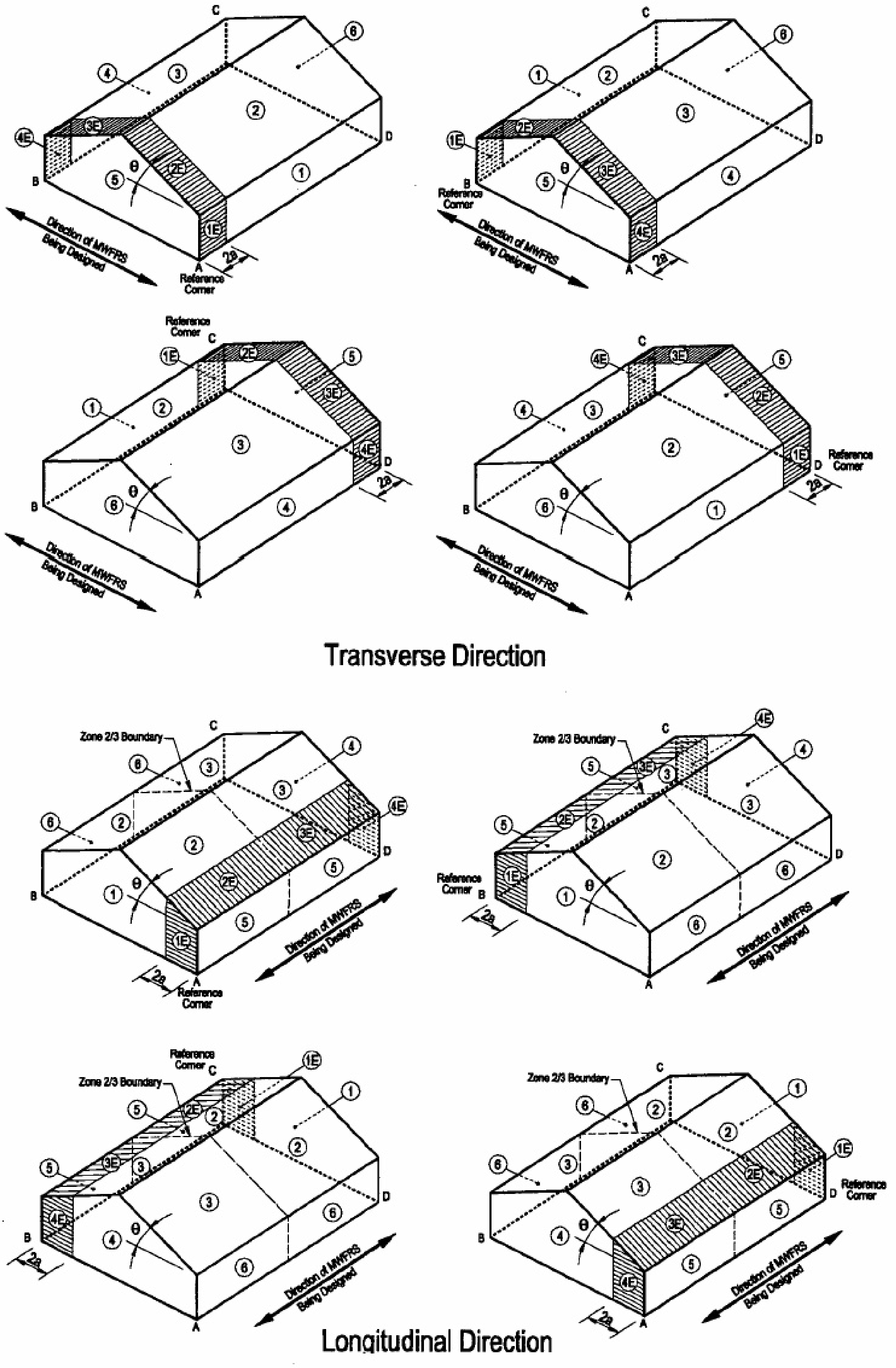
Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 7.2.10.2.1 and 7.2.10.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

   P_{wx}, P_{wy}: Windward face design pressure acting in the x, y principal axis, respectively.
   P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   e (e_x, e_y) : Eccentricity for the x, y principal axis of the structure, respectively.
   M_T: Torsional moment per unit height acting about a vertical axis of the building.

Main Wind Force Resisting System - Method 2		$h \leq 18m$
Figure 7.2-6	External Pressure Coefficients, GC_{pf}	Low-rise Walls & Roofs
Enclosed, Partially Enclosed Buildings		



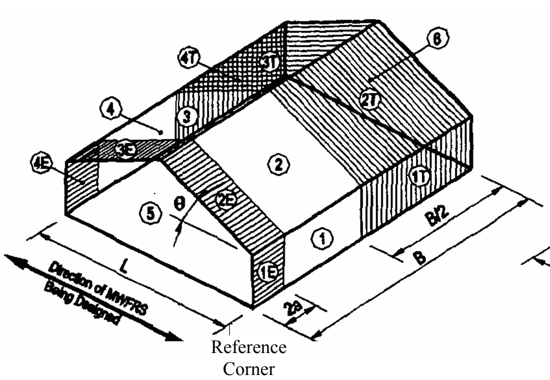
Basic Load Cases

Main Wind Force Resisting System - Method 2		$h \le 18m$
Figure 7.2-6 (Cont'd)	**External Pressure Coefficients, GC_{pf}**	**Low-rise Walls & Roofs**
Enclosed, Partially Enclosed Buildings		

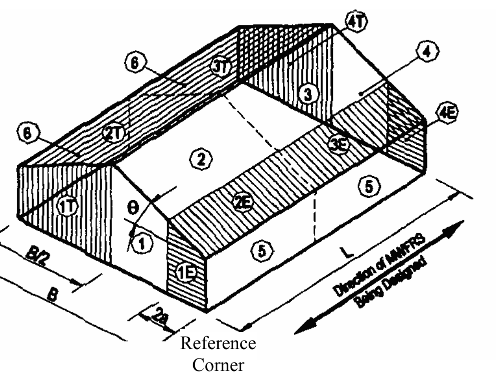
Roof Angle θ (Degrees)	Building Surface									
	1	2	3	4	5	6	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	-0.45	-0.45	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	-0.45	-0.45	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	-0.45	-0.45	0.69	0.69	-0.48	-0.48

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. For values of θ other than those shown, linear interpolation is permitted.
3. The building must be designed for all wind directions using the 8 loading patterns shown. The load patterns are applied to each building corner in turn as the Reference Corner.
4. Combinations of external and internal pressures (see Figure 7.2-1) shall be evaluated as required to obtain the most severe loadings.
5. For the torsional load cases shown below, the pressures in zones designated with a "T" (1T, 2T, 3T, 4T) shall be 25% of the full design wind pressures (zones 1, 2, 3, 4).
   Exception: One story buildings with h less than or equal to 9m, buildings two stories or less framed with light frame construction, and buildings two stories or less designed with flexible diaphragms need not be designed for the torsional load cases.
   Torsional loading shall apply to all eight basic load patterns using the figures below applied at each reference corner.
6. Except for moment-resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
7. For the design of the MWFRS providing lateral resistance in a direction parallel to a ridge line or for flat roofs, use θ = 0° and locate the zone 2/3 boundary at the mid-length of the building.
8. The roof pressure coefficient GC_{pf} when negative in Zone 2, shall be applied in Zone 2 for a distance from the edge of roof equal to 0.5 times the horizontal dimension of the building parallel to the direction of the MWFRS being designed or 2.5h, whichever is less; the remainder of Zone 2 extending shall use the pressure coefficient GC_{pf} for Zone 3.
9. Notation:
   a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
   h: Mean roof height, in meters, except that eave height shall be used for θ ≤ 10°.
   θ : Angle of plane of roof from horizontal, in degrees.



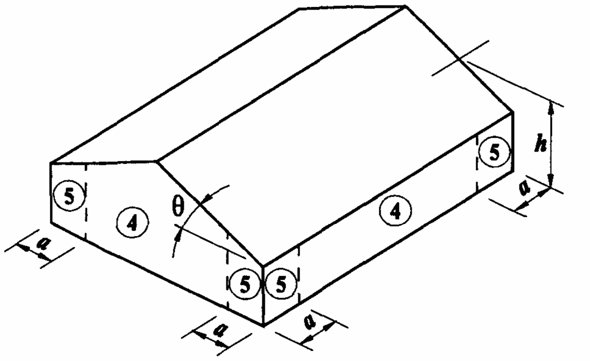
Transverse Direction

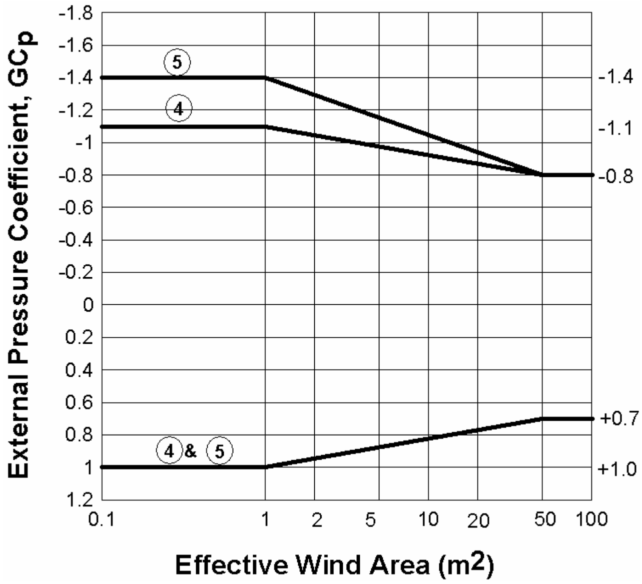


Longitudinal Direction

<u>Torsional Load Cases</u>

Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-7A	External Pressure Coefficients, GC_p	**Walls**
Enclosed, Partially Enclosed Buildings		

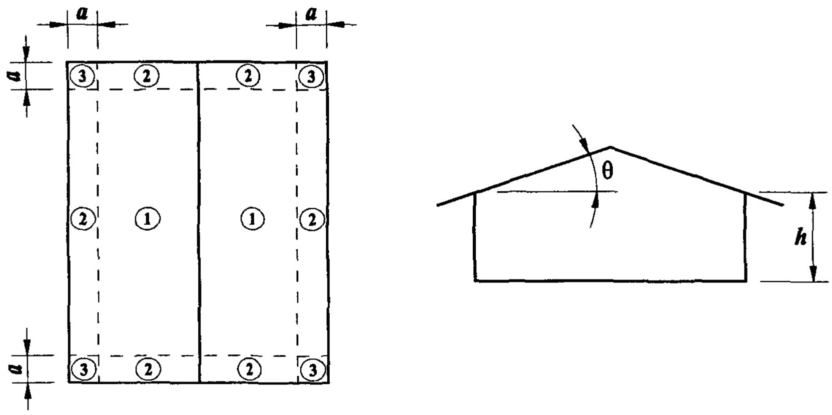


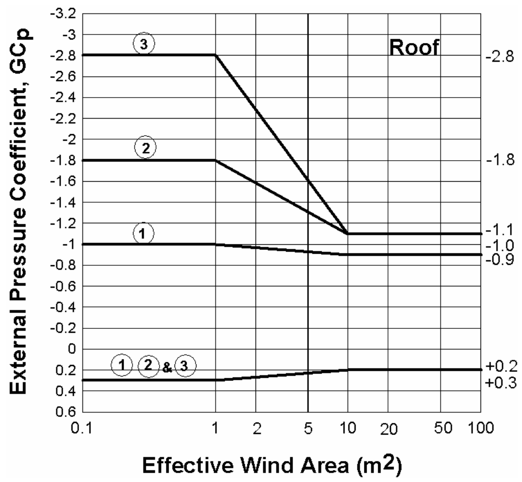


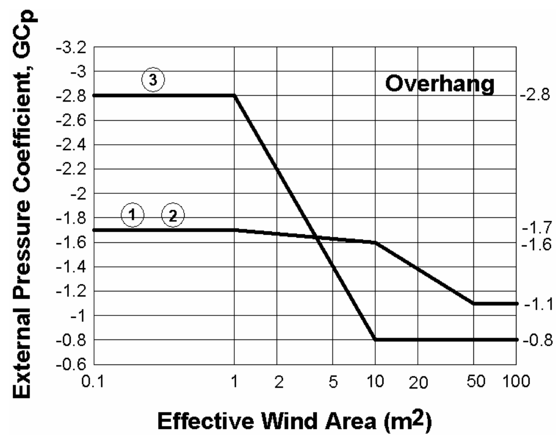
Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_P for walls shall be reduced by 10% when $\theta \le 10^0$.
6. Notation:
   a : 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
   h: Mean roof height, in metres, except that eave height shall be used for $\theta \le 10^0$.
   θ : Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-7B	External Pressure Coefficients, GC_p	**Gable Roof** $\theta \le 7^o$
Enclosed, Partially Enclosed Buildings		



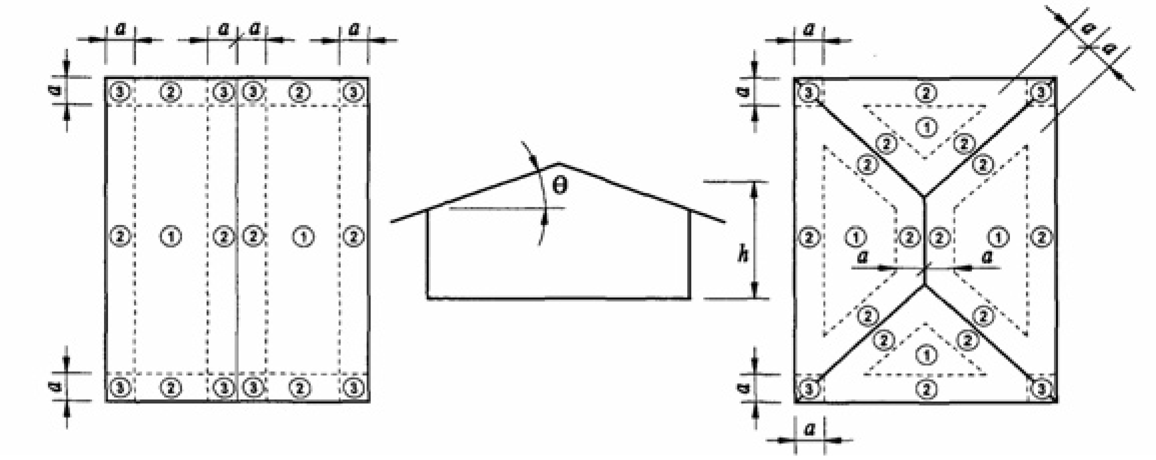


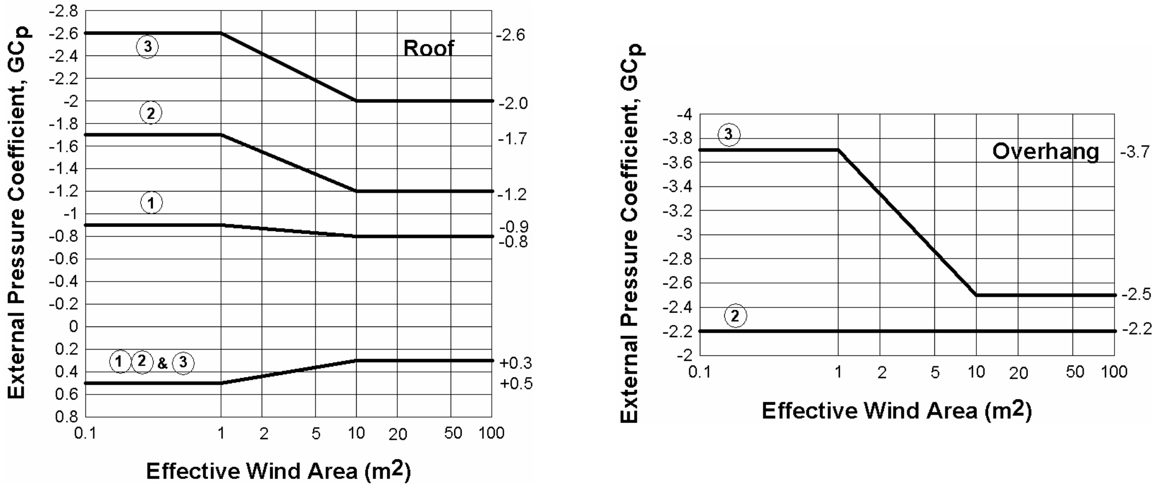


Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. If a parapet equal to or higher than 0.9m is provided around the perimeter of the roof with $\theta \le 7^0$, Zone 3 shall be treated as Zone 2.
6. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
7. Notation:
   - a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
   - h: Eave height shall be used for $\theta \le 10^0$
   - θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \leq 18m$
Figure 7.2-7C	**External Pressure Coefficients, GC_p**	**Gable/Hip Roof** $7^o < \theta \leq 27^o$
Enclosed, Partially Enclosed Buildings		

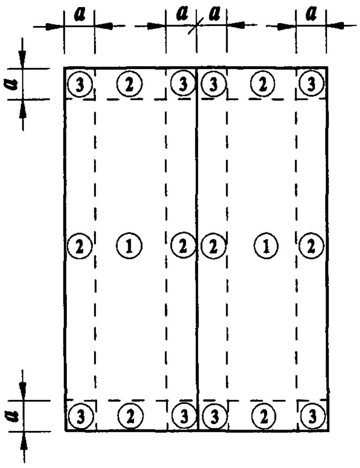


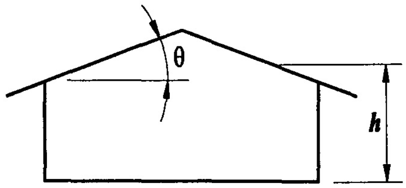


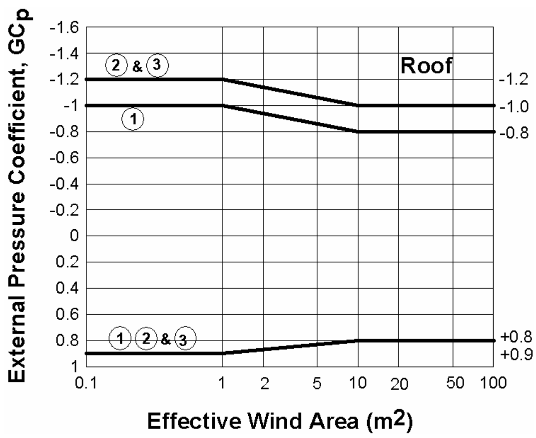
Notes:

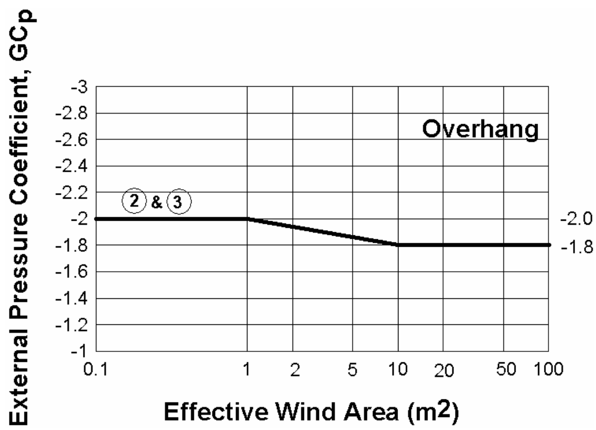
1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
6. For hip roofs with $7^o < \theta \leq 27^o$ edge/ridge strips and pressure coefficients for ridges of gabled roofs shall apply on each hip.
7. For hip roofs with $\theta \leq 25^o$ Zone 3 shall be treated as Zone 2.
8. Notation:
   a : 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
   h: Mean roof height, in metres, except that eave height shall be used for $\theta \leq 10^o$
   θ : Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \leq 18m$
Figure 7.2-7D	**External Pressure Coefficients, GC_p**	**Gable Roofs** $27^o < \theta \leq 45^o$
Enclosed, Partially Enclosed Buildings		









Notes:

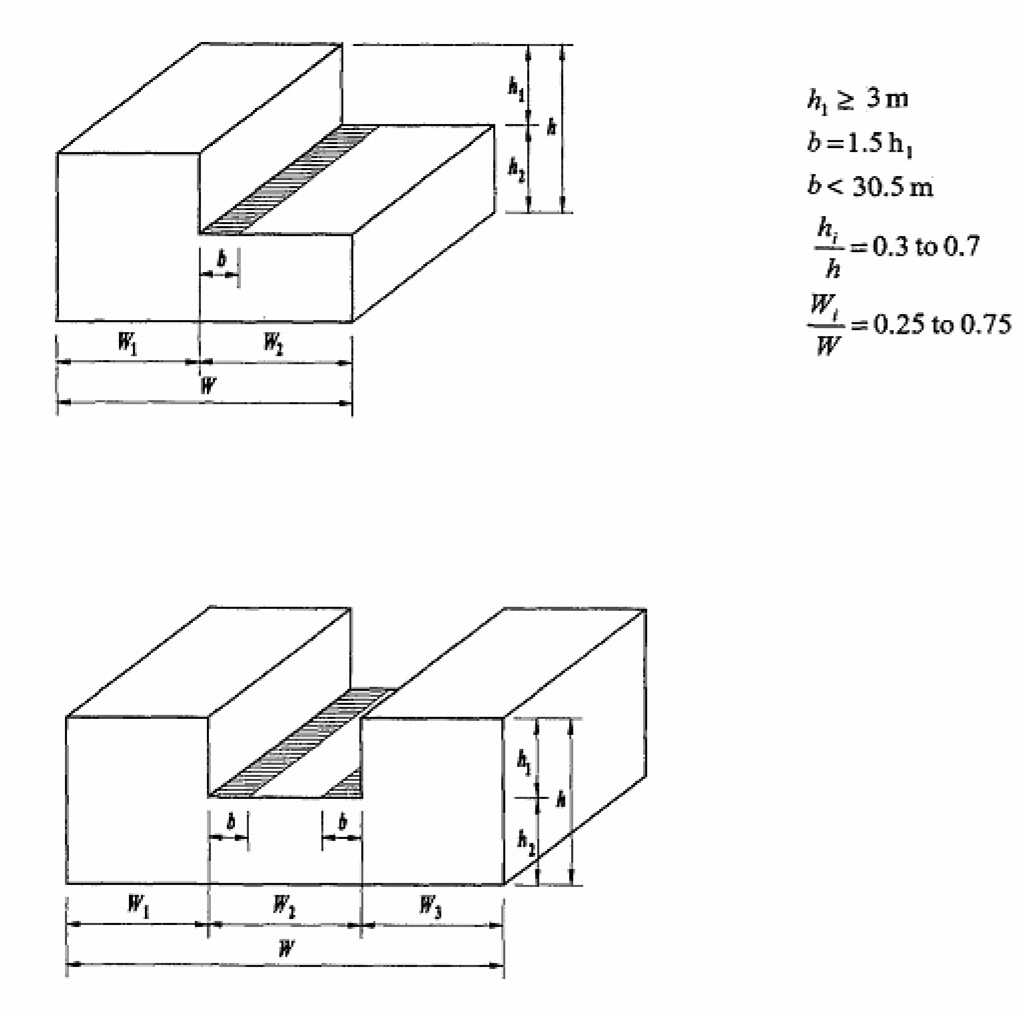
1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
6. Notation:

   a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

   h: Mean roof height, in metres.

   θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-8	**External Pressure Coefficients, GC_p**	**Stepped Roofs**
Enclosed, Partially Enclosed Buildings		



Notes:

1. On the lower level of flat, stepped roofs shown in Fig. 7.2-8, the zone designations and pressure coefficients shown in Fig. 7.2-7 B shall apply, except that at the roof-upper wall intersection(s), Zone *3* shall be treated as Zone 2 and Zone 2 shall be treated as Zone 1. Positive values of GC_p equal to those for walls in Fig. 7.2-7 A shall apply on the cross-hatched areas shown in Fig. 7.2-8.
2. Notation:

   b: $1.5h_1$ in Fig. 7.2-8, but not greater than *30.5 m.*

   h: Mean roof height, in metres.

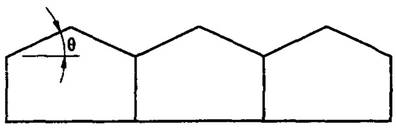
   h_i: h_1 or h_2 in Fig. 7.2-8; $h = h_1 + h_2$; $h_1 \ge 3.1$ m; $h_i/h = 0.3$ to 0.7.

   W: Building width in Fig. 7.2-8.

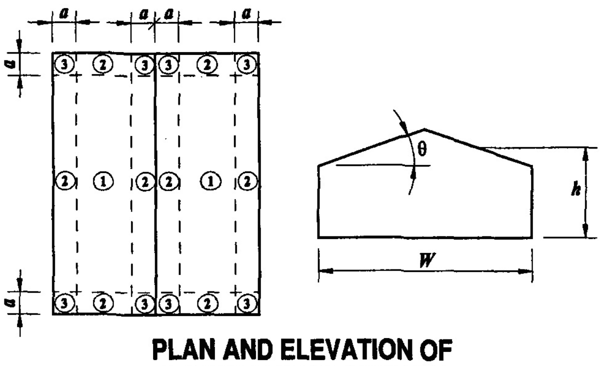
   W_i: W_1 or W_2 or W_3 in Fig. 7.2-8. $W = W_1 + W_2$ or $W_1 + W_2 + W_3$; $W_i/W = 0.25$ to 0.75.

   θ: Angle of plane of roof from horizontal, in degrees.

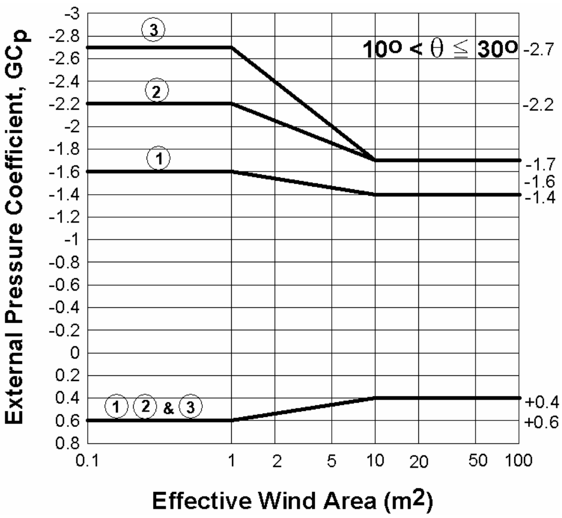
Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-9	External Pressure Coefficients, GC_p	**Multispan Gable Roofs**
Enclosed, Partially Enclosed Buildings		

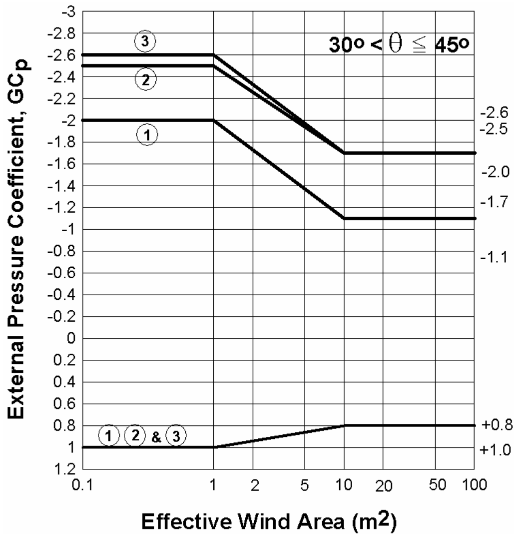


ELEVATION OF BUILDING
(2 or More Spans)



PLAN AND ELEVATION OF A SINGLE SPAN MODULE

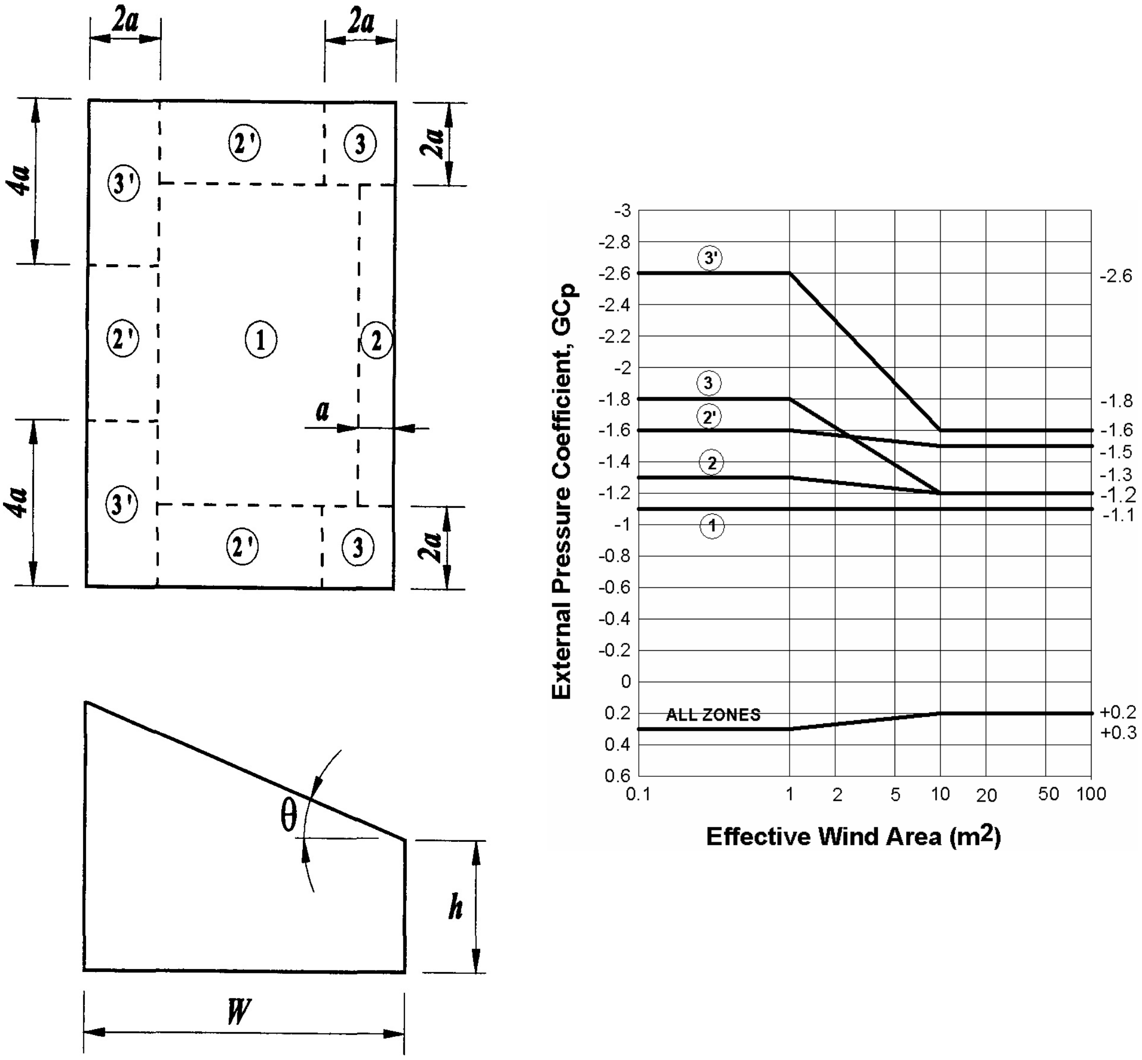




Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \le 10^o$, values of GC_P from Fig. 7.2-7 shall be used.
6. Notation:
   - a : 10 percent of least horizontal dimension of a single-span module or 0.4h, whichever is smaller, but not less than either 4 percent of least horizontal dimension of a single-span module or 0.9 m.
   - h: Mean roof height, in metres, except that eave height shall be used for $\theta \le 10^o$.
   - W: Building module width, in metres.
   - θ : Angle of plane of roof from horizontal, in degrees.

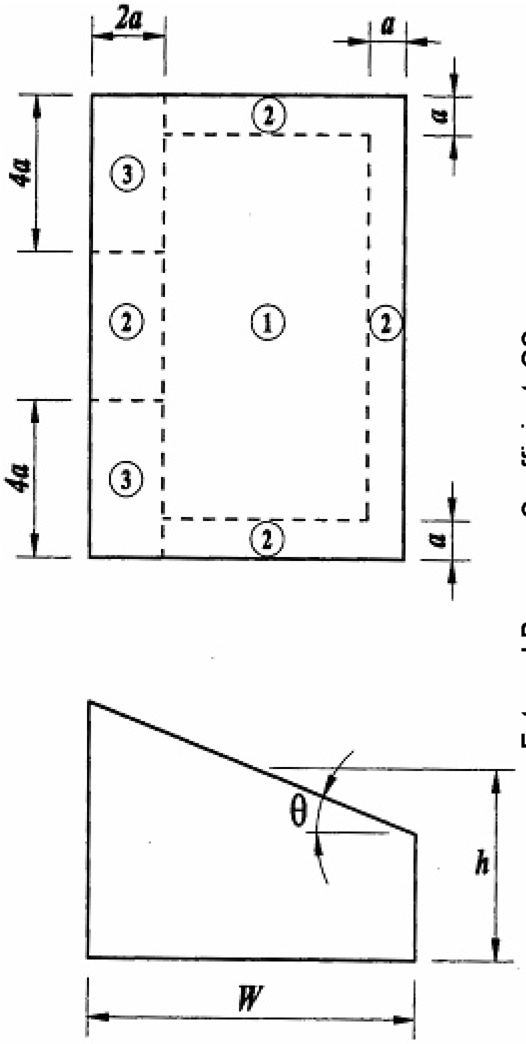
Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-10A	**External Pressure Coefficients, GC_p**	**Monoslope Roofs**
Enclosed, Partially Enclosed Buildings		$3^o < \theta \le 10^o$

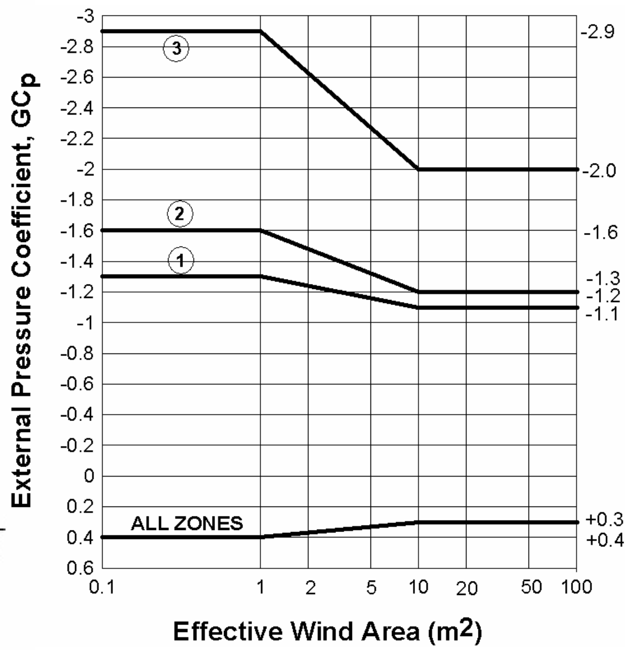


Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \le 3^o$, values of GC_P from Fig. 7.2-7B shall be used.
6. Notation:
   - a: 10 percent of least horizontal dimension or 0.4 h, whichever is smaller, but not less than either 4 percent of least horizontal dimension or 0.9 m.
   - h: Eave height shall be used for $\theta \le 10^o$.
   - W: Building width, in metres.
   - θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \leq 18m$
Figure 7.2-10B	**External Pressure Coefficients, GC_p**	**Monoslope Roofs**
Enclosed, Partially Enclosed Buildings		$10^o < \theta \leq 30^o$

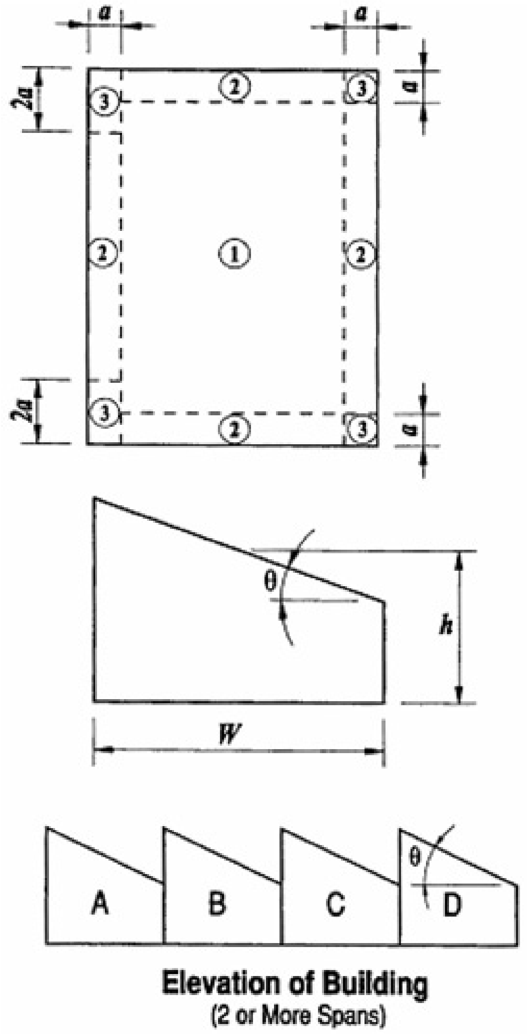


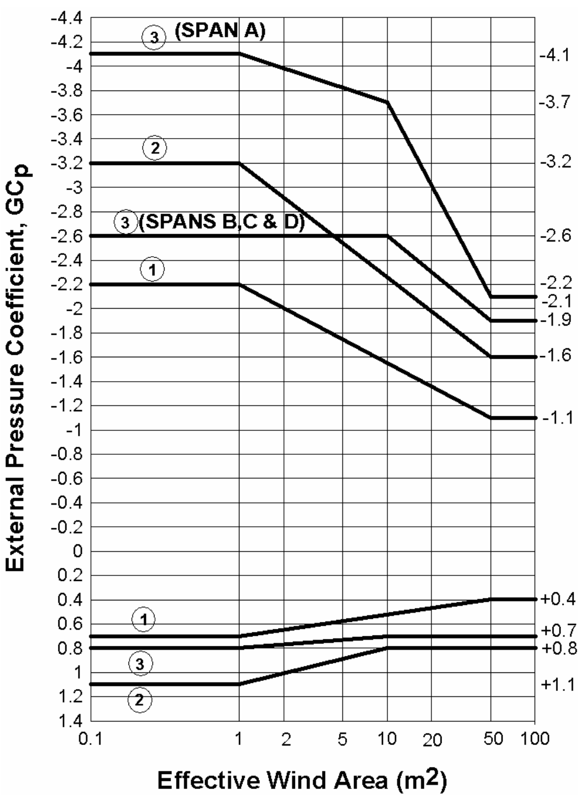


Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Notation:
   - a: 10 percent of least horizontal dimension or 0.4 h, whichever is smaller, but not less than either 4 percent of least horizontal dimension or 0.9 m.
   - h: Mean roof height, in metres.
   - W: Building width, in metres.
   - θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		$h \le 18m$
Figure 7.2-11	External Pressure Coefficients, GC_p	**Sawtooth Roofs**
Enclosed, Partially Enclosed Buildings		

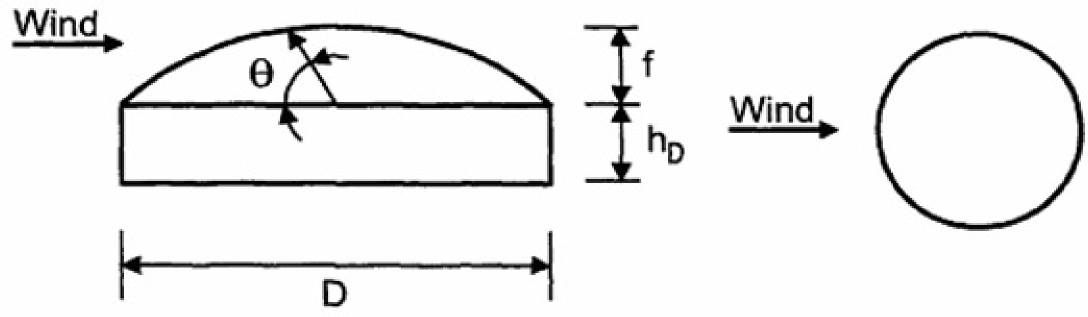




Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \le 10^o$ values of GC_P from Fig. 7.2-7 shall be used.
6. Notation:
   - a: 10 percent of least horizontal dimension or 0.4 h, whichever is smaller, but not less than either 4 percent of least horizontal dimension or 0.9 m.
   - h: Mean roof height, in metres, except that eave height shall be used for $\theta \le 10^o$
   - W: Building width, in metres.
   - θ: Angle of plane of roof from horizontal, in degrees.

Components and Cladding - Method 2		All Heights
Figure 7.2-12	**External Pressure Coefficients, GC_p**	**Domed Roofs**
Enclosed, Partially Enclosed Buildings and Structures		

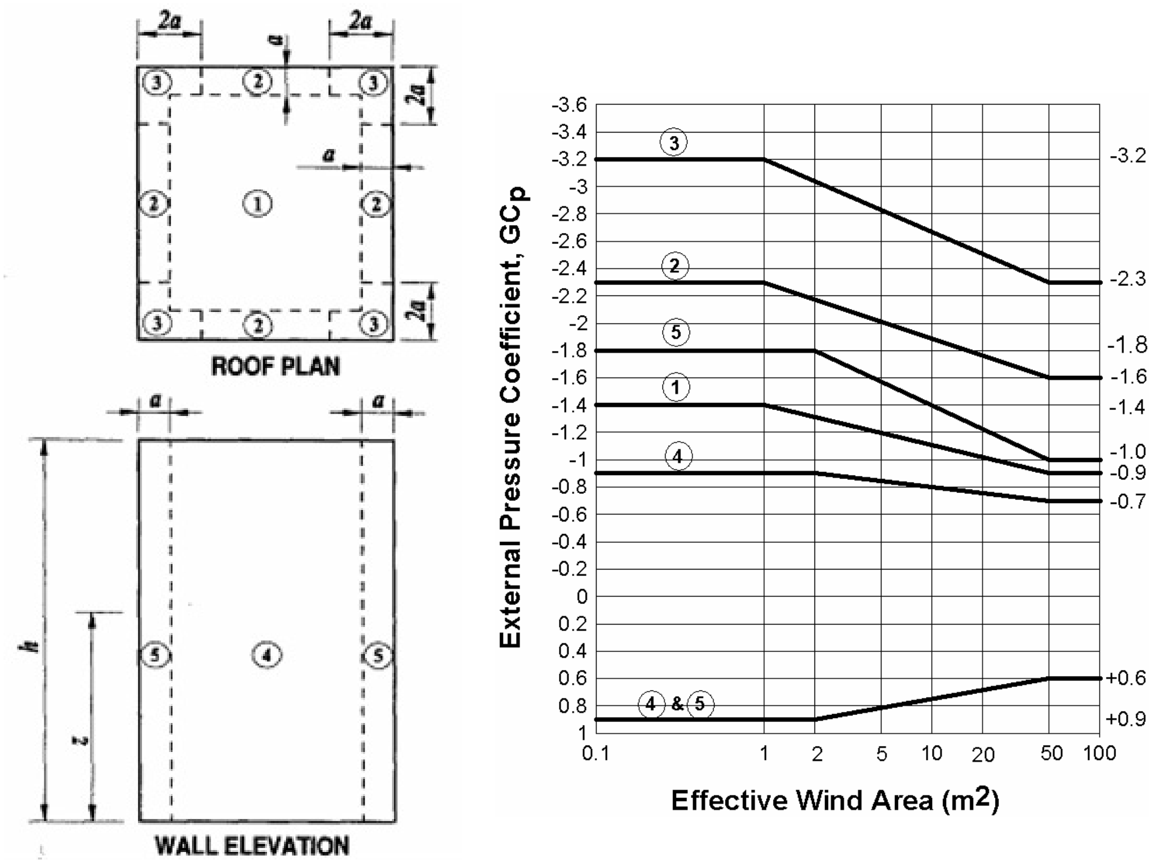


External Pressure Coefficients for Domes with a Circular Base			
	Negative Pressures	**Positive Pressures**	**Positive Pressures**
θ, degrees	**0 – 90**	**0 – 60**	**61 – 90**
GC_P	**-0.9**	**+0.9**	**+0.5**

Notes:

1. Values denote GC_P to be used with q_{h_D+f} where h_D + f is the height at the top of the dome.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. Each component shall be designed for the maximum positive and negative pressures.
4. Values apply to $\theta \leq h_D/D \leq 0.5, 0.2 \leq f/D \leq 0.5$.
5. θ = 0 degrees on dome springline, θ = 90 degrees at dome center top point. f is measured from springline to top.

Components and Cladding - Method 2		$h \leq 18m$
Figure 7.2-13	**External Pressure Coefficients, GC_p**	**Wall & Roofs**
Enclosed, Partially Enclosed Buildings		



Notes:

1. Vertical scale denotes GC_P to be used with appropriate q_z or q_h.
2. Horizontal scale denotes effective wind area A, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Use q_z with positive values of GC_P and q_h with negative values of GC_P.
5. Each component shall be designed for maximum positive and negative pressures.
6. Coefficients are for roofs with angle $\theta \leq 10^o$. For other roof angles and geometry, use GC_P values from Fig. 7.2-7 and attendant q_h based on exposure defined in 6.4.2.
7. If a parapet equal to or higher than 0.9 m is provided around the perimeter of the roof with $\theta \leq 10^o$ Zone 3 shall be treated as Zone 2.
8. Notation:
   - a: 10 percent of least horizontal dimension, but not less than 0.9 m.
   - h: Mean roof height, in metres, except that eave height shall be used for $\theta \leq 10^o$.
   - z: height above ground, in metres.
   - θ: Angle of plane of roof from horizontal, in degrees.

Main Wind Force Resisting System - Method 2		All Heights
Figure 7.2-14	**Force Coefficients, C_f**	**Monoslope Roofs**
Open Buildings		

Roof angle θ degrees	L/B						
	5	**3**	**2**	**1**	**1/2**	**1/3**	**1/5**
10	0.2	0.25	0.3	0.45	0.55	0.7	0.75
15	0.35	0.45	0.5	0.7	0.85	0.9	0.85
20	0.5	0.6	0.75	0.9	1.0	0.95	0.9
25	0.7	0.8	0.95	1.15	1.1	1.05	0.95
30	0.9	1.0	1.2	1.3	1.2	1.1	1.0

Roof angle θ degrees	Center of Pressure X/L		
	L/B		
	2 to 5	**1**	**1/5 to 1/2**
10 to 20	0.35	0.3	0.3
25	0.35	0.35	0.4
30	0.35	0.4	0.45

Notes:

1. Wind forces act normal to the surface. Two cases shall be considered: (1) wind forces directed inward; and (2) wind forces directed outward.
2. The roof angle shall be assumed to vary ± 10^0 from the actual angle and the angle resulting in the greatest force coefficient shall be used.
3. Notation:
   B: Dimension of roof measured normal to wind direction, in metres;
   L: Dimension of roof measured parallel to wind direction, in metres;
   X: Distance to center of pressure from windward edge of roof, in metres; and
   θ : Angle of plane of roof from horizontal, in degrees.

Other structures - Method 2		All Heights
Figure 7.2-15	**Force Coefficients, C_f**	**Chimneys, Tanks, Rooftop, Equipment and Similar Structures**

Cross-Section	Type of Surface	h/D		
		1	**7**	**25**
Square (wind normal to face)	All	1.3	1.4	2.0
Square (wind along diagonal)	All	1.0	1.1	1.5
Hexagonal or octagonal	All	1.0	1.2	1.4
Round ($D\sqrt{q_z} > 0.17$, D in m, q_z in kN/m^2)	Moderately smooth	0.5	0.6	0.7
	Rough (D' /D = 0.02)	0.7	0.8	0.9
	Very rough (D'/D = 0.08)	0.8	1.0	1.2
Round ($D\sqrt{q_z} \leq 0.17$, D in m, q_z in kN/m^2)	All	0.7	0.8	1.2

Notes:

1. Design wind force shall be calculated based on the area of the structure projected on a plane normal to the wind direction. The force shall be assumed to act parallel to the wind direction.
2. Linear interpolation is permitted for *h/D* values other than shown.
3. Notation:

   D: diameter of circular cross-section and least horizontal dimension of square, hexagonal or octagonal cross-sections at elevation under consideration, in metres;

   D': depth of protruding elements such as ribs and spoilers, in metres; and

   h: height of structure, in metres; and

   q_z: velocity pressure evaluated at height z above ground, in kN/m^2.

Other structures - Method 2		All Heights
Figure 7.2-16	Force Coefficients, C_f	Solid Freestanding Walls and Solid Signs

At Ground Level		Above Ground Level	
ν	**C_f**	**M/N**	**C_f**
$\leq$ 3	1.2	$\leq$ 6	1.2
5	1.3	10	1.3
8	1.4	16	1.4
10	1.5	20	1.5
20	1.75	40	1.75
30	1.85	60	1.85
$\geq$ 40	2.0	$\geq$ 80	2.0

Notes:

1. The term “signs” in notes below applies also to “freestanding walls”.
2. Signs with openings comprising less than 30% of the gross area are classified as solid signs.
3. Signs for which the distance from the ground to the bottom edge is less than 0.25 times the vertical dimension shall be considered to be at ground level.
4. To allow for both normal and oblique wind directions, two cases shall be considered

    a. resultant force acts normal to the face of the sign on a vertical line passing through the geometric center, and
    b. resultant force acts normal to the face of the sign at a distance from a vertical line passing through the geometric center equal to 0.2 times the average width of the sign.

5. Notation:

    ν : ratio of height to width
    M: larger dimension of sign, in metres; and
    N: smaller dimension of sign, in metres.

Other structures - Method 2		All Heights
Figure 7.2-17	Force Coefficients, C_f	Open Signs and Lattice Frameworks

ε	Flat-Sided Members	Rounded Members	
		$D\sqrt{q_z} \leq 0.17$	$D\sqrt{q_z} > 0.17$
< 0.1	2.0	1.2	0.8
0.1 to 0.29	1.8	1.3	0.9
0.3 to 0.7	1.6	1.5	1.1

Notes:

1. Signs with openings comprising 30% or more of the gross area are classified as open signs.
2. The calculation of the design wind forces shall be based on the area of all exposed members and elements projected on a plane normal to the wind direction. Forces shall be assumed to act parallel to the wind direction.
3. The area A_f consistent with these force coefficients is the solid area projected normal to the wind direction.
4. Notation:

   ε : ratio of solid area to gross area;

   D: diameter of a typical round member, in metres;

   q_z: velocity pressure evaluated at height z above ground in kN/m^2.

Other structures - Method 2		All Heights
Figure 7.2-18	Force Coefficients, C_f	Trussed Towers
Open Structures		

Tower Cross Section	C_f
Square	$4.0\,\varepsilon^2 - 5.9\ \varepsilon + 4.0$
Triangle	$3.4\,\varepsilon^2 - 4.7\ \varepsilon + 3.4$

Notes:

1. For all wind directions considered, the area A_f consistent with the specified force coefficients shall be the solid area of a tower face projected on the plane of that face for the tower segment under consideration.
2. The specified force coefficients are for towers with structural angles or similar flat sided members.
3. For towers containing rounded members, it is acceptable to multiply the specified force coefficients by the following factor when determining wind forces on such members:

   $0.51\ \varepsilon^2 + 0.57$, but not > 1.0
4. Wind forces shall be applied in the directions resulting in maximum member forces and reactions. For towers with square cross-sections, wind forces shall be multiplied by the following factor when the wind is directed along a tower diagonal:

   $1 + 0.75\ \varepsilon$, but not > 1.2
5. Wind forces on tower appurtenances such as ladders, conduits, lights, elevators, etc., shall be calculated using appropriate force coefficients for these elements.
6. Notation:

   ε : ratio of solid area to gross area of one tower face for the segment under consideration.

Terrain Exposure Constants
Table 7.2-1

Exposure	α	Z_g(m)	$\hat{a}$	$\hat{b}$	$\bar{a}$	$\bar{b}$	c	ℓ(m)	$\bar{\varepsilon}$	Z_{min}(m)*
B	7.0	365	1/7	0.84	1/4.0	0.45	0.30	100	1/3.0	10
C	9.5	275	1/9.5	1.00	1/6.5	0.65	0.20	150	1/5.0	5
D	11.5	215	1/11.5	1.07	1/9.0	0.80	0.15	200	1/8.0	2.5

*z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is greater of 0.6h or z_{min}.

For buildings with $h \le z_{min}$, $\bar{z}$ shall be taken as z_{min}.

Velocity Pressure Exposure Coefficients, K_h and K_z

Table 7.2-2

Height above Ground Level, z (m)	Exposure (Note 1)			
	B		C	D
	Case 1	Case 2	Cases 1 & 2	Cases 1 & 2
0-5	0.72	0.59	0.86	1.04
6	0.72	0.62	0.90	1.08
8	0.72	0.67	0.95	1.13
10	0.72	0.72	1.00	1.18
12	0.76	0.76	1.04	1.22
14	0.79	0.79	1.07	1.25
16	0.82	0.82	1.10	1.28
18	0.85	0.85	1.13	1.31
20	0.88	0.88	1.16	1.33
22	0.90	0.90	1.18	1.35
25	0.93	0.93	1.21	1.38
30	0.98	0.98	1.26	1.43
35	1.03	1.03	1.30	1.47
40	1.07	1.07	1.34	1.50
50	1.14	1.14	1.40	1.56
60	1.20	1.20	1.46	1.61
75	1.28	1.28	1.53	1.67
90	1.35	1.35	1.59	1.73
105	1.41	1.41	1.64	1.77
120	1.46	1.46	1.69	1.82
135	1.51	1.51	1.73	1.85
150	1.56	1.56	1.77	1.89

Notes:

1. **Case 1**: a. All components and cladding.
   b. Main wind force resisting system in low-rise buildings designed using Figure 7.2-6.

   Case 2: a. All main wind force resisting systems in buildings except those in low-rise buildings designed using Figure 7.2-6.
   b. All main wind force resisting systems in other structures.
2. The velocity pressure exposure coefficient K_z may be determined from the following formula:

   For $5\text{m} \leq z \leq z_g$: $K_z = 2.01\ (z/z_g)^{2/\alpha}$

   For $z < 5\text{m}$: $K_z = 2.01\ (5/z_g)^{2/\alpha}$

   Note: z shall not be taken less than 10 m for Case 1 in exposure B.

3. α and z_g are tabulated in Table 7.2-1.

4. Linear interpolation for intermediate values of height z is acceptable.

5. Exposure categories are defined in 6.4.2.

CHAPTER 8
RAIN LOADS

SECTION 8.1
SYMBOLS AND NOTATIONS

A = roof area serviced by a single drainage system, in m^2.
Q = flow rate out of a single drainage system, in m^3/s.
R = rain load on the undeflected roof, in kN/m^2. When the phrase "undeflected roof" is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.
I_n = design rainfall intensity, 180 mm/h for all regions of Saudi Arabia, for fifty years recurrence interval and for storm duration up to two hours.
d_s = depth of water on the undeflected roof up to the inlet of the secondary drainage system when the primary drainage system is blocked (i.e., the static head), in mm.
d_h = additional depth of water on the undeflected roof above the inlet of the secondary drainage system at its design flow (i.e., the hydraulic head), in mm.

SECTION 8.2
ROOF DRAINAGE

Roof drainage systems shall be designed in accordance with the provisions of the SBC. The flow capacity of secondary (overflow) drains or scuppers shall not be less than that of the primary drains or scuppers.

SECTION 8.3
DESIGN RAIN LOADS

Each portion of a roof shall be designed to sustain the load of all rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow.

$$R = 0.0098\ (d_s + d_h) \qquad \textbf{(Eq. 8-1)}$$

$d_s + d_h \geq 200$ mm

If the secondary drainage systems contain drain lines, such lines and their point of discharge shall be separate from the primary drain lines.

The depth of water, d_h, above the inlet of the secondary drainage system (i.e., the hydraulic head) is a function of the rainfall intensity at the site, the area of roof serviced by that drainage system, and the size of the drainage system.
The flow rate through a single drainage system is as follows:

$$Q = 0.278 \times 10^{-6}\ A\ I_n \qquad \textbf{(Eq. 8-2)}$$

The hydraulic head, d_h, is related to flow rate, Q, for various drainage systems in Table 8-1. The hydraulic head, d_h, is zero when the secondary drainage system is simply overflow all along a roof edge.

SECTION 8.4
PONDING INSTABILITY

"Ponding" refers to the retention of water due solely to the deflection of relatively flat roofs. Roofs with a slope less than 1.2^o (degrees) shall be investigated by structural analysis to ensure that they possess adequate stiffness to preclude progressive deflection (i.e., instability) as rain falls on them. The primary drainage system within an area subjected to ponding shall be considered to be blocked in this analysis.

SECTION 8.5
CONTROLLED DRAINAGE

Roofs equipped with hardware to control the rate of drainage shall be equipped with a secondary drainage system at a higher elevation that limits accumulation of water on the roof above that elevation. Such roofs shall be designed to sustain the load of all rainwater that will accumulate on them to the elevation of the secondary drainage system, plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow (determined from Section 8.3).

Such roofs shall also be checked for ponding instability (determined from Section 8.4). Table 8-1 shows flow rate in cubic meters per second of various drainage systems at various hydraulic heads.

Table 8-1:
Flow rate, Q (m^3/sec) of various drainage systems at various hydraulic heads, d_h in millimetres.

	HYDRAULIC HEAD d_h mm						
Drainage System	**25**	**50**	**75**	**100**	**125**	**175**	**200**
100 mm diameter drain	.0051	.0107					
150 mm diameter drain	.0063	.0120	.0240				
200 mm diameter drain	.0079	.0145	.0353	.0694			
150 mm wide, channel scupper*	.0011	.0032	.0057	.0088	.0122	.0202	.0248
600 mm wide, channel scupper	.0045	.0126	.0227	.0353	.0490	.0810	.0992
150 mm wide, 100 mm high, closed scupper*	.0011	.0032	.0057	.0088	.0112	.0146	.0160
600 mm wide, 100 mm high, closed scupper	.0045	.0126	.0227	.0353	.0447	.0583	.0638
150 mm wide, 150 mm high, closed scupper	.0011	.0032	.0057	.0088	.0122	.0191	.0216
600 mm wide, 150 mm high, closed scupper	.0045	.0126	.0227	.0353	.0490	.0765	.0866

* Channel scuppers are open-topped (i.e., 3-sided). Closed scuppers are 4-sided.

CHAPTER 9
SEISMIC DESIGN CRITERIA

SECTION 9.1
GENERAL

9.1.1 **Purpose.** Chapters 9 through 16 present criteria for the design and construction of buildings and similar structures subject to earthquake ground motions. The specified earthquake loads are based on post-elastic energy dissipation in the structure, and because of this fact, the provisions for design, detailing, and construction shall be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake.

9.1.2 **Scope and Applicability.**

9.1.2.2 **Scope.** Every building, and portion thereof, shall be designed and constructed to resist the effects of earthquake motions as prescribed by these provisions. Certain nonbuilding structures, as described in Chapter 13, are within the scope and shall be designed and constructed as required for buildings. Additions to existing structures also shall be designed and constructed to resist the effects of earthquake motions as prescribed by these provisions. Existing structures and alterations to existing structures need only comply with these provisions when required by Chapter 16.

9.1.2.2 **Applicability.** Structures shall be designed and constructed in accordance with the requirements of the following chapters based on the type of structure:

a. Buildings – Chapter 10

b. Nonbuilding Structures – Chapter 13

SECTION 9.2
DEFINITIONS

The definitions presented in this Section provide the meaning of the terms used in Chapter 9 through 16 of these provisions.

Addition. An increase in building area, aggregate floor area, height, or number of stories of a structure.

Alteration. Any construction or renovation to an existing structure other than an addition.

Appendage. An architectural component such as a canopy; marquee, ornamental balcony, or statuary.

Approval. The written acceptance by the authority having jurisdiction of documentation that establishes the qualification of a material, system, component, procedure, or person to fulfill the requirements of these provisions for the intended use.

Architectural Component Support. Those structural members or assemblies of members, including braces, frames, struts, and attachments that transmit all loads and forces between architectural systems, components, or elements and the structure.

Attachments. Means by which components and their supports are secured or connected to the seismic force-resisting system of the structure. Such attachments include anchor bolts, welded connections, and mechanical fasteners.

Base. The level at which the horizontal seismic ground motions are considered to be imparted to the structure.

Base Shear. Total design lateral force or shear at the base.

Basement. A basement is any story below the lowest story above grade.

Boundary Elements. Diaphragm and shear wall boundary members to which the diaphragm transfers forces. Boundary members include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities, and re-entrant corners.

Boundary Members. Portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary members include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities, and re-entrant corners.

Building. Any structure whose use could include shelter of human occupants.

Cantilevered Column System. A seismic force resisting system in which lateral forces are resisted entirely by columns acting as cantilevers from the foundation.

Component. A part or element of an architectural, electrical, mechanical, or structural system.

> **Component, Equipment.** A mechanical or electrical component or element that is part of a mechanical and/or electrical system within or without a building system.
>
> **Component, flexible.** Component, including its attachments, having a fundamental period greater than 0.06 sec.
>
> **Component, rigid.** Component, including its attachments, having a fundamental period less than or equal to 0.06 sec.

Concrete, Plain. Concrete that is either unreinforced or contains less reinforcement than the minimum amount specified in SBC 304 for reinforced concrete.

Concrete, Reinforced. Concrete reinforced with no less than the minimum amount required by SBC 304, prestressed or nonprestressed, and designed on the assumption that the two materials act together in resisting forces.

Confined Region. That portion of a reinforced concrete or reinforced masonry component in which the concrete or masonry is confined by closely spaced special transverse reinforcement restraining the concrete or masonry in directions perpendicular to the applied stress.

Construction Documents. The written, graphic, electronic, and pictorial documents describing the design, locations, and physical characteristics of the project required to verify compliance with this Code.

Container. A large-scale independent component used as a receptacle or vessel to accommodate plants, refuse, or similar uses, not including liquids.

Deformability. The ratio of the ultimate deformation to the limit deformation.

High deformability element. An element whose deformability is not less than 3.5 when subjected to four fully reversed cycles at the limit deformation.

Limited deformability element. An element that is neither a low deformability nor a high deformability element.

Low deformability element. An element whose deformability is 1.5 or less.

Deformation.

Limit deformation. Two times the initial deformation that occurs at a load equal to 40% of the maximum strength.

Ultimate deformation. The deformation at which failure occurs and which shall be deemed to occur if the sustainable load reduces to 80% or less of the maximum strength.

Design Earthquake. The earthquake effects that are two-thirds of the corresponding maximum considered earthquake.

Designated Seismic Systems. The seismic force-resisting system and those architectural, electrical, and mechanical systems or their components that require design in accordance with Section 12.1 and for which the component importance factor, I_p, is 1.0.

Diaphragm. Roof, floor, or other membrane or bracing system acting to transfer the lateral forces to the vertical resisting elements. Diaphragms are classified as either flexible or rigid according to the requirements of Section 10.3.1.

Diaphragm Boundary. A location where shear is transferred into or out of the diaphragm element. Transfer is either to a boundary element or to another force-resisting element.

Diaphragm Chord. A diaphragm boundary element perpendicular to the applied load that is assumed to take axial stresses due to the diaphragm moment in a manner analogous to the flanges of a beam. Also applies to shear walls.

Displacement.

Design displacement. The design earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion, required for design of the isolation system.

Total design displacement. The design earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the isolation system or an element thereof.

Total maximum displacement. The maximum considered earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the isolation system or elements thereof, design of structure separations, and vertical load testing of isolator unit prototypes.

Displacement Restraint System. A collection of structural elements that limits lateral displacement of seismically isolated structures due to the maximum considered earthquake.

Enclosure. An interior space surrounded by walls.

Equipment Support. Those structural members or assemblies of members or manufactured elements, including braces, frames, legs, lugs, snuggers, hangers, or saddles, that transmit gravity loads and operating loads between the equipment and the structure.

Essential Facility. A structure required for post-earthquake recovery.

FRAME.

Braced frame. An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a bearing wall, building frame, or dual system to resist seismic forces.

Concentrically braced frame (CBF). A braced frame in which the members are subjected primarily to axial forces.

Eccentrically braced frame (EBF). A diagonally braced frame in which at least one end of each brace frames into a beam a short distance from a beam-column joint or from another diagonal brace.

Ordinary concentrically braced frame (OCBF). A steel concentrically braced frame in which members and connections are designed in accordance with the provisions of SBC 306 without modification.

Special concentrically braced frame (SCBF). A steel or composite steel and concrete concentrically braced frame in which members and connections are designed for ductile behavior. Special concentrically braced frames shall conform to Section 15.4.

Moment frame.

Intermediate moment frame (IMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Intermediate moment frames of reinforced concrete shall conform to SBC 304. Intermediate moment frames of structural steel construction shall conform to Ref. 11.1-1. Intermediate moment frames of composite construction shall conform to Ref. 11.3-1, Part II, Section 6.4b, 7, 8, and 10.

Ordinary moment frame (OMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Ordinary moment frames shall conform to SBC 304.

Special moment frame (SMF). A moment frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Special moment frames shall conform to SBC 304 for concrete or Ref. 11.1-1 for steel.

FRAME SYSTEM.

Building frame system. A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.

Dual frame system. A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment-resisting frames and shear walls or braced frames as prescribed in Section 10.2.1.

Space frame system. A structural system composed of interconnected members, other than bearing walls, that is capable of supporting vertical loads and, when designed for such an application, is capable of providing resistance to seismic forces.

Glazed Curtain Wall. A nonbearing wall that extends beyond the edges of building floor slabs and includes a glazing material installed in the curtain wall framing.

Glazed Storefront. A nonbearing wall that is installed between floor slabs, typically including entrances, and includes a glazing material installed in the store-front framing.

Grade Plane. A reference plane representing the average of finished ground level adjoining the structure at all exterior walls. Where the finished ground level slopes away from the exterior walls, the reference plane shall be established by the lowest points within the area between the buildings and the lot line or, where the lot line is more than (2.0 m) from the structure, between the structure and a point (2.0 m) from the structure.

Hazardous Contents. A material that is highly toxic or potentially explosive and in sufficient quantity to pose a significant life safety threat to the general public if an uncontrolled release were to occur.

High Temperature Energy Source. A fluid, gas, or vapor whose temperature exceeds 105 °C.

Inspection, Special. The observation of the work by the special inspector to determine compliance with the approved construction documents and these standards.

Continuous special inspection. The full-time observation of the work by an approved special inspector who is present in the area where work is being performed.

Periodic special inspection. The part-time or intermittent observation of the work by an approved special inspector who is present in the area where work has been or is being performed.

Inspector, Special (who shall be identified as the owner's inspector).

A person approved by the authority having jurisdiction to perform special inspection. The authority having jurisdiction shall have the option to approve the quality assurance personnel of a fabricator as a special inspector.

Inverted Pendulum-Type Structures.

Structures that have a large portion of their mass concentrated near the top and, thus, have essentially one degree of freedom in horizontal translation. The structures are usually T-shaped with a single column supporting the beams or framing at the top.

Joint. The geometric volume common to intersecting members.

LOAD.

Dead load. The gravity load due to the weight of all permanent structural and nonstructural components of a building such as walls, floors, roofs, and the operating weight of fixed service equipment.

Gravity load (W). The total dead load and applicable portions of other loads as defined in Section 10.7.

Live load. The load superimposed by the use and occupancy of the building not including the wind load, earthquake load, or dead load, see Section 10.7.

Maximum Considered Earthquake Ground Motion. The most severe earthquake effects considered by these standards as defined in Section 9.4.3.

Nonbuilding Structure. A structure, other than a building, constructed of a type included in Chapter 13 and within the limits of Section 13.1.1.

Occupancy Importance Factor. A factor assigned to each structure according to its Occupancy Category as prescribed in Section 9.5.

Owner. Any person, agent, firm, or corporation having a legal or equitable interest in the property.

Partition. A nonstructural interior wall that spans horizontally or vertically from support to support. The supports may be the basic building frame, subsidiary structural members, or other portions of the partition system.

P-Delta Effect. The secondary effect on shears and moments of structural members due to the action of the vertical loads induced by displacement of the structure resulting from various loading conditions.

Quality Assurance Plan. A detailed written procedure that establishes the systems and components subject to special inspection and testing. The type and frequency of testing and the extent and duration of special inspection are given in the quality assurance plan.

Registered Design Professional. An engineer, registered or licensed to practice professional engineering.

Roofing Unit. A unit of roofing tile or similar material weighing more than 5 N.

Seismic Design Category. A classification assigned to a structure based on its Occupancy Category and the severity of the design earthquake ground motion at the site as defined in Section 9.6.

Seismic Force-Resisting System. That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed herein.

Seismic Forces. The assumed forces prescribed herein, related to the response of the structure to earthquake motions, to be used in the design of the structure and its components.

Seismic Response Coefficient. Coefficient C_s as determined from Section 10.9.2.1.

Shallow Anchor. Anchors with embedment length-to-diameter ratios of less than 8.

Shear Panel. A floor, roof, or wall component sheathed to act as a shear wall or diaphragm.

Site Class. A classification assigned to a site based on the types of soils present and their engineering properties as defined in Section 9.4.2.

Site Coefficients. The values of F_a and F_v as indicated in Tables 9.4.3a and 9.4.3b, respectively.

Special Lateral Reinforcement. Reinforcement composed of spirals, closed stirrups, or hoops and supplementary crossties provided to restrain the concrete and qualify the portion of the component, where used, as a confined region.

Storage Racks. Include industrial pallet racks, moveable shelf racks, and stacker racks made of cold-formed or hot-rolled structural members. Does not include other types of racks such as drive-in and drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel.

Story. The portion of a structure between the top of two successive, finished floor surfaces and, for the topmost story, from the top of the floor finish to the top of the roof structural element.

Story Above Grade. Any story having its finished floor surface entirely above grade, except that a story shall be considered as a story above grade where the finished floor surface of the story immediately above is more than (2.0 m) above the grade plane, more than (2.0 m) above the finished ground level for more than 40% of the total structure perimeter, or more than (4.0 m) above the finished ground level at any point.

Story Drift. The difference of horizontal deflections at the top and bottom of the story as determined in Section 10.9.7.1.

Story Drift Ratio. The story drift, as determined in Section 10.9.7.1, divided by the story height.

Story Shear. The summation of design lateral seismic forces at levels above the story under consideration.

STRENGTH.

Design strength. Nominal strength multiplied by a strength reduction factor, ϕ.

Nominal strength. Strength of a member or cross-section calculated in accordance with the requirements and assumptions of the strength design methods of this Code (or the referenced standards) before application of any strength reduction factors.

Required strength. Strength of a member, cross-section, or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by this Code.

Structure. That which is built or constructed and limited to buildings and nonbuilding structures as defined herein.

Structural Observations. The visual observations performed by the registered design professional in responsible charge (or another registered design professional) to determine that the seismic force-resisting system is constructed in general conformance with the construction documents.

Subdiaphragm. A portion of a diaphragm used to transfer wall anchorage forces to diaphragm crossties.

Testing Agency. A company or corporation that provides testing and/or inspection services. The person in charge of the special inspector(s) and the testing services shall be a registered design professional.

Tie-down (hold-down). A device used to resist uplift of the boundary elements of shear walls. These devices are intended to resist load without significant slip between the device and the shear wall boundary element or be shown with cyclic testing to not reduce the wall capacity or ductility.

Torsional Force Distribution. The distribution of horizontal shear through a rigid diaphragm when the center of mass of the structure at the level under consideration does not coincide with the center of rigidity (sometimes referred to as diaphragm rotation).

Toughness. The ability of a material to absorb energy without losing significant strength.

Utility or Service Interface. The connection of the structure's mechanical and electrical distribution systems to the utility or service company's distribution system.

Veneers. Facings or ornamentation of brick, concrete, stone, tile, or similar materials attached to a backing.

Wall. A component that has a slope of 60 degrees or greater with the horizontal plane used to enclose or divide space.

Bearing wall. Any wall meeting either of the following classifications:

1. Any metal wall that supports more than (1500 N/m) of vertical load in addition to its own weight.
2. Any concrete or masonry wall that supports more than (3000 N/m) of vertical load in addition to its own weight.

Nonbearing wall. Any wall that is not a bearing wall.

Nonstructural wall. All walls other than bearing walls or shear walls.

Shear wall (vertical diaphragm). A wall, bearing or nonbearing, designed to resist lateral seismic forces acting in the plane of the wall (sometimes referred to as a vertical diaphragm).

WALL SYSTEM, BEARING. A structural system with bearing walls providing support for all or major portions of the vertical loads. Shear walls or braced frames provide seismic force resistance.

SECTION 9.3
SYMBOLS AND NOTATIONS

The unit dimensions used with the items covered by the symbols shall be consistent throughout except where specifically noted. The symbols and definitions presented in this Section apply to these provisions as indicated.

A, B, C, D	=	the Seismic Design Categories as defined in Tables 9.6.a and 9.6.b
A, B, C, D, E, F	=	the Site Classes as defined in Section 9.4.2
A_{ch}	=	cross-sectional area (mm^2) of a component measured to the outside of the special lateral reinforcement
A_o	=	the area of the load-carrying foundation (m^2)
A_{sh}	=	total cross-sectional area of hoop reinforcement (mm^2), including supplementary crossties, having a spacing of s_h and crossing a section with a core dimension of h_c.
A_{vd}	=	required area of leg (mm^2) of diagonal reinforcement
a_p	=	the amplification factor related to the response of a system or component as affected by the type of seismic attachment, determined in Section 12.1.3

b	=	the shortest plan dimension of the structure, in (mm) measured perpendicular to d
C_d	=	the deflection amplification factor as given in Table 10.2
C_s	=	the seismic response coefficient determined in Section 10.9.2 (dimensionless)
C_{sm}	=	the modal seismic response coefficient determined in Section 10.10.5 (dimensionless)
C_{vx}	=	the vertical distribution factor as determined in Section 10.9.4
c	=	distance from the neutral axis of a flexural member to the fiber of maximum compressive strain (mm)
D	=	the effect of dead load
D_p	=	relative seismic displacement that the component must be designed to accommodate as defined in Section 12.1.4
d	=	overall depth of member (mm) in Chapter 10
d_p	=	the longest plan dimension of the structure, in (mm)
E	=	the effect of horizontal and vertical earthquake-induced forces, in Section 10.4
e	=	the actual eccentricity, (mm), measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in (mm), taken as 5% of the maximum building dimension perpendicular to the direction of force under consideration
F_a	=	acceleration-based site coefficient (at 0.2-sec period)
F_i , F_n, F_x	=	the portion of the seismic base shear, V, induced at Level i, n, or x, respectively, as determined in Section 10.9.4
F_p	=	the seismic force acting on a component of a structure
F_v	=	velocity-based site coefficient (at 1.0-sec period)
F_{xm},	=	the portion of the seismic base shear, V_m, induced at Level x as determined in Section 10.10.6
$f_c^{'}$	=	specified compressive strength of concrete used in design
$f_S^{'}$	=	ultimate tensile strength (MPa) of the bolt, stud, or insert leg wires. For A307 bolts or A108 studs, it is permitted to be assumed to be 415 MPa.
f_y	=	specified yield strength of reinforcement (MPa)
g	=	the acceleration due to gravity
H	=	thickness of soil
h	=	the height of a shear wall measured as the maximum clear height from top of foundation to bottom of diaphragm framing above, or the maximum clear height from top of diaphragm to bottom of diaphragm framing above
h	=	the roof elevation of a structure in Chapter 12
h	=	the effective height of the building as determined in Chapter 10
h_c	=	the core dimension of a component measured to the outside of the special lateral reinforcement (mm)
h_i, h_n, h_x	=	the height above the base Level i, n, or x, respectively
h_{sx}	=	the story height below Level $x = (h_x - h_{x-1})$
I	=	the occupancy importance factor in Section 9.5
I_p	=	the component importance factor as prescribed in Section 12.1.5

i	=	the building level referred to by the subscript i; i = 1 designates the first level above the base
K_p	=	the stiffness of the component or attachment, Section 12.3.3
KL/r	=	the lateral slenderness of a compression member measured in terms of its effective buckling length, KL, and the least radius of gyration of the member cross-section, r
k	=	the distribution exponent given in Section 10.9.4
k	=	the stiffness of the building
L	=	the overall length of the building (m) at the base in the direction being analyzed
l	=	the dimension of a diaphragm perpendicular to the direction of application of force. For open-front structures, ℓ is the length from the edge of the diaphragm at the open-front to the vertical resisting elements parallel to the direction of the applied force. For a cantilevered diaphragm, ℓ is the length of the cantilever
M_f	=	the foundation overturning design moment as defined in Section 10.9.6 (kN-m)
M_t	=	the torsional moment resulting from the location of the building masses, Section 10.9.5
M_{ta}	=	the accidental torsional moment as determined in Section 10.9.5
M_x	=	the building overturning design moment at Level x as defined in Section 10.9.6
N	=	number of stories, Section 10.9.3
$\bar{N}$	=	standard penetration resistance, ASTM D1586-84
N	=	average field standard penetration resistance for the top (30 m), see Section 14.1.1
N_{ch}	=	average standard penetration resistance for cohesionless soil layers for the top (30 m), see Section 14.1.1
n	=	designates the level that is uppermost in the main portion of the building
P_n	=	the algebraic sum of the shear wall and the minimum gravity loads on the joint surface acting simultaneously with the shear (N)
P_x	=	the total unfactored vertical design load at, and above, Level x for use in Section 10.9.7.2
Q_E	=	plasticity index, ASTM D4318-93
R		The effect of horizontal seismic (earthquake-induced) forces, Section 10.4
R_p	=	the component response modification factor as defined in Section 12.1.3
r_x	=	the ratio of the design story shear resisted by the most heavily loaded single element in the story, in direction x, to the total story shear
S_S	=	the mapped maximum considered earthquake, 5% damped, spectral response acceleration at short periods as defined in Section 9.4.1
S_1	=	the mapped maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 sec as

		defined in Section 9.4.1
S_{DS}	=	the design, 5% damped, spectral response acceleration at short periods as defined in Section 9.4.4
S_{D1}	=	the design, 5% damped, spectral response acceleration at a period of 1 sec as defined in Section 9.4.4
S_{MS}	=	the maximum considered earthquake, 5% damped, spectral response acceleration at short periods adjusted for site class effects as defined in Section 9.4.3
S_{M1}	=	the maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 sec adjusted for site class effects as defined in Section 9.4.3
$\overline{s_u}$	=	average undrained shear strength in top (30 m); see Section 14.1.1, ASTM D2166-91 or ASTM D2850-87
s_h	=	spacing of special lateral reinforcement (mm)
T	=	the fundamental period of the building as determined in Section 10.9.3
T_a	=	the approximate fundamental period of the building as determined in Section 10.9.3.2
T_m	=	the modal period of vibration (sec) of the mth mode of the building as determined in Section 10.10.5
T_p	=	the fundamental period of the component and its attachment, Section 12.3.3
T_o	=	$0.2S_{D1}/S_{DS}$
T_s	=	S_{D1}/S_{DS}
V	=	the total design lateral force or shear at the base
V_t	=	the design value of the seismic base shear as determined in Section 10.10.8
V_x	=	the seismic design shear in story x as determined in Section 10.9.5
$\overline{V_s}$	=	average shear wave velocity in top (30 m), see Section 14.1.1
$\overline{V_{so}}$	=	the average shear wave velocity for the soils beneath the foundation at small strain levels, (m/s)
W	=	the total gravity load of the building. For calculation of seismic-isolated building period, W is the total seismic dead load weight of the building
W_c	=	the gravity load of a component of the building
W_m	=	the effective modal gravity load determined in accordance with Eq. 10.10.5-2
W_p	=	component operating weight (N)
w	=	the width of a diaphragm or shear wall in the direction of application of force. For sheathed diaphragms, the width shall be defined as the dimension between the outside faces of the tension and compression chords
w	=	moisture content (in percent), ASTM D2216-92 [3]
w_i, w_n, w_x	=	the portion of W that is located at or assigned to Level *i, n,* or x, respectively
x	=	the level under consideration
x	=	1 designates the first level above the base
y	=	elevations difference between points of attachment
z	=	the level under consideration; z = 1 designates the first level above the base

β	=	ratio of shear demand to shear capacity for the story between Level x and x - 1
γ	=	the average unit weight of soil (kg/m^3)
Δ	=	the design story drift as determined in Section 10.9.7.1
Δ_a	=	the allowable story drift as specified in Section 10.12
δ_{max}	=	the maximum displacement at Level x, considering torsion, Section 10.9.5.2
δ_{avg}	=	the average of the displacements at the extreme points of the structure at Level x, Section 10.9.5.2
θ	=	the stability coefficient for P-delta effects as determined in Section 10.9.7.2
ρ	=	a reliability coefficient based on the extent of structural redundance present in a building
ϕ	=	the strength reduction factor or resistance factor
ϕ_{im}	=	the displacement amplitude at the ith level of the building for the fixed-base condition when vibrating in its mth mode, Section 10.10.5
Ω_o	=	over strength factor as defined in Table 10.2

SECTION 9.4
SEISMIC GROUND MOTION VALUES

9.4.1 **Mapped Acceleration Parameters.** The Kingdom of Saudi Arabia has been divided into seven regions for determining the maximum considered earthquake ground motion as shown in Figure 9.4.1(a). The parameter S_S shall be determined from the 0.2 second spectral response accelerations shown on Figures 9.4.1(b) through 9.4.1(i). The parameter S_1 shall be determined from the 1.0 second spectral response accelerations shown on Figures 9.4.1(j) through 9.4.1(q). Where S_1, is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned Seismic Design Category A.

9.4.2 **Site Class.** Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E or F in accordance with Chapter 14. Where the soil properties are not known in sufficient detail to determine the Site Class, Site Class D or E shall be used, as per Section 14.1.

9.4.3 **Site Coefficients and Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameters.** The maximum considered earthquake spectral response acceleration for short periods (S_{MS}) and at 1-sec (S_{M1}), adjusted for site class effects, shall be determined by Eqs. 9.4.3-1 and 9.4.3-2, respectively.

$$S_{MS} = F_a S_S \qquad \textbf{(Eq. 9.4.3-1)}$$

$$S_{M1} = F_v S_1 \qquad \textbf{(Eq. 9.4.3-2)}$$

where

S_1 = the mapped maximum considered earthquake spectral response acceleration at a period of 1-sec as determined in accordance with Section 9.4.1

S_S = the mapped maximum considered earthquake spectral response acceleration at short periods as determined in accordance with Section 9.4.1 where site coefficients F_a and F_v are defined in Table 9.4.3a and Table 9.4.3b, respectively.

9.4.4 **Design Spectral Response Acceleration Parameters.** Design earthquake spectral response acceleration at short periods, S_{DS}, and at 1-sec period, S_{D1}, shall be determined from Eqs. 9.4.4-1 and 9.4.4-2, respectively.

$$S_{DS} = \frac{2}{3} S_{MS} \qquad \textbf{(Eq. 9.4.4-1)}$$

$$S_{D1} = \frac{2}{3} S_{M1} \qquad \textbf{(Eq. 9.4.4-2)}$$

TABLE 9.4.3a:
VALUES OF F_a AS A FUNCTION OF SITE CLASS AND MAPPED SHORT PERIOD MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration at Short Periods				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	a	a	a	a	a

Note: Use straight-line interpolation for intermediate values of S_S.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed except that for structures with periods of vibration equal to or less than 0.5-seconds, values of F_a for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquefaction in Step 3 of Section 14.1.2.

TABLE 9.4.3b:

VALUES OF F_v AS A FUNCTION OF SITE CLASS AND MAPPED 1-SECOND PERIOD MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration at 1-Second Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	a	a	a	a	a

Note: Use straight-line interpolation for intermediate values of S_1.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed except that for structures with periods of vibration equal to or less than 0.5-seconds, values of F_v for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquefaction in Step 3 of Section 14.1.2.

9.4.5 **Design Response Spectrum.** Where a design response spectrum is required by these provisions, the design response spectrum curve shall be developed as indicated in Figure 9.4.5 and as follows:

1. For periods less than or equal to T_o, the design spectral response acceleration, S_a, shall be taken as given by Eq. 9.4.5-1:

$$S_a = S_{DS}\,(0.4 + 0.6\,\frac{T}{T_o}) \qquad \textbf{(Eq. 9.4.5-1)}$$

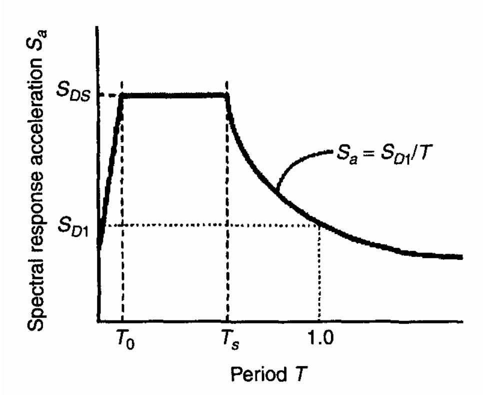


FIGURE 9.4.5
DESIGN RESPONSE SPECTRUM

2. For periods greater than or equal to T_o and less than or equal to T_s, the design spectral response acceleration, S_a, shall be taken as equal to S_{DS}.

3. For periods greater than T_s, the design spectral response acceleration, S_a, shall be taken as given by Eq. 9.4.5-2:

$$S_a = \frac{S_{D1}}{T} \qquad \textbf{(Eq. 9.4.5-2)}$$

S_{DS} = the design spectral response acceleration at short periods

S_{D1} = the design spectral response acceleration at 1-sec period, in units of g-sec

T = the fundamental period of the structure (sec)

T_o = 0.2 S_{D1}/S_{DS} and

T_S = S_{D1}/S_{DS}

SECTION 9.5
OCCUPANCY IMPORTANCE FACTOR

An occupancy importance factor, I, shall be assigned to each structure in accordance with Table 9.5. The Occupancy Category shall be determined from Table 1.6-1.

TABLE 9.5: OCCUPANCY IMPORTANCE FACTORS

Occupancy Category	I
I or II	1.0
III	1.25
IV	1.5

SECTION 9.6
SEISMIC DESIGN CATEGORY

Structures shall be assigned a Seismic Design Category in accordance with Section 9.6.1.

9.6.1 **Determination of Seismic Design Category.** All structures shall be assigned to a Seismic Design Category based on their Occupancy Category and the design spectral response acceleration coefficients, S_{DS} and S_{D1}, determined in accordance with Section 9.4.4. Each building and structure shall be assigned to the most severe Seismic Design Category in accordance with Table 9.6.a or 9.6.b, irrespective of the fundamental period of vibration of the structure, T.

TABLE 9.6. a: SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATIONS

Value of S_{DS}	Occupancy Category		
	I-II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$S_{DS} \geq 0.50g$	D	D	D

TABLE 9.6. b: SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATIONS

Value of S_{D1}	Occupancy Category		
	I-II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$S_{D1} \geq 0.20g$	D	D	D

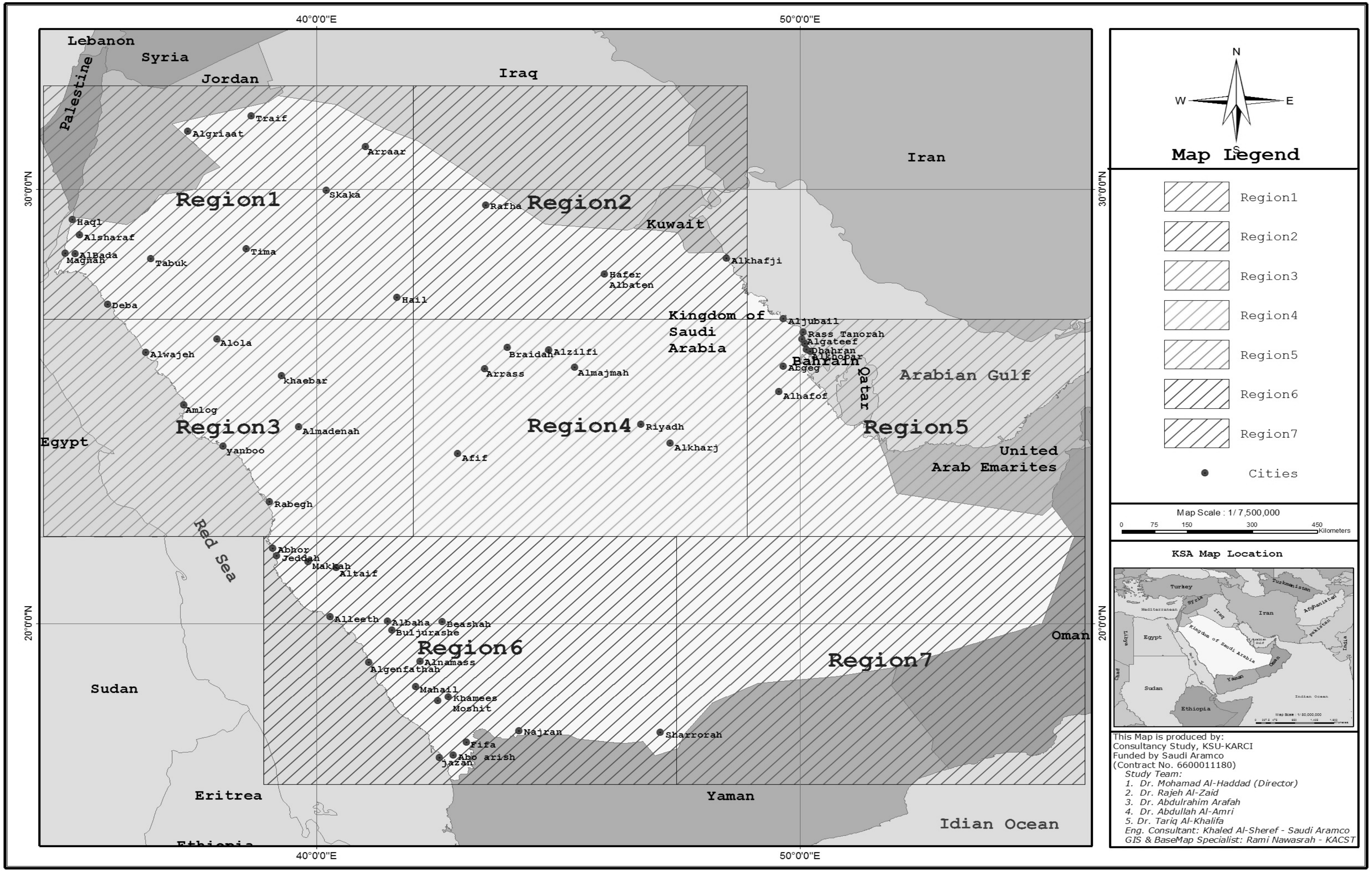


Figure 9.4.1(a): Regions for Determination of the Maximum Considered Earthquake Ground Motion in the Kingdom of Saudi Arabia.

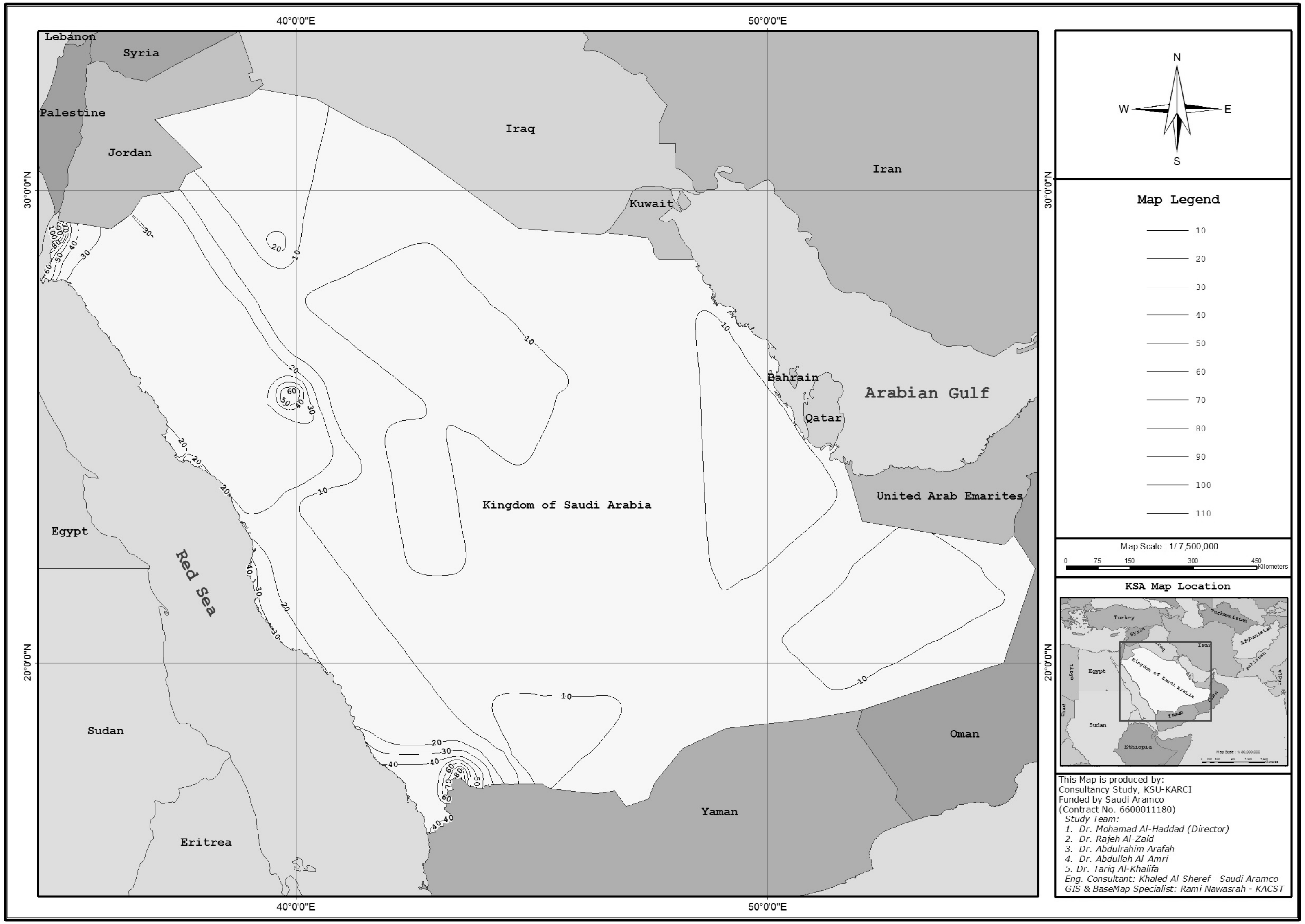


Figure 9.4.1(b): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(All Regions)

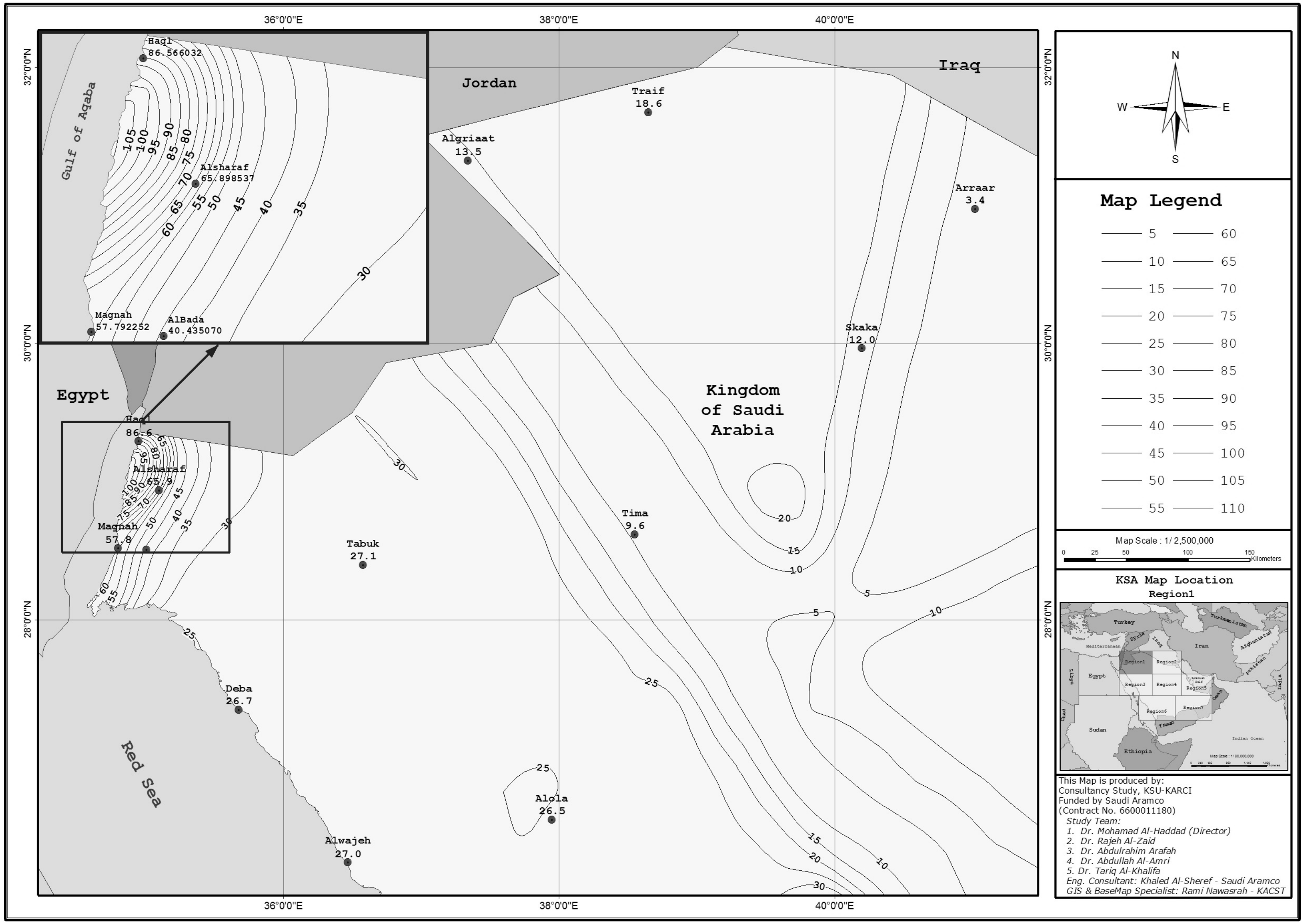


Figure 9.4.1(c): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(Region 1)

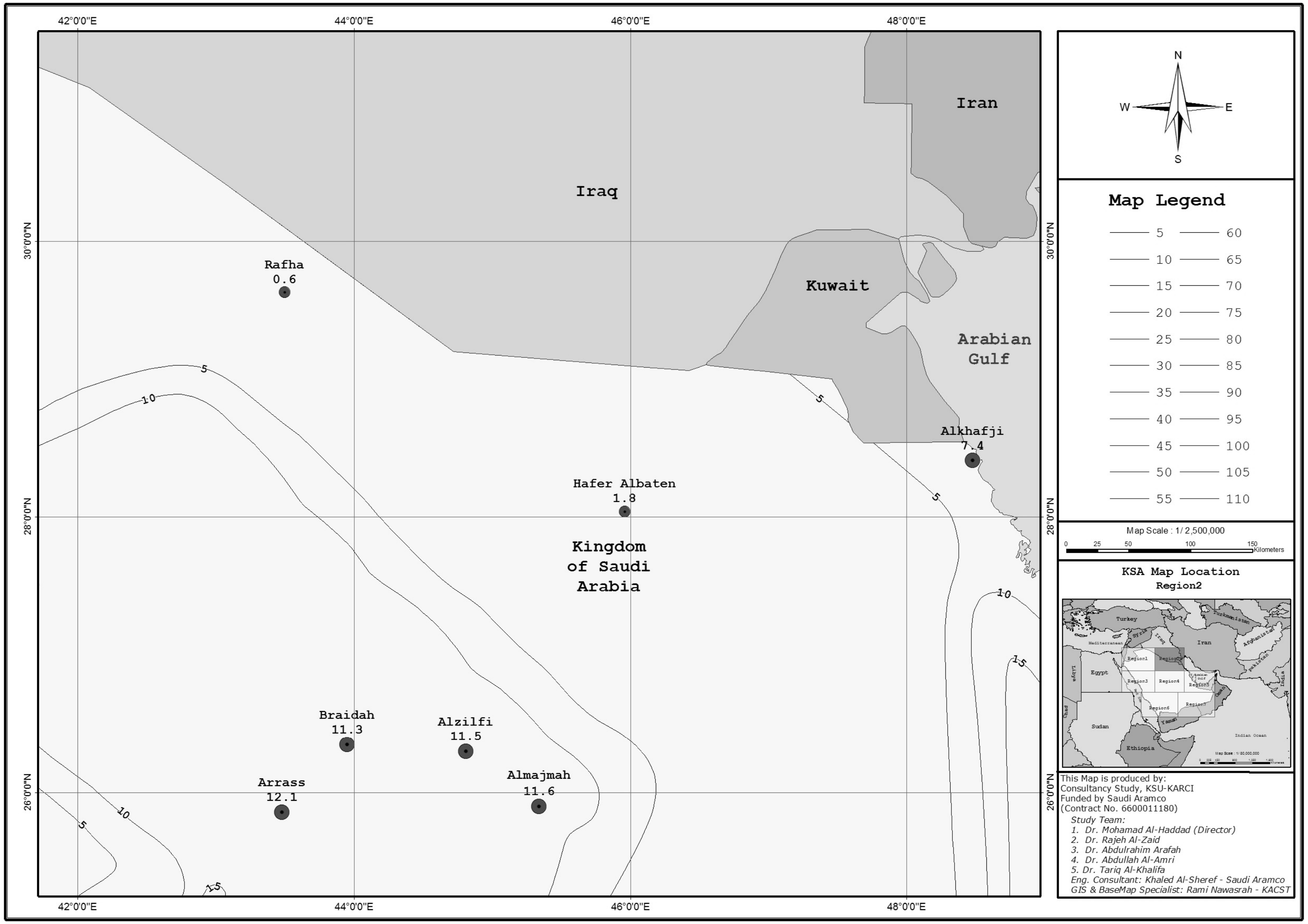


Figure 9.4.1(d): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in%g) (5 Percent of Critical Damping), Site Class B.

(Region 2)

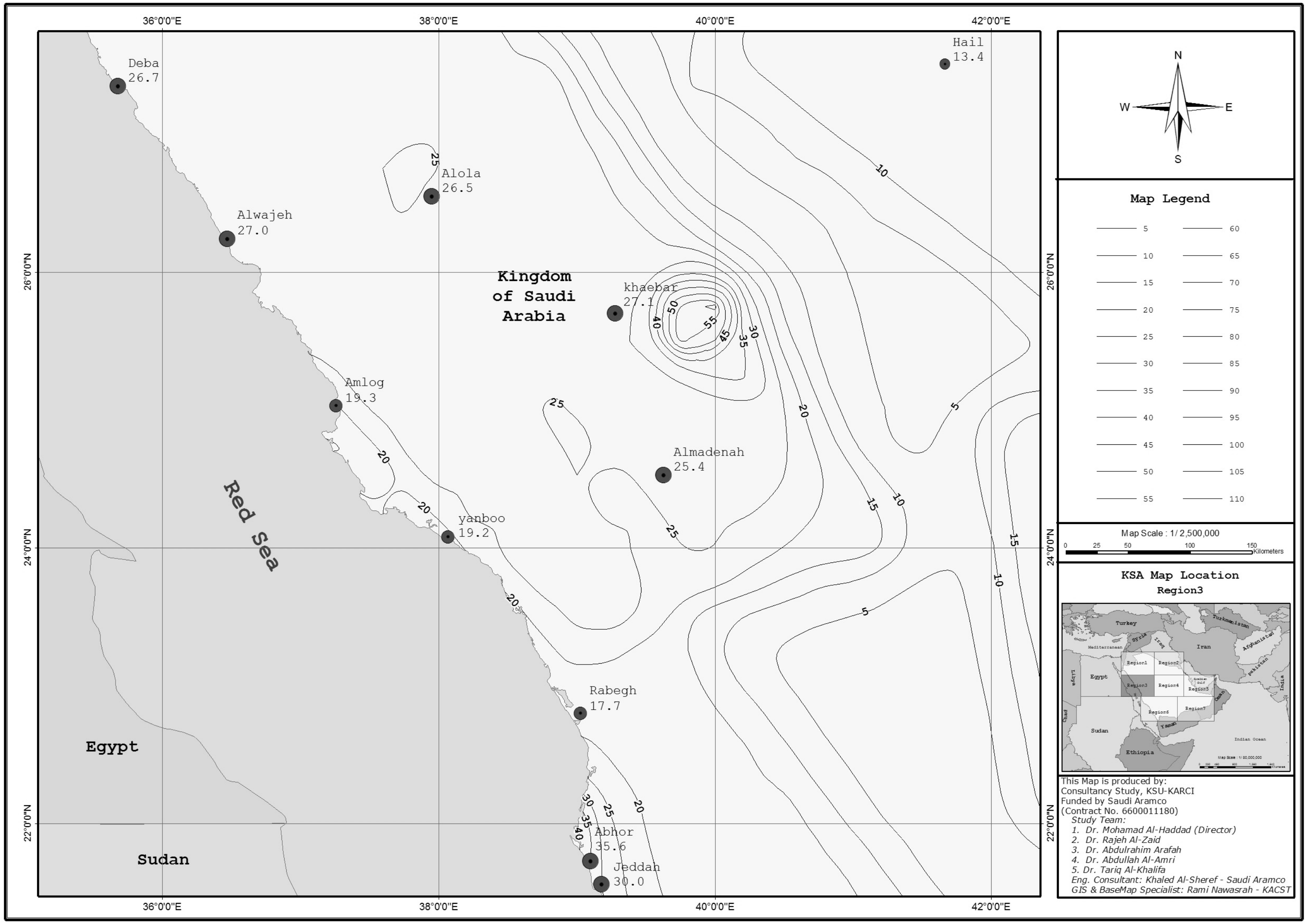


Figure 9.4.1(e): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(Region 3)

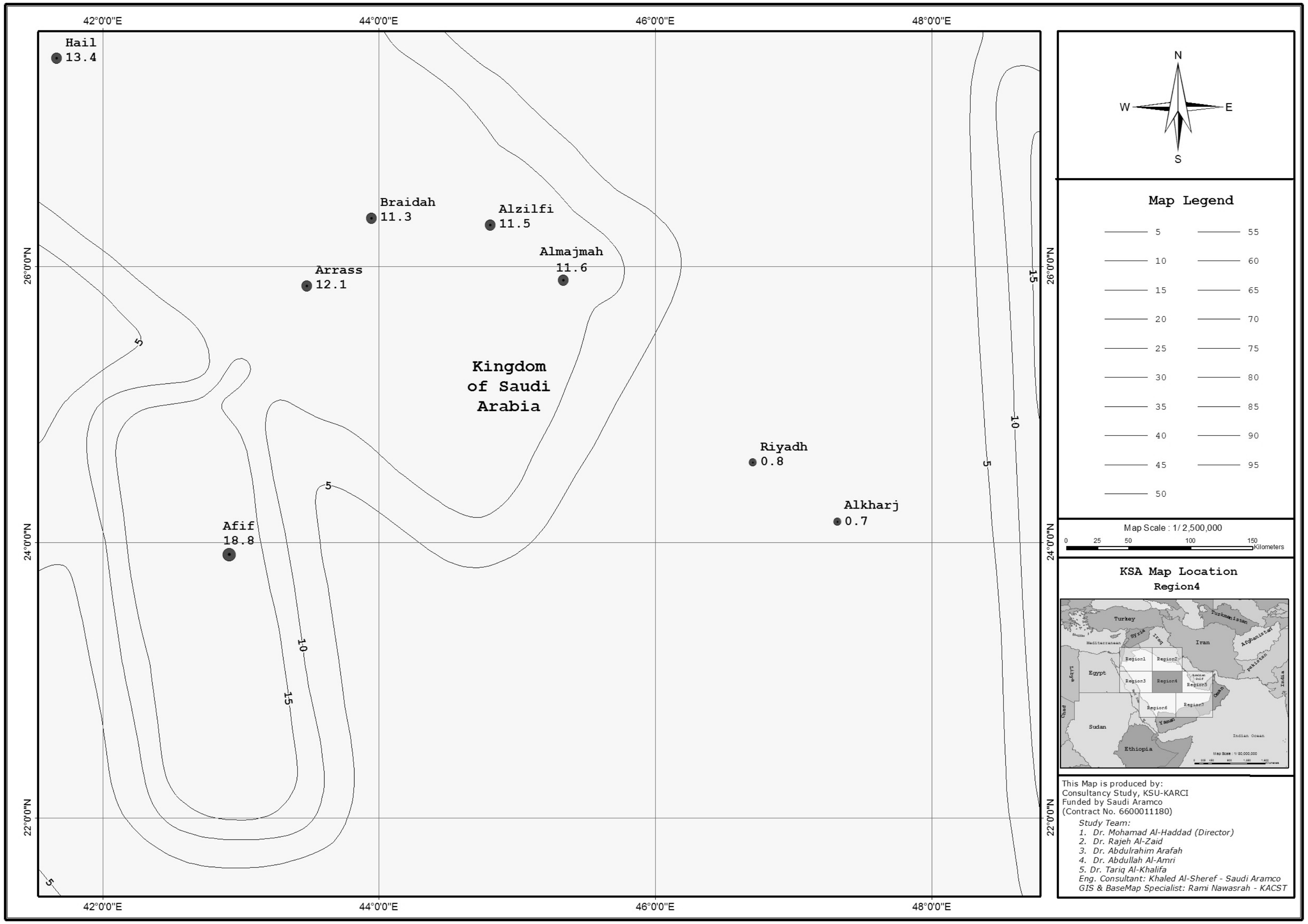


Figure 9.4.1(f): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(Region 4)

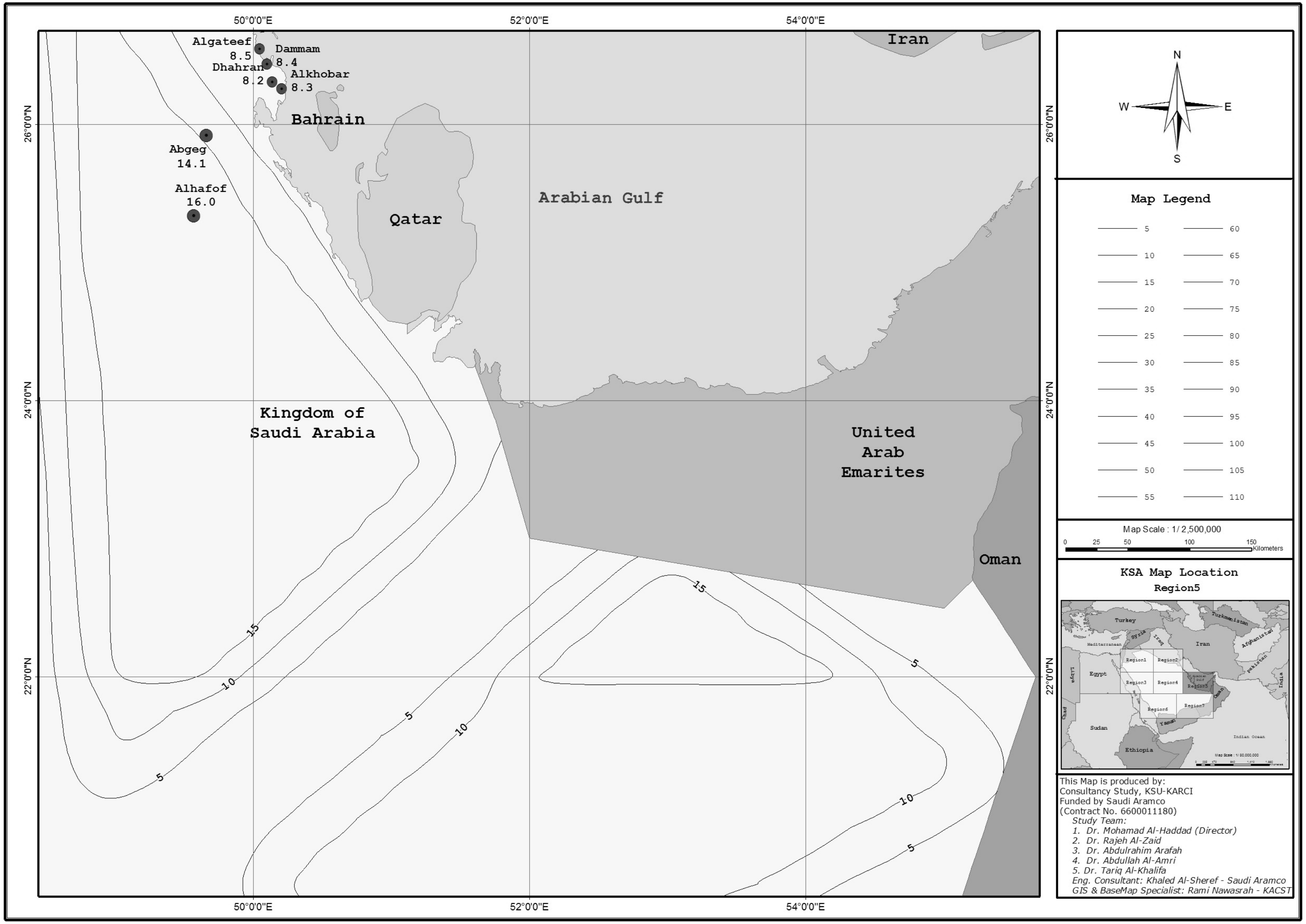


Figure 9.4.1(g): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in%g) (5 Percent of Critical Damping), Site Class B.

(Region 5)

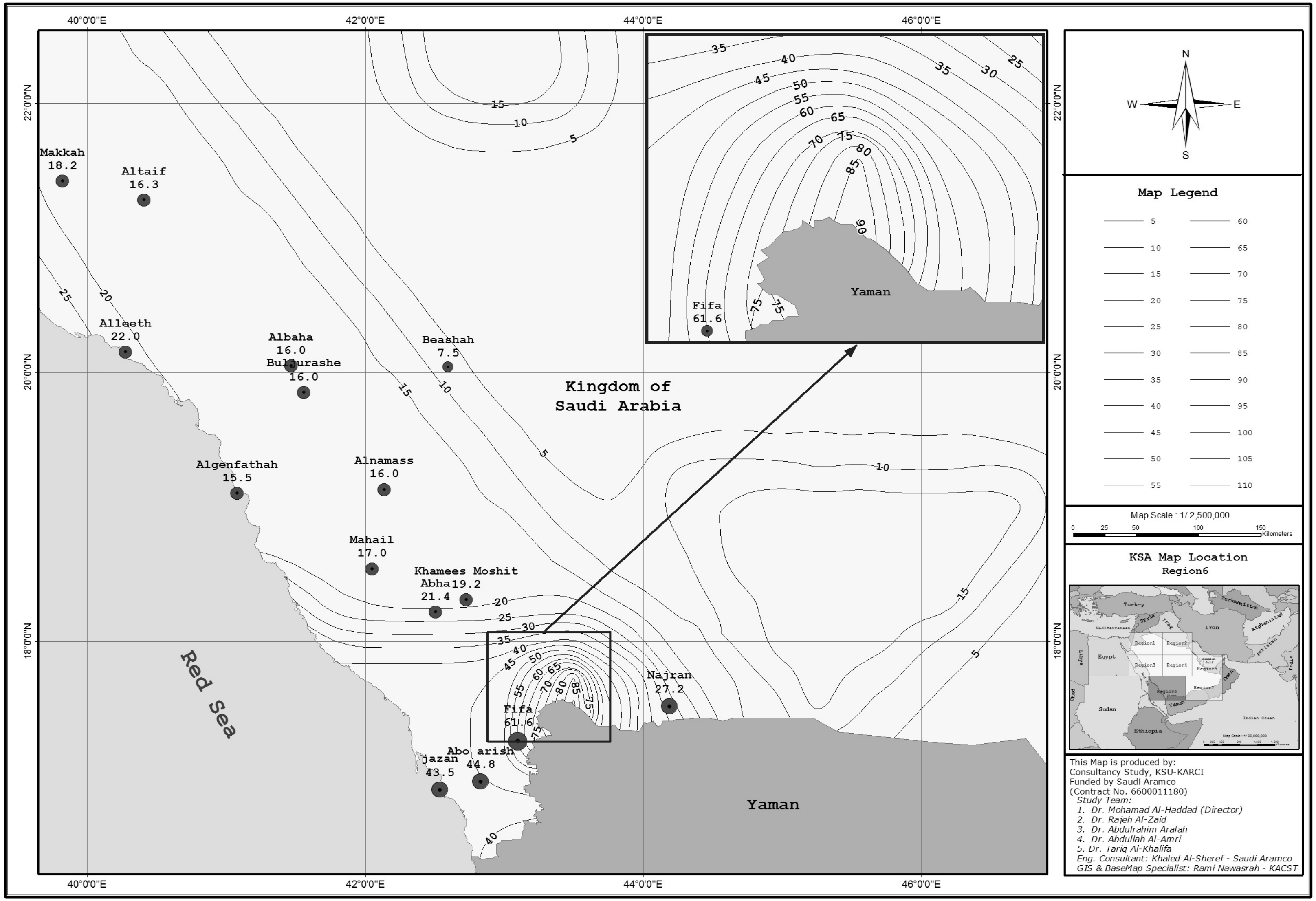


Figure 9.4.1(h): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(Region 6)

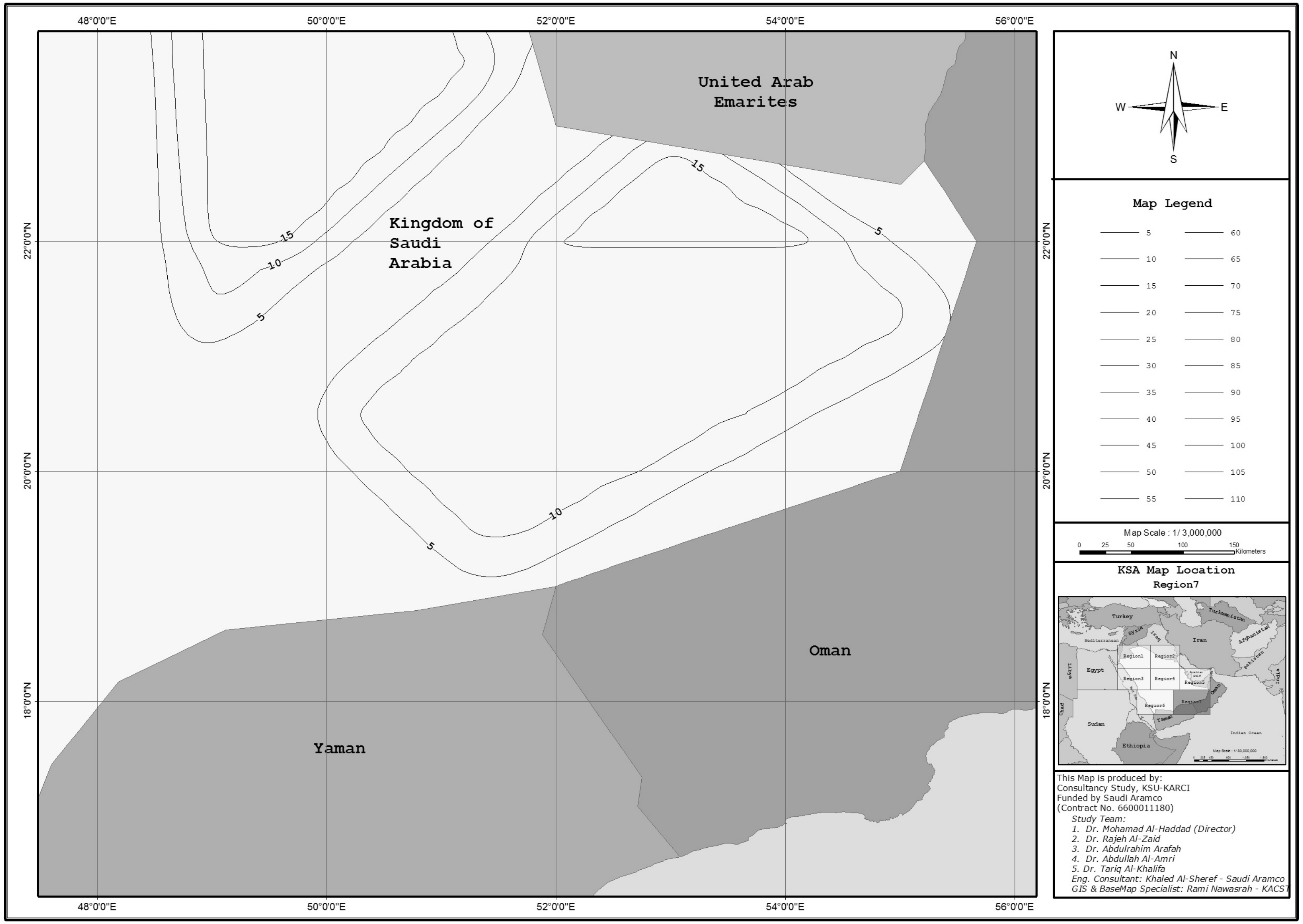


Figure 9.4.1(i): Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 SEC Spectral Response Acceleration (S_s in %g) (5 Percent of Critical Damping), Site Class B.

(Region 7)

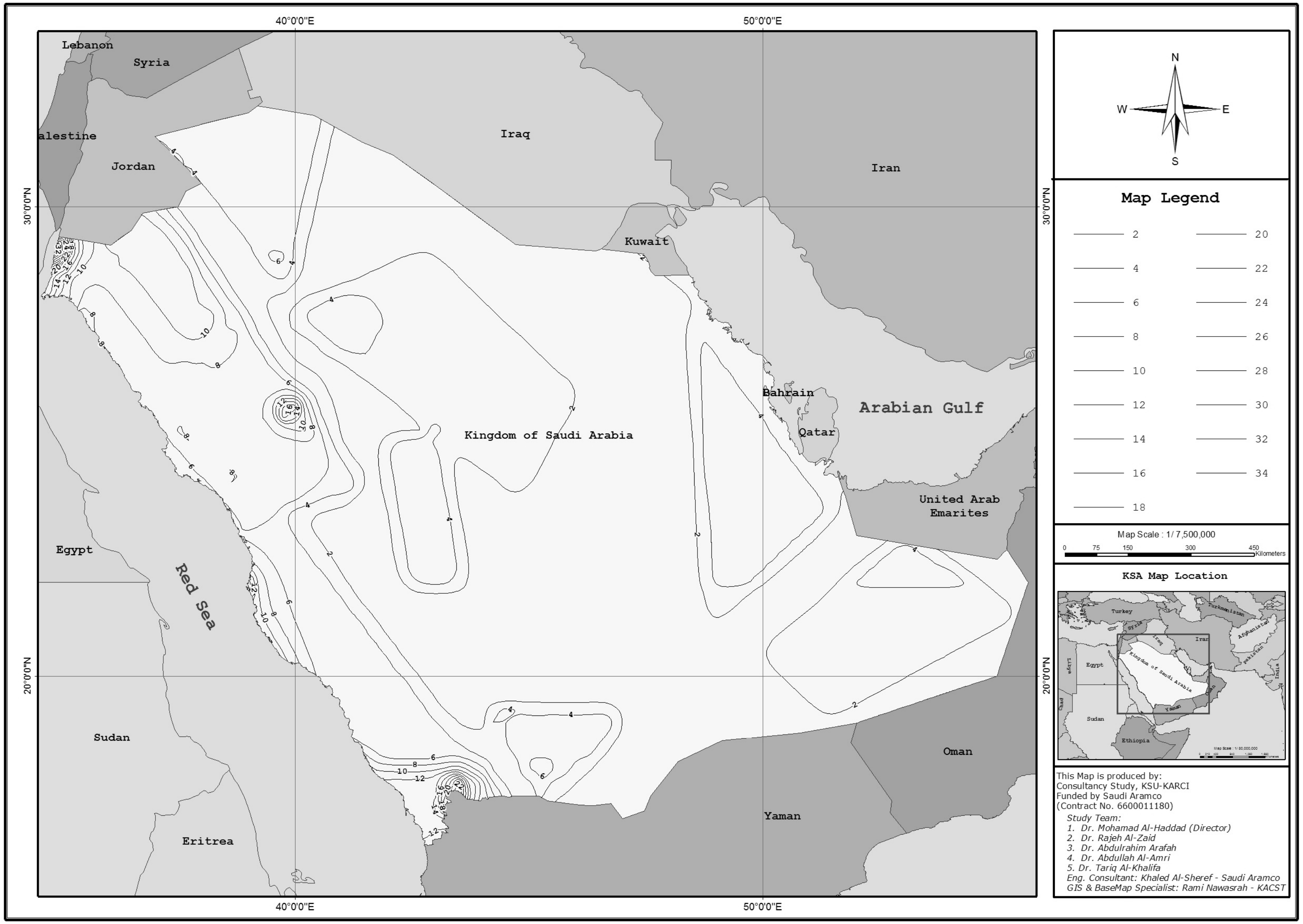


Figure 9.4.1(j): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.

(All Regions)

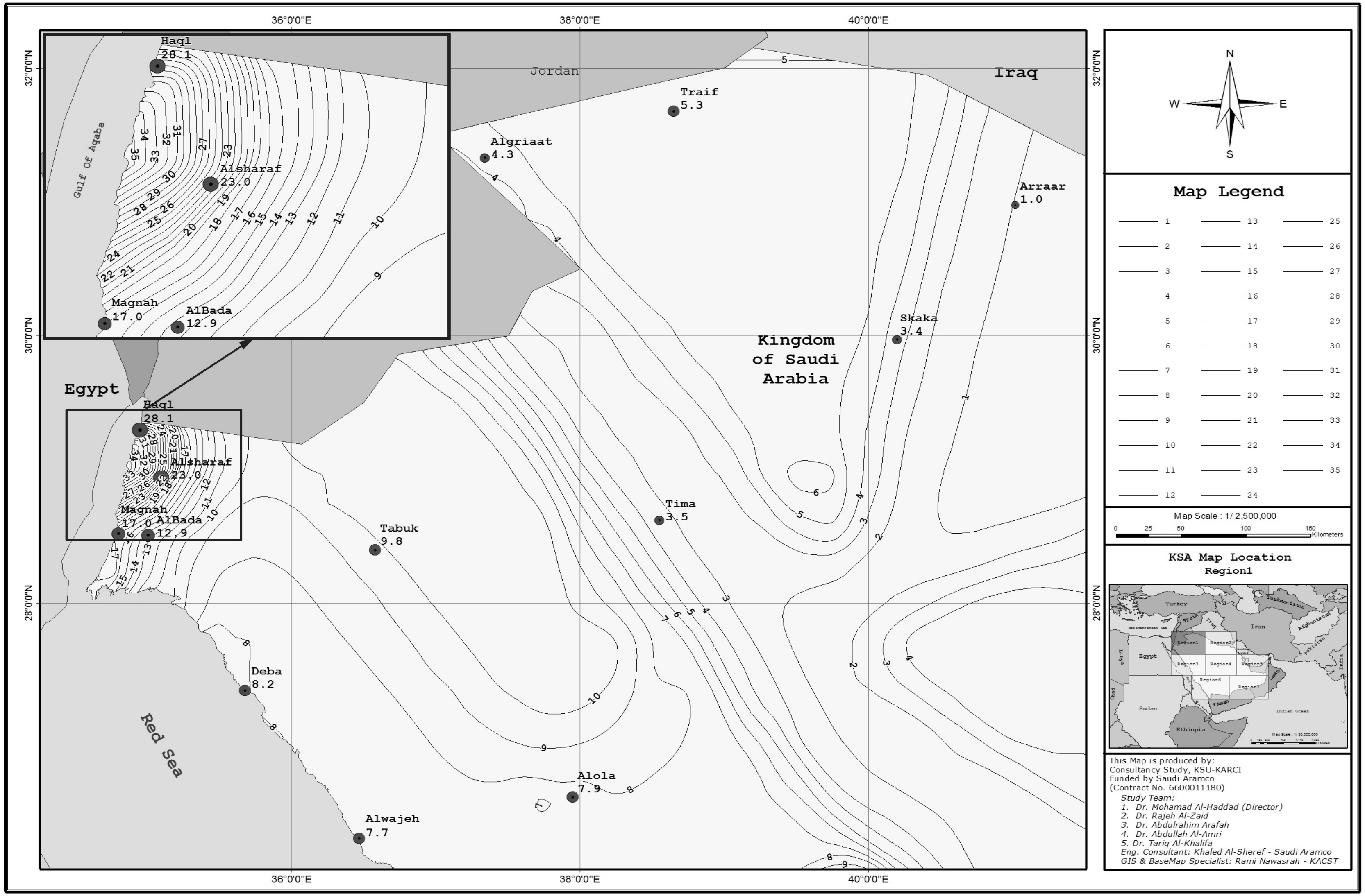


Figure 9.4.1(k): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.

(Region 1)

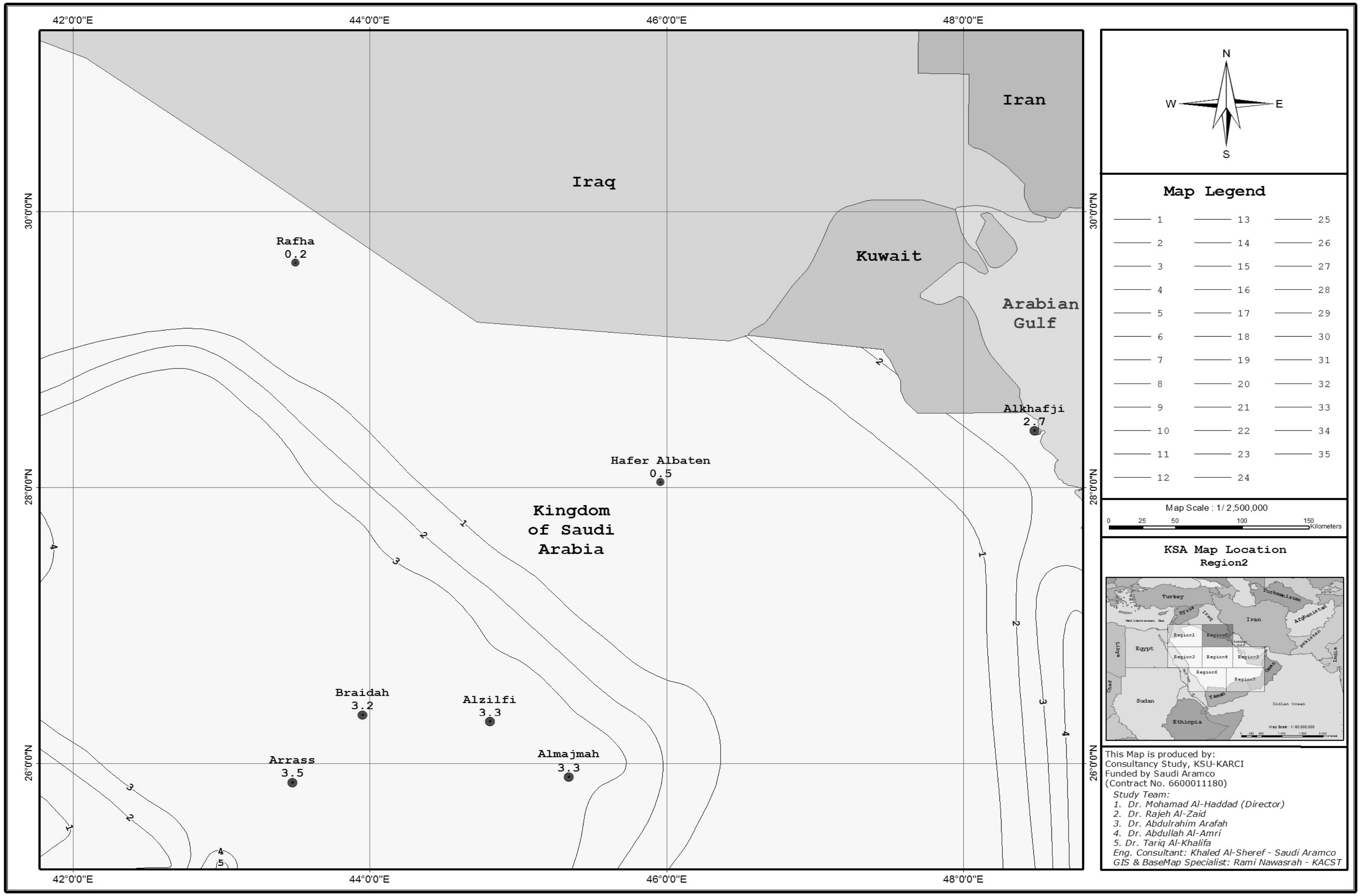


Figure 9.4.1(l): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in % g) (5 Percent of Critical Damping), Site Class B.

(Region 2)

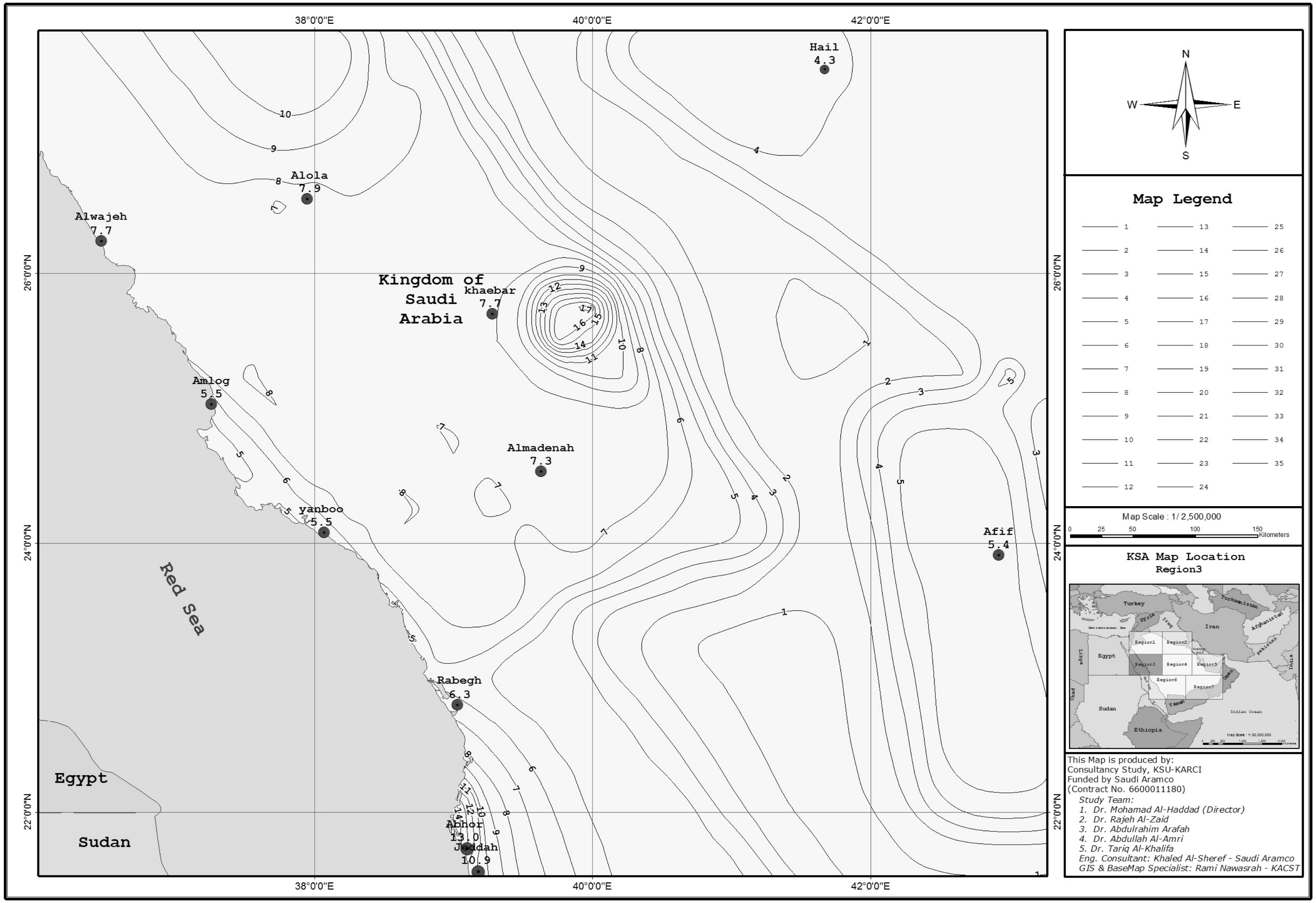


Figure 9.4.1(m): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.

(Region 3)

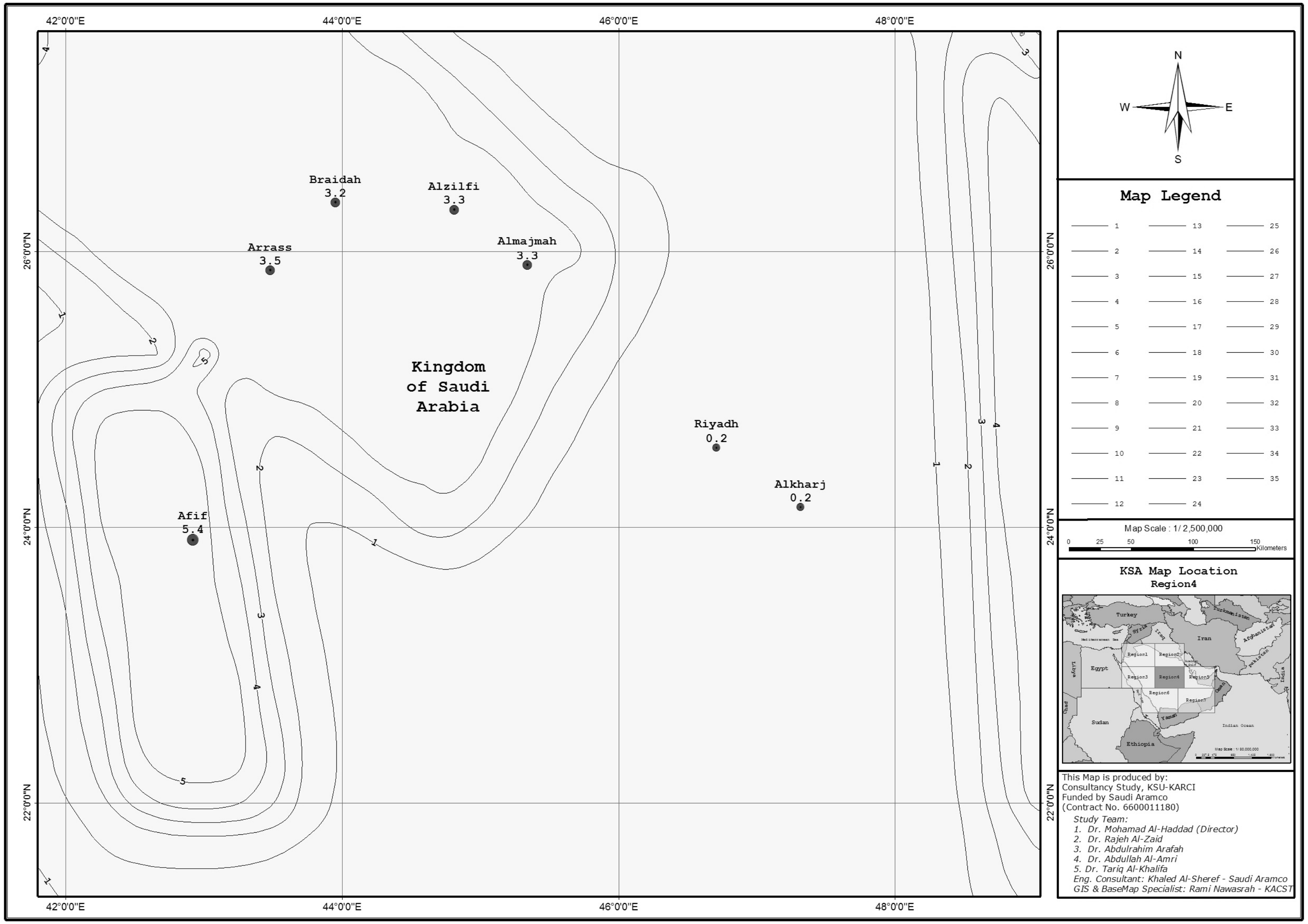


Figure 9.4.1(n): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.

(Region 4)

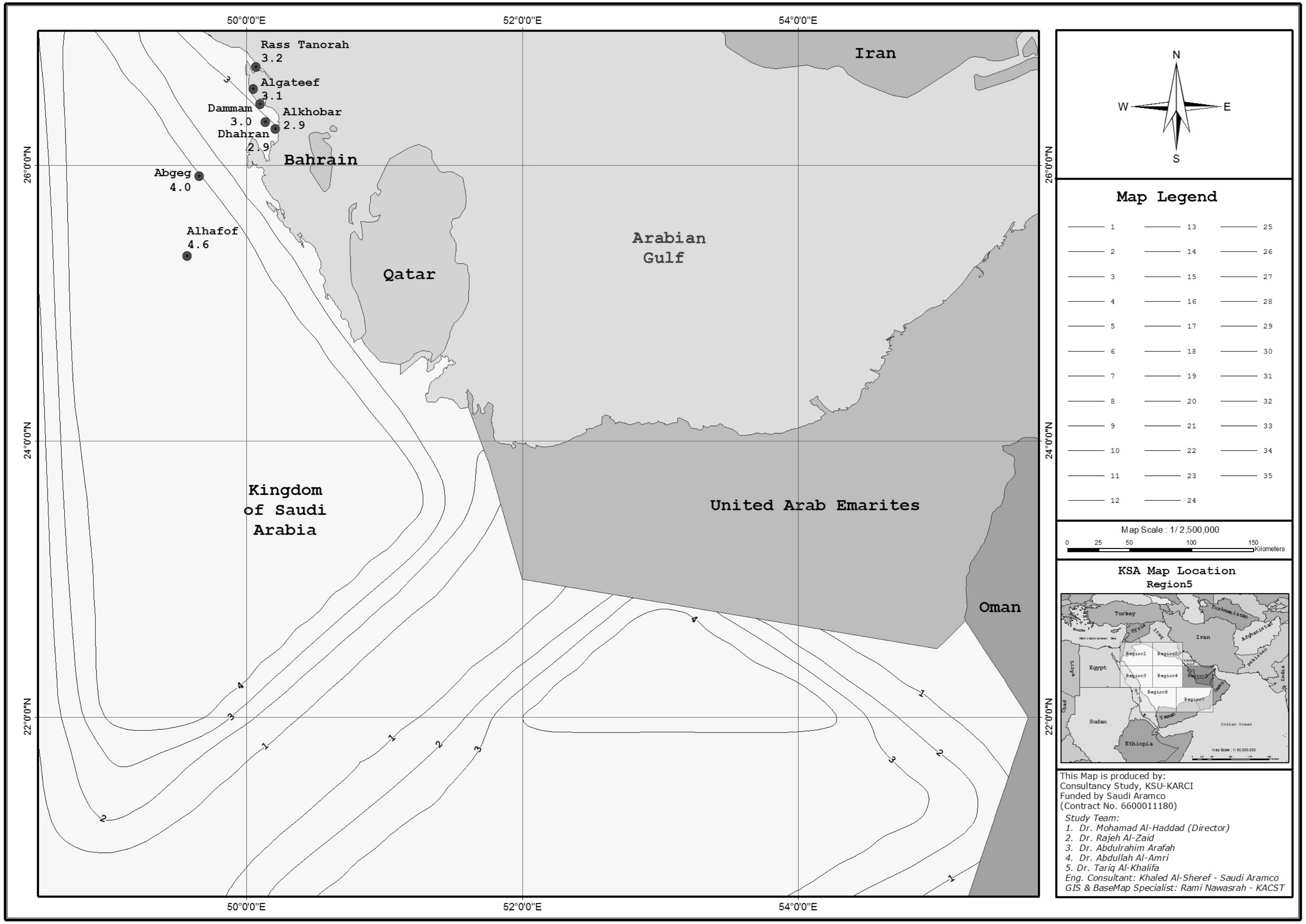


Figure 9.4.1(o): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.

(Region 5)

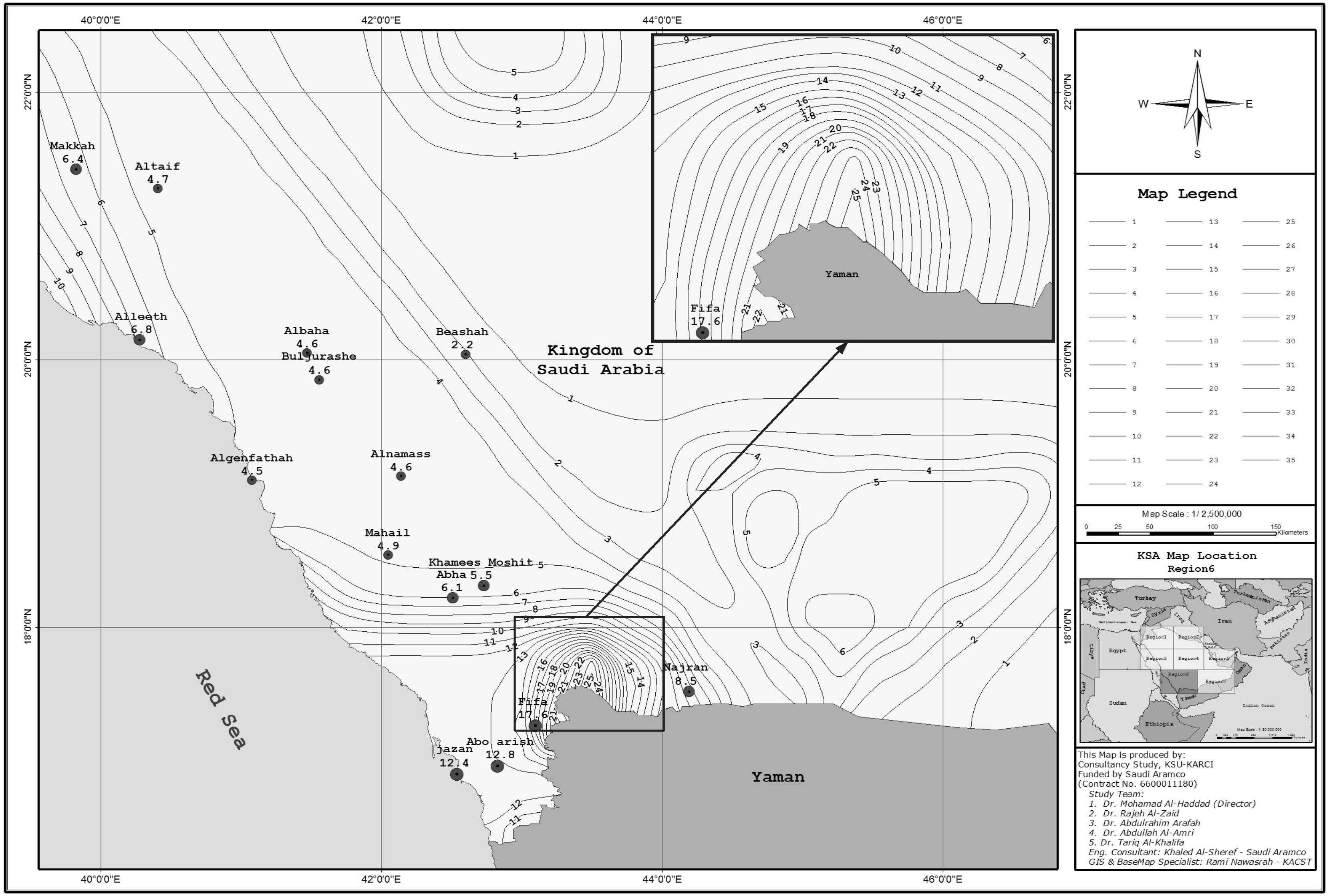


Figure 9.4.1(p): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration (S_1 in %g) (5 Percent of Critical Damping), Site Class B.

(Region 6)

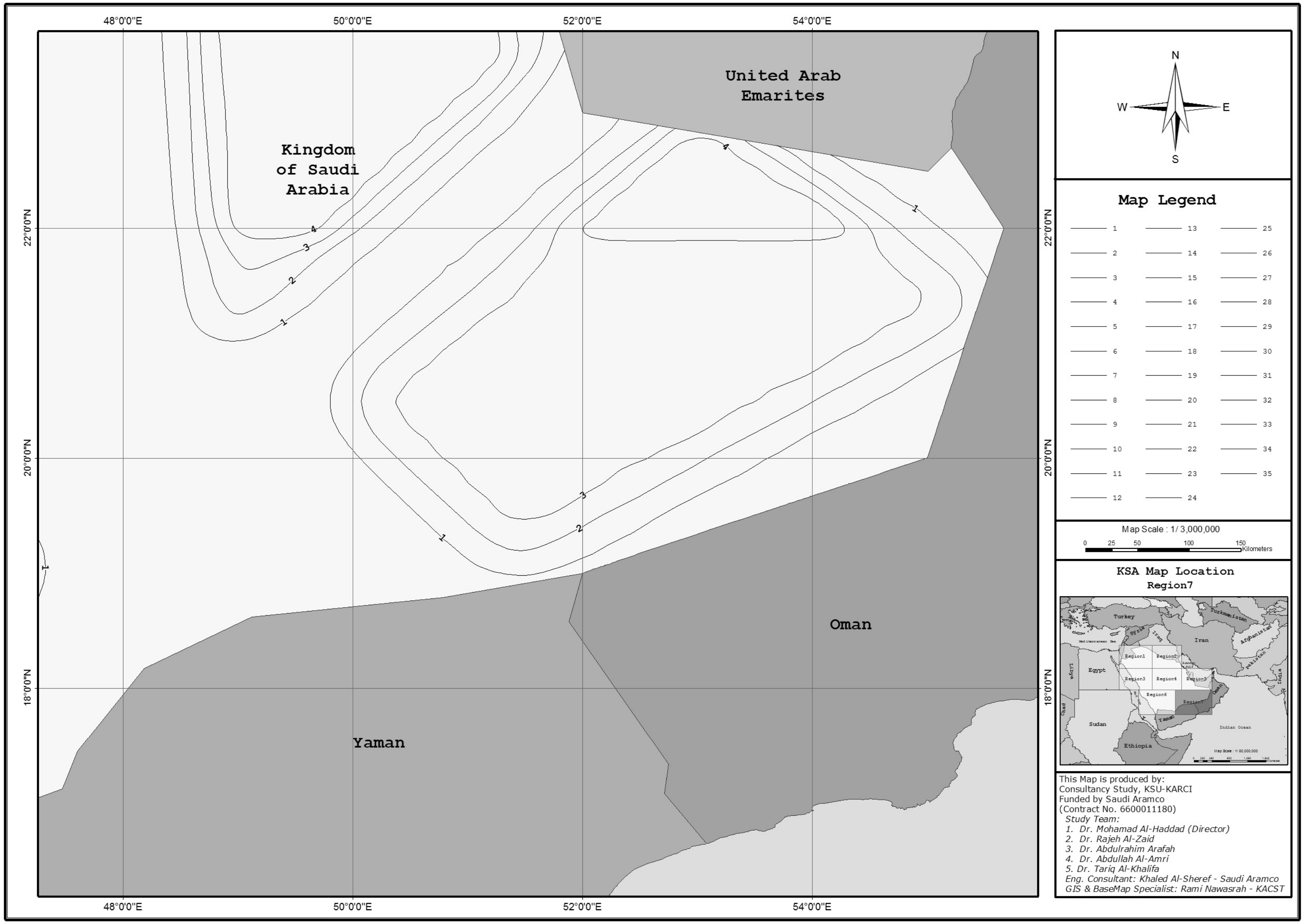


Figure 9.4.1(q): Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC Spectral Response Acceleration ($S_{1\ in}$ %g) (5 Percent of Critical Damping), Site Class B.

(Region 7)

CHAPTER 10
SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

SECTION 10.1
STRUCTURAL DESIGN BASIS

10.1.1 **Basic Requirements.** The seismic analysis and design procedures to be used in the design of structures and their components shall be as prescribed in this Section. The structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions. The design seismic forces, and their distribution over the height of the structure, shall be established in accordance with one of the applicable procedures indicated in Section 10.6 and the corresponding internal forces and deformations in the members of the structure shall be determined. An approved alternative procedure shall not be used to establish the seismic forces and their distribution unless the corresponding internal forces and deformations in the members are determined using a model consistent with the procedure adopted.

Individual members shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with these provisions, and connections shall develop the strength of the connected members or the forces indicated above. The deformation of the structure shall not exceed the prescribed limits when the structure is subjected to the design seismic forces.

A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the final point of resistance. The foundation shall be designed to resist the forces developed and accommodate the movements imparted to the structure by the design ground motions. The dynamic nature of the forces, the expected ground motion, and the design basis for strength and energy dissipation capacity of the structure shall be included in the determination of the foundation design criteria.

Allowable Stress Design is permitted to be used to evaluate sliding, overturning, and soil bearing at the soil-structure interface regardless of the design approach used in the design of the structure.

SECTION 10.2
STRUCTURAL SYSTEM SELECTION

The basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 10.2. Each type is subdivided by the types of vertical element used to resist lateral seismic forces. The structural system used shall be in accordance with the Seismic Design Category and height limitations indicated in Table 10.2. The appropriate response modification coefficient, R, system overstrength factor, Ω_o, and the deflection amplification factor (C_d) indicated in Table 10.2 shall be used in determining the base shear, element design forces, and design story drift. Special framing requirements are indicated in Section 10.11 and Sections 11.1, 11.2, 11.3 and 11.4 for structures assigned to the various Seismic Design Categories.

Seismic force-resisting systems that are not contained in Table 10.2 shall be permitted if analytical and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 10.2 for equivalent response modification coefficient, *R,* system overstrength coefficient, Ω_o, and deflection amplification factor, C_d, values.

10.2.1 **Dual System.** For a dual system, the moment frame shall be capable of resisting at least 25% of the design seismic forces. The total seismic-force resistance is to be provided by the combination of the moment frame and the shear walls or braced frames in proportion to their rigidities.

10.2.2 **Combinations of Framing Systems.** Different seismic force-resisting systems are permitted along the two orthogonal axes of the structure. Combinations of seismic force-resisting systems shall comply with the requirements of this Section.

10.2.2.1 **R and Ω_o Factors.** The response modification coefficient, *R,* in the direction under consideration at any story shall not exceed the lowest response modification coefficient, *R,* for the seismic force-resisting system in the same direction considered above that story excluding penthouses. For other than dual systems, where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of *R* used in that direction shall not be greater than the least value of any of the systems utilized in the same direction. If a system other than a dual system with a response modification coefficient, *R,* with a value of less than 5 is used as part of the seismic force-resisting system in any direction of the structure, the lowest such value shall be used for the entire structure. The system overstrength factor, Ω_o, in the direction under consideration at any story shall not be less than the largest value of this factor for the seismic force-resisting system in the same direction considered above that story.

Exceptions: The limit does not apply to supported structural systems with a weight equal to or less than 10% of the weight of the structure.

10.2.2.2 **Combination Framing Detailing Requirements.** The detailing requirements of Section 10.11 required by the higher response modification coefficient, *R*, shall be used for structural components common to systems having different response modification coefficients.

10.2.3 **Seismic Design Categories B and C. The** structural framing system for structures assigned to Seismic Design Categories B and C shall comply with the structure height and structural limitations in Table 10.2.

TABLE 10.2:
DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC FORCE-RESISTING SYSTEMS

Basic Seismic Force-Resisting System	Response Modification Coefficient, R[a]	System Over-strength Factor, Ω_o [f]	Deflection Amplification Factor, C_d[b]	Structural System Limitations and Building Height (m) Limitations[c] — Seismic Design Category A&B	C	D[d]
Bearing Wall Systems						
Special reinforced concrete shear walls	4	2.5	5	NL	NL	50
Ordinary reinforced concrete shear walls	3	2.5	4	NL	NL	NP
Special reinforced masonry shear walls	4	2.5	3.5	NL	NL	50
Intermediate reinforced masonry shear walls	2.5	2.5	2.25	NL	NL	NP
Ordinary reinforced masonry shear walls	1.5	2.5	1.75	NL	50	NP
Building Frame Systems						
Steel eccentrically braced frames, moment resisting connections at columns away from Links	7	2	4	NL	NL	50
Steel eccentrically braced frames, non-moment resisting connections at columns away from links	6	2	4	NL	NL	50
Special steel concentrically braced frames	5	2	5	NL	NL	50
Ordinary steel concentrically braced frames	4	2	4.5	NL	NL	10[i]
Special reinforced concrete shear walls	5	2.5	5	NL	NL	50
Ordinary reinforced concrete shear walls	4	2.5	4.5	NL	NL	NP
Composite eccentrically braced frames	7	2	4	NL	NL	50
Composite concentrically braced frames	4	2	4.5	NL	NL	50
Ordinary composite braced frames	2.5	2	3	NL	NL	NP
Composite steel plate shear walls	5	2.5	5.5	NL	NL	50
Special composite reinforced concrete shear walls with steel elements	5	2.5	5	NL	NL	50
Ordinary composite reinforced concrete shear walls with steel elements	4	2.5	4.25	NL	NL	NP
Special reinforced masonry shear walls	4	2.5	4	NL	NL	50
Intermediate reinforced masonry shear walls	3	2.5	4	NL	NL	NP
Ordinary reinforced masonry shear walls	2	2.5	2.25	NL	50	NP
Moment Resisting Frame Systems						
Special steel moment frames	7	3	5.5	NL	NL	NL
Intermediate steel moment frames	4	3	4	NL	NL	10g
Ordinary steel moment frames	3	3	3	NL	NL	NPg,h
Special reinforced concrete moment frames	6.5	3	5.5	NL	NL	NL
Intermediate reinforced concrete moment frames	4	3	4.5	NL	NL	NP
Ordinary reinforced concrete moment frames	2.5	3	2.5	NL	NP	NP

TABLE 10.2: DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC FORCE-RESISTING SYSTEMS – continued

Basic Seismic Force-Resisting System	Response Modification Coefficient, R^a	System Over-strength Factor, Ω_o [f]	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (m) Limitations[c] Seismic Design Category A&B	C	D[d]
Dual Systems with Special Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces						
Steel eccentrically braced frames, moment resisting connections, at columns away from links	7	2.5	4	NL	NL	NL
Steel eccentrically braced frames, non-moment resisting connections, at columns away from links	6	2.5	4	NL	NL	NL
Special steel concentrically braced frames	7	2.5	6.5	NL	NL	NL
Special reinforced concrete shear walls	6.5	2.5	6.5	NL	NL	NL
Ordinary reinforced concrete shear walls	5.5	2.5	6	NL	NL	NP
Composite eccentrically braced frames	6.5	2.5	4	NL	NL	NL
Composite concentrically braced frames	5	2.5	5	NL	NL	NL
Composite steel plate shear walls	6.5	2.5	6.5	NL	NL	NL
Special composite reinforced concrete shear walls with steel elements	6.5	2.5	6.5	NL	NL	NL
Ordinary composite reinforced concrete shear walls with steel elements	5.5	2.5	6	NL	NL	NP
Special reinforced masonry shear walls	5.5	3	6.5	NL	NL	NL
Intermediate reinforced masonry shear walls	4.5	2.5	5	NL	NL	NL
Ordinary steel concentrically braced frames	5	2.5	5	NL	NL	NL
Dual Systems with Intermediate Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces						
Special steel concentrically braced frames[e]	4	2.5	4.5	NL	NL	10
Special reinforced concrete shear walls	4.5	2.5	5	NL	NL	50
Ordinary reinforced masonry shear walls	2.5	3	2.5	NL	50	NP
Intermediate reinforced masonry shear walls	4	3	4.5	NL	NL	NP
Composite concentrically braced frames	4	2.5	4.5	NL	NL	50
Ordinary composite braced frames	3.5	2.5	3	NL	NL	NP
Ordinary composite reinforced concrete shear walls with steel elements	4	3	4.5	NL	NL	NP
Ordinary steel concentrically braced frames	4	2.5	4.5	NL	NL	50
Ordinary reinforced concrete shear walls	4.5	2.5	4.5	NL	NL	NP

TABLE 10.2: DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC FORCE-RESISTING SYSTEMS – continued

Basic Seismic Force-Resisting System	Response Modification Coefficient, R^a	System Over-strength Factor, Ω_o [f]	Deflection Amplification Factor, C_d[b]	Structural System Limitations and Building Height (m) Limitations[c] Seismic Design Category		
				A&B	C	D[d]
Inverted Pendulum Systems and Cantilevered Column Systems						
Special steel moment frames	2	2	2.5	NL	NL	NL
Ordinary steel moment frames	1	2	2.5	NL	NL	NP
Special reinforced concrete moment frames	2	2	1.25	NL	NL	NL
Structural Steel Systems Not Specifically Detailed for Seismic Resistance	2.5	3	3	NL	NL	NP

[a] Response modification coefficient, *R,* for use throughout the code. Note *R* reduces forces to a strength level, not an allowable stress level. The given values are approximate and require further study.
[b] Deflection amplification factor, C_d, for use in Sections 10.9.7.1 and 10.9.7.2
[c] NL = Not Limited and NP = Not Permitted. Heights are measured from the base of the structure as defined in Section 9.2.
[d] See Section 10.2.4.1 for a description of building systems limited to buildings with a height of 75 m or less.
[e] Ordinary moment frame is permitted to be used in lieu of intermediate moment frame in Seismic Design Categories B and C.
[f] The tabulated value of the overstrength factor, Ω_o may be reduced by subtracting 0.5 for structures with flexible diaphragms but shall not be taken as less than 2.0 for any structure.
[g] Steel ordinary moment frames and intermediate moment frames are permitted in single-story buildings up to a height of 18 m, when the moment joints of field connections are constructed of bolted end plates and the dead load of the roof does not exceed 0.75 kPa.
[h] Steel ordinary moment frames are permitted in buildings up to a height of 10 m where the dead load of the walls, floors, and roofs does not exceed 0.75 kPa.
[i] Steel ordinary concentrically braced frames are permitted in single-story buildings up to a height of 18 m when the dead load of the roof does not exceed 0.75 kPa and in penthouse structures.

10.2.4 **Seismic Design Category D.** The structural framing system for a structure assigned to Seismic Design Category D shall comply with Section 10.2.3 and the additional provisions of this Section.

10.2.4.1 **Interaction Effects.** Moment resisting frames that are enclosed or adjoined by more rigid elements not considered to be part of the seismic force-resisting system shall be designed so that the action or failure of those elements will not impair the vertical load and seismic force-resisting capability of the frame. The design shall provide for the effect of these rigid elements on the structural system at structure deformations corresponding to the design story drift (Δ) as determined in Section 10.9.7. In addition, the effects of these elements shall be considered when determining whether a structure has one or more of the irregularities defined in Section 10.3.

10.2.4.2 **Deformational Compatibility.** Every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the vertical load-carrying capacity and the induced moments and shears resulting from the design story drift (Δ) as determined in accordance with Section 10.9.7; see also Section 10.12.

Exception: Reinforced concrete frame members not designed as part of the seismic force-resisting system shall comply with Section 21.9 of SBC 304.

When determining the moments and shears induced in components that are not included in the seismic force-resisting system in the direction under consideration, the stiffening effects of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used.

10.2.4.3 **Special Moment Frames**. A special moment frame that is used but not required by Table 10.2 shall not be discontinued and supported by a more rigid system with a lower response modification coefficient (R) unless the requirements of Sections 10.11.2.4 and 10.11.4.2 are met. Where a special moment frame is required by Table 10.2, the frame shall be continuous to the foundation.

SECTION 10.3
DIAPHRAGM FLEXIBILITY, CONFIGURATION IRREGULARITIES AND REDUNDANCY

10.3.1 **Diaphragm Flexibility.** The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic force-resisting system. Unless a diaphragm can be idealized as either flexible or rigid in accordance with Sections 10.3.1.1, 10.3.1.2 or 10.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e. semi-rigid modeling assumption).

10.3.1.1 **Flexible Diaphragm Condition.** Diaphragms constructed of untopped steel decking shall be permitted to be idealized as flexible in structures in which the vertical elements are steel or composite braced frames, or concrete, masonry, steel or composite shear walls. Diaphragms of untopped steel decks in one- and two-family residential buildings shall also be permitted to be idealized as flexible.

10.3.1.2 **Rigid Diaphragm Condition.** Diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities shall be permitted to be idealized as rigid.

10.3.1.3 **Calculated Flexible Diaphragm Condition.** Diaphragms not satisfying the conditions of Sections 10.3.1.1 or 10.3.1.2 shall be permitted to be idealized as flexible when the computed maximum in-plane deflection of the diaphragm itself under lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system of the associated story under equivalent tributary lateral load as shown in Figure 10.3-1. The loadings used for this calculation shall be those prescribed by Section 10.9.

10.3.2 **Irregular and Regular Classification.** Structures shall be classified as regular or irregular based on the criteria in this Section. Such classification shall be based on the plan and vertical configuration.

10.3.2.1 **Plan Irregularity.** Structures having one or more of the irregularity types listed in Table 10.3.2.1 shall be designated as having plan structural irregularity. Such structures assigned to the Seismic Design Categories listed in Table 10.3.2.1 shall comply with the requirements in the sections referenced in that table.

10.3.2.2 **Vertical Irregularity.** Structures having one or more of the irregularity types listed in Table 10.3.2.2 shall be designated as having vertical irregularity. Such structures assigned to the Seismic Design Categories listed in Table 10.3.2.2 shall comply with the requirements in the sections referenced in that table.

Exception

1. Vertical structural irregularities of Types la, lb, or 2 in Table 10.3.2.2 do not apply where no story drift ratio under design lateral seismic force is greater than 130% of the story drift ratio of the next story above. Torsional effects need not be considered in the calculation of story drifts. The story drift ratio relationship for the top 2 stories of the structure are not required to be evaluated.
2. Irregularities Types la, lb, and 2 of Table 10.3.2.2 are not required to be considered for 1- or 2-story buildings in Seismic Design Categories A, B, C, or D.

10.3.3 **Redundancy.** A reliability factor, ρ, shall be assigned to all structures in accordance with this Section, based on the extent of structural redundancy inherent in the lateral force-resisting system.

10.3.3.1 **Seismic Design Categories A, B, and C.** For structures in Seismic Design Categories A, B, and C, the value of ρ is 1.0.

10.3.3.2 **Seismic Design Category D.** For structures in Seismic Design Category D, the value of ρ is 1.25.

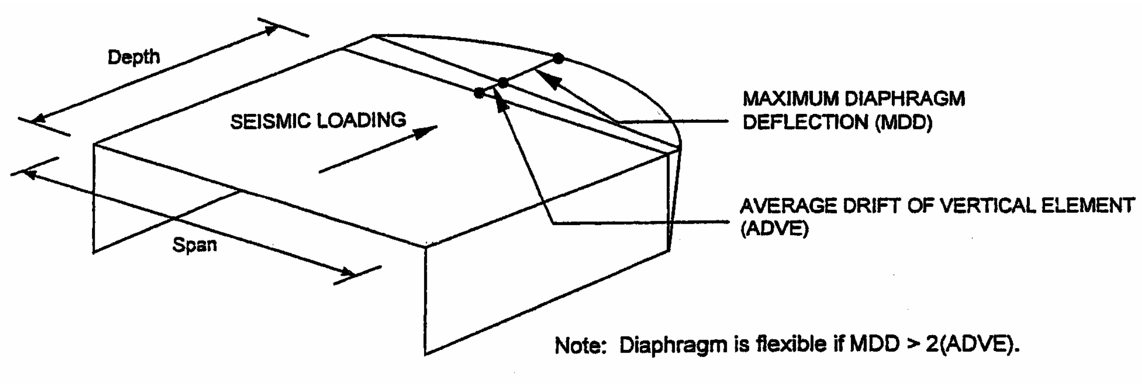


Figure 10.3-1: Flexible Diaphragm.

TABLE 10.3.2.1:
PLAN STRUCTURAL IRREGULARITIES

Irregularity Type and Description		Reference Section	Seismic Design Category Application
1a.	Torsional Irregularity Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	10.11.4.2 10.9.5.2	D C, D
1b.	Extreme Torsional Irregularity Extreme Torsional Irregularity is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	10.11.4.2 10.9.5.2	D C and D
2.	Re-entrant Corners Plan configurations of a structure and its lateral force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15% of the plan dimension of the structure in the given direction.	10.11.4.2	D
3.	Diaphragm Discontinuity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one-story to the next.	10.11.4.2	D
4.	Out-of-Plane Offsets Discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	10.11.4.2 10.11.2.11	D, B, C, D
5.	Nonparallel Systems The vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.	10.11.3.1	C, D

TABLE 10.3.2.2: VERTICAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	Stiffness Irregularity: Soft Story A soft story is one in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	10.6.1	D
1b.	Stiffness Irregularity: Extreme Soft Story An extreme soft story is one in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	10.6.1	D
2.	Weight (Mass) Irregularity Mass irregularity shall be considered to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	10.6.1	D
3.	Vertical Geometric Irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130% of that in an adjacent story.	10.6.1	D
4.	In-Plane Discontinuity in Vertical Lateral Force-Resisting Elements In-plane discontinuity in vertical lateral force-resisting elements shall be considered to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.	10.6.1 10.11.2.11	B, C, D
5.	Discontinuity in Lateral Strength: Weak Story A weak story is one in which the story lateral strength is less than 80% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	10.11.2.2 10.6.1	B, C, D D

SECTION 10.4
SEISMIC LOAD EFFECTS AND COMBINATIONS

The effects on the structure and its components due to seismic forces shall be combined with the effects of other loads in accordance with the combinations of load effects given in Chapter 2. For use with those combinations, the earthquake-induced force effect shall include vertical and horizontal effects as given by Eq. 10.4-1 or 10.4-2, as applicable. The vertical seismic effect term $0.2S_{DS}D$ need not be included where S_{DS} is equal to or less than 0.125 in Eqs. 10.4-1, 10.4-2, 10.4.1-1, and 10.4.1-2. The vertical seismic effect term $0.2S_{DS}D$ need not be included in Eq. 10.4-2 when considering foundation overturning.

For Eq. 2.3.2-5 in Section 2.3.2 or Eq. 2.4.1-5 and Eq. 2.4.1-6 in Section 2.4.1:

$$E = \rho Q_E + 0.2S_{DS}D \quad \textbf{(Eq. 10.4-1)}$$

For Eq. 2.3.2-7 in Section 2.3.2 or Eq. 2.4.1-8 in Section 2.4.1:

$$E = \rho\, Q_E - 0.2 S_{DS} D \qquad \textbf{(Eq. 10.4-2)}$$

Where

E	=	the effect of horizontal and vertical earthquake-induced forces
S_{DS}	=	the design spectral response acceleration at short periods obtained from Section 9.4.4
D	=	the effect of dead load, D
Q_E	=	the effect of horizontal seismic (earthquake-induced) forces
ρ	=	the reliability factor as per Section 10.3.3

10.4.1 **Special Seismic Load.** Where specifically indicated in SBC 301, the special seismic load of Eq. 10.4.1-1 shall be used to compute E for use in Eq. 2.3.2-5 in Section 2.3.2 or Eq. 2.4.1-5 and Eq. 2.4.1-6 in Section 2.4.1 and the special seismic load of Eq. 10.4.1-2 shall be used to compute E in Eq. 2.3.2-7 in Section 2.3.2 or Eq. 2.4.1-8 in Section 2.4.1:

$$E = \Omega_o Q_E + 0.2 S_{DS} D \qquad \textbf{(Eq. 10.4.1-1)}$$

$$E = \Omega_o Q_E - 0.2 S_{DS} D \qquad \textbf{(Eq. 10.4.1-2)}$$

Where

Ω_o = over strength factor as defined in Table 10.2.

The value of the quantity $\Omega_o Q_E$ in Eqs. 10.4.1-1 and 10.4.1-2 need not be taken greater than the capacity of other elements of the structure to transfer force to the component under consideration.

Where allowable stress design methodologies are used with the special load of this Section applied in Eq. 2.4.1-5 or Eq. 2.4.1-6 of Section 2.4.1, allowable stresses are permitted to be determined using an allowable stress increase of 1.2. This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted by SBC 301 or the material reference standard.

SECTION 10.5
DIRECTION OF LOADING

The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. It shall be permitted to satisfy this requirement using the procedures of Section 10.5.1 for Seismic Design Category A and B, Section 10.5.2 for Seismic Design Category C, and Section 10.5.3 for Seismic Design Category D. All structural components and their connections shall be provided with strengths sufficient to resist the effects of the seismic forces prescribed herein. Loads shall be combined as prescribed in Section 10.4.

10.5.1 **Seismic Design Categories A and B.** For structures assigned to Seismic Design Category A and B, the design seismic forces are permitted to be applied separately in each of two orthogonal directions and orthogonal interaction effects may be neglected.

10.5.2 **Seismic Design Category C.** Loading applied to structures assigned to Seismic Design Category C shall, as a minimum, conform to the requirements of Section 10.5.1 for Seismic Design Categories A and B and the requirements of this Section. Structures that have plan structural irregularity Type 5 in Table 10.3.2.1 shall be analyzed for seismic forces using a three-dimensional representation and the following procedure:

a. The structure shall be analyzed using the equivalent lateral force analysis procedure of Section 10.9 or the modal response spectrum analysis procedure of Section 10.10, as permitted under Section 10.6.1, with the loading applied independently in any two orthogonal directions and the most critical load effect due to direction of application of seismic forces on the structure may be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100% of the forces for one direction plus 30% of the forces for the perpendicular direction; the combination requiring the maximum component strength shall be used.

10.5.3 **Seismic Design Category D**. Structures assigned to Seismic Design Category D shall, as a minimum, conform to the requirements of Section 10.5.2 In addition, any column or wall that forms part of two or more intersecting seismic force-resisting systems and is subjected to axial load due to seismic forces acting along either principal plan axis equaling or exceeding 20% of the axial design strength of the column or wall shall be designed for the most critical load effect due to application of seismic forces in any direction. The procedure of Section 10.5.2a shall be permitted to be used to satisfy this requirement. Two-dimensional analyses shall be permitted for structures with flexible diaphragms.

SECTION 10.6
ANALYSIS PROCEDURES

A structural analysis conforming to one of the types permitted in Section 10.6.1 shall be made for all structures. Application of loading shall be as indicated in Section 10.5, and as required by the selected analysis procedure. All members of the structure's seismic force-resisting system and their connections shall have adequate strength to resist the forces, Q_E, predicted by the analysis, in combination with other loads, as required by Section 10.4. Drifts predicted by the analysis shall be within the limits specified by Section 10.12.

Exception: For structures designed using the index force analysis procedure of Section 10.7 or the simplified analysis procedure of Section 10.8, drift need not be evaluated.

10.6.1 **Analysis Procedures.** The structural analysis required by Section 10.6 shall consist of one of the types permitted in Table 10.6.1, based on the structure's Seismic Design Category, structural system, dynamic properties and regularity, or as per Section 10.14.

SECTION 10.7
INDEX FORCE ANALYSIS PROCEDURE FOR SEISMIC DESIGN OF BUILDINGS

See Section 10.6.1 for limitations on the use of this procedure. An index force analysis shall consist of the application of static lateral index forces to a linear mathematical model of the structure, independently in each of two orthogonal directions. The lateral index forces shall be as given by Eq. 10.7-1 and shall be applied simultaneously at each floor level. For purposes of analysis, the structure shall be considered to be fixed at the base:

$$F_x = 0.01\,\omega_x \qquad \textbf{(Eq. 10.7-1)}$$

where

F_x = the design lateral force applied at story x

ω_x = the portion of the total gravity load of the structure, W, located or assigned to Level x

W = the effective seismic weight of the structure, including the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25% of the floor live load (floor live load in public garages and open parking structures need not be included.)
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 0.5 kN/m^2 of floor area, whichever is greater.
3. Total operating weight of permanent equipment.

TABLE 10.6.1:
PERMITTED ANALYTICAL PROCEDURES

Seismic Design Category	Structural Characteristics	Index Force Analysis Section 10.7	Simplified Analysis Section 10.8	Equivalent Lateral Force Analysis Section 10.9	Modal Response Spectrum Analysis Section 10.10
A	Regular or irregular	P	P	P	P
B, C	Regular or irregular	NP	NP	P	P
D	Regular structures with $T < 3.5\ T_s$	NP	NP	P	P
	Irregular structures with $T < 3.5\ T_s$ and having only plan irregularities type 2, 3, 4, or 5 of Table 10.3.2.1 or vertical irregularities type 4 or 5 of Table 10.3.2.2	NP	NP	P	P
	All other structures	NP	NP	NP	P

Notes: P - indicates permitted, NP -indicates not permitted

SECTION 10.8
SIMPLIFIED ANALYSIS PROCEDURE FOR SEISMIC DESIGN OF BUILDINGS

For purposes of this analysis procedure, a building is considered to be fixed at the base. See Section 10.6 for limitations on the use of this procedure.

10.8.1 **Seismic Base Shear.** The seismic base shear, V, in a given direction shall be determined in accordance with the following formula:

$$V = \frac{1.2\, S_{DS}}{R} W \qquad \textbf{(Eq. 10.8.1)}$$

where

S_{DS} = the design elastic response acceleration at short period as determined in accordance with Section 9.4.4

R = the response modification factor from Table 10.2

W = the effective seismic weight of the structure as defined in Section 10.7

10.8.2 **Vertical Distribution.** The forces at each level shall be calculated using the following formula:

$$F_x = \frac{1.2\, S_{DS}}{R} \omega_x \qquad \textbf{(Eq. 10.8.2)}$$

where

ω_x = the portion of the effective seismic weight of the structure, W, at level x

10.8.3 **Horizontal Distribution.** Diaphragms constructed of untopped steel decking are permitted to be considered as flexible.

10.8.4 **Design Drift.** For the purposes of Section 10.12, the design story drift, Δ, shall be taken as 1% of the story height unless a more exact analysis is provided.

SECTION 10.9
EQUIVALENT LATERAL FORCE PROCEDURE

10.9.1 **General.** Section 10.9 provides required minimum standards for the equivalent lateral force procedure of seismic analysis of structures. An equivalent lateral force analysis shall consist of the application of equivalent static lateral forces to a linear mathematical model of the structure. The directions of application of lateral forces shall be as indicated in Section 10.5. The lateral forces applied in each direction shall be the total seismic base shear given by Section 10.9.2 and shall be distributed vertically in accordance with the provisions of Section 10.9.4. For purposes of analysis, the structure is considered to be fixed at the base. See Section 10.6 for limitations on the use of this procedure.

10.9.2 **Seismic Base Shear.** The seismic base shear (V) in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \qquad \textbf{(Eq. 10.9.2-1)}$$

where

C_s = the seismic response coefficient determined in accordance with Section 10.9.2.1.

W = the total dead load and applicable portions of other loads as indicated in Section 10.7

10.9.2.1 Calculation of Seismic Response Coefficient. When the fundamental period of the structure is computed, the seismic design coefficient (C_S) shall be determined in accordance with the following equation:

$$C_S = \frac{S_{DS}}{R/I} \qquad \textbf{(Eq. 10.9.2.1-1)}$$

where

S_{DS} = the design spectral response acceleration in the short period range as determined from Section 9.4.4

R = the response modification factor in Table 10.2

I = the occupancy importance factor determined in accordance with Section 9.5

The value of the seismic response coefficient, (Cs), need not be greater than the following equation:

$$C_S = \frac{S_{D1}}{T\left(R/I\right)} \qquad \textbf{(Eq. 10.9.2.1-2)}$$

but shall not be taken less than

$$C_s = 0.044\, S_{DS} I \qquad \textbf{(Eq. 10.9.2.1-3)}$$

where I and R are defined above and

S_{D1} = the design spectral response acceleration at a period of 1.0 sec, in units of g-sec, as determined from Section 9.4.4

T = the fundamental period of the structure (sec) as determined in Section 10.9.3

For regular structures 5 stories or less in height and having a period, T, of 0.5 sec or less, the seismic response coefficient, C_S shall be permitted to be calculated using values of 1.5 g and 0.6 g, respectively, for the mapped maximum considered earthquake spectral response accelerations S_S and S_1.

10.9.3 Period Determination. The fundamental period of the structure (T) in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period (T) shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 10.9.3.1 and the approximate fundamental period (T_a) determined from Eq. 10.9.3.2-1. As an alternative to performing an analysis to determine the fundamental period (T), it shall be permitted to use the approximate building period, (T_a), calculated in accordance with Section 10.9.3.2, directly.

10.9.3.1 Upper Limit on Calculated Period. The fundamental building period (T) determined in a properly substantiated analysis shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 10.9.3.1 and the approximate fundamental period *(T_a)* determined in accordance with Section 10.9.3.2.

TABLE 10.9.3.1: COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD

Design Spectral Response Acceleration at 1 Second, S_{D1}	Coefficient C_u
$\geq$ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
0.1	1.7
$\leq$ 0.05	1.7

10.9.3.2 **Approximate Fundamental Period.** The approximate fundamental period *(T_a)*, in seconds, shall be determined from the following equation:

$$T_a = C_t h_n^{\,x} \qquad \textbf{(Eq. 10.9.3.2-1)}$$

where h_n is the height in (m) above the base to the highest level of the structure and the coefficients C_t and x are determined from Table 10.9.3.2.

Alternatively, it shall be permitted to determine the approximate fundamental period *(T_a)*, in seconds, from the following equation for structures not exceeding 12 stories in height in which the lateral-force-resisting system consists entirely of concrete or steel moment resisting frames and the story height is at least 3 m:

$$T_a = 0.1N \qquad \textbf{(Eq. 10.9.3.2-la)}$$

where N = number of stories

The approximate fundamental period, T_a, in seconds for masonry or concrete shear wall structures shall be permitted to be determined from Eq. 10.9.3.2-2 as follows:

$$T_a = \frac{0.0062}{\sqrt{C_w}} h_n \qquad \textbf{(Eq. 10.9.3.2-2)}$$

where h_n is as defined above and C_w, is calculated from Eq. 10.9.3.2-3 as follows:

$$C_w = \frac{100}{A_B} \sum_{i=1}^{n} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \qquad \textbf{(Eq. 10.9.3.2-3)}$$

where

A_B = the base area of the structure m^2

A_i = the area of shear wall "i" in m^2

D_i = the length of shear wall "i" in m

n = the number of shear walls in the building effective in resisting lateral forces in the direction under consideration

TABLE 10.9.3.2:
VALUES OF APPROXIMATE PERIOD PARAMETERS C_t AND *x*

Structure Type	C_t	*x*
Moment resisting frame systems of steel in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces	0.068	0.8
Moment resisting frame systems of reinforced concrete in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frame from deflecting when subjected to seismic forces	0.044	0.9
Eccentrically braced steel frames	0.07	0.75
All other structural systems	0.055	0.75

10.9.4 Vertical Distribution of Seismic Forces. The lateral seismic force (F_x) (kN) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \qquad \textbf{(Eq. 10.9.4-1)}$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \textbf{(Eq. 10.9.4-2)}$$

where

C_{vx} = vertical distribution factor

V = total design lateral force or shear at the base of the structure, (kN)

w_i and w_x = the portion of the total gravity load of the structure (W) located or assigned to Level i or x

h_i and h_x = the height (m) from the base to Level i or x

k = an exponent related to the structure period as follows:

for structures having a period of 0.5 sec or less, $k = 1$

for structures having a period of 2.5 sec or more, $k = 2$

for structures having a period between 0.5 and 2.5 seconds, k shall be 2 or shall be determined by linear interpolation between 1 and 2

10.9.5 Horizontal Shear Distribution and Torsion. The seismic design story shear in any story (V_x) (kN) shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \qquad \textbf{(Eq. 10.9.5)}$$

where F_i = the portion of the seismic base shear (V) (kN) induced at Level i.

10.9.5.1 Direct Shear. The seismic design story shear (V_x) (kN) shall be distributed to the various vertical elements of the seismic force-resisting system in the story under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

10.9.5.2 **Torsion.** Where diaphragms are not flexible, the design shall include the torsional moment *(M_t)* (kN·m) resulting from the location of the structure masses plus the accidental torsional moments *(M_{ta})* (kN·m) caused by assumed displacement of the mass each way from its actual location by a distance equal to 5% of the dimension of the structure perpendicular to the direction of the applied forces. Where earthquake forces are applied concurrently in two orthogonal directions, the required 5% displacement of the center of mass need not be applied in both of the orthogonal directions at the same time, but shall be applied in the direction that produces the greater effect.

Structures of Seismic Design Categories C and D where Type 1 torsional irregularity exists as defined in Table 10.3.2.1 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\,\delta_{avg}}\right)^2 \qquad \textbf{(Eq. 10.9.5.2)}$$

where

δ_{max} = the maximum displacement at Level x (mm)

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x (mm)

The torsional amplification factor (A_x) is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

10.9.6 **Overturning.** The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 10.9.4. At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical elements of the lateral force-resisting system in the same proportion as the distribution of the horizontal shears to those elements.
The overturning moments at Level x *(M_x)* (kN·m) shall be determined from the following equation:

$$M_x = \sum_{i=x}^{n} F_i\left(h_i - h_x\right) \qquad \textbf{(Eq. 10.9.6)}$$

where

F_i = the portion of the seismic base shear (V) induced at Level i

h_i and h_x = the height (m) from the base to Level i or x

The foundations of structures, except inverted pendulum-type structures, shall be permitted to be designed for 75% of the foundation overturning design moment (M_f) (kN·m) at the foundation-soil interface determined using the equation for the overturning moment at Level x *(M_x)* (kN·m).

10.9.7 **Drift Determination and P-Delta Effects.** Story drifts and, where required, member forces and moments due to P-delta effects shall be determined in accordance with this Section. Determination of story drifts shall be based on the application of the design seismic forces to a mathematical model of the physical structure. The model shall include the stiffness and strength of all elements that are significant to the distribution of forces and deformations in the structure and shall represent the spatial distribution of the mass and stiffness of the structure. In addition, the model shall comply with the following:

1. Stiffness properties of reinforced concrete and masonry elements shall consider the effects of cracked sections, and
2. For steel moment resisting frame systems, the contribution of panel zone deformations to overall story drift shall be included.

10.9.7.1 Story Drift Determination. The design story drift (Δ) shall be computed as the difference of the deflections at the top and bottom of the story under consideration. Where allowable stress design is used, Δ shall be computed using code-specified earthquake forces without reduction.

Exception: For structures of Seismic Design Categories C and D having plan irregularity Types la or 1b of Table 10.3.2.1, the design story drift, D, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

The deflections of Level x at the center of the mass (δ_x) (mm) shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad \textbf{(Eq. 10.9.7.1)}$$

where

C_d = the deflection amplification factor in Table 10.2

δ_{xe} = the deflections determined by an elastic analysis

I = the importance factor determined in accordance with Section 9.5

The elastic analysis of the seismic force-resisting system shall be made using the prescribed seismic design forces of Section 9.5.5.4. For the purpose of this Section, the value of the base shear, V, used in Eq. 10.9.2-1 need not be limited by the value obtained from Eq. 10.9.2.1-3.

For determining compliance with the story drift limitation of Section 10.12, the deflections at the center of mass of Level x (δ_x) (mm) shall be calculated as required in this Section. For the purposes of this drift analysis only, the upper-bound limitation specified in Section 9.5.5.3 on the computed fundamental period, T, in seconds, of the building does not apply for computing forces and displacements.

Where applicable, the design story drift (Δ) (mm) shall be increased by the incremental factor relating to the P-delta effects as determined in Section 10.9.7.2.

When calculating drift, the redundancy coefficient, ρ, is not used.

10.9.7.2 P-Delta Effects. P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered when the stability coefficient (θ) as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \textbf{(Eq. 10.9.7.2-1)}$$

where

P_x = the total vertical design load at and above Level x. (kN); when computing P_x, no individual load factor need exceed 1.0

Δ = the design story drift as defined in Section 10.9.7.1 occurring simultaneously with V_x, (mm)

V_x = the seismic shear force acting between Levels x and x - 1, (kN)

h_{sx} = the story height below Level x, (mm)

C_d = the deflection amplification factor in Table 10.2

The stability coefficient (θ) shall not exceed θ_{max} determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad \textbf{(Eq. 10.9.7.2-2)}$$

where β is the ratio of shear demand to shear capacity for the story between Level x and x - 1. This ratio may be conservatively taken as 1.0.

When the stability coefficient (θ) is greater than 0.10 but less than or equal to θ_{max} the incremental factor related to P-delta effects (a_d) shall be determined by rational analysis. To obtain the story drift for including the P-delta effect, the design story drift determined in Section 10.9.7.1 shall be multiplied by 1.0/(1 - θ).

When θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

When the P-delta effect is included in an automated analysis, Eq. 10.9.7.2-2 must still be satisfied, however, the value of θ computed from Eq. 10.9.7.2-1 using the results of the P-delta analysis may be divided by $(1 + \theta)$ before checking Eq. 10.9.7.2-2.

SECTION 10.10
MODAL ANALYSIS PROCEDURE

10.10.1 General. Section 10.10 provides required standards for the modal analysis procedure of seismic analysis of structures. See Section 10.6 for requirements for use of this procedure. The symbols used in this method of analysis have the same meaning as those for similar terms used in Section 10.7, with the subscript m denoting quantities in the m^{th} mode.

10.10.2 Modeling. A mathematical model of the structure shall be constructed that represents the spatial distribution of mass and stiffness throughout the structure.

For regular structures with independent orthogonal seismic force-resisting systems, independent two-dimensional models are permitted to be constructed to represent each system. For irregular structures or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the structure. Where the diaphragms are not rigid compared to the vertical elements of the lateral force-resisting system, the model should include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response. In addition, the model shall comply with the following:

1. Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections, and
2. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

10.10.3 **Modes.** An analysis shall be conducted to determine the natural modes of vibration for the structure, including the period of each mode, the modal shape vector ϕ, the modal participation factor, and modal mass. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90% of the actual mass in each of two orthogonal directions.

10.10.4 **Periods.** The required periods, mode shapes, and participation factors of the structure in the direction under consideration shall be calculated by established methods of structural analysis for the fixed-base condition using the masses and elastic stiffnesses of the seismic force-resisting system.

10.10.5 **Modal Base Shear.** The portion of the base shear contributed by the m^{th} mode (V_m) shall be determined from the following equations:

$$V_m = C_{sm} W_m \qquad \textbf{Eq. (10.10.5-1)}$$

$$W_m = \frac{\left(\sum_{i=1}^{n} w_i \phi_{im}\right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \qquad \textbf{Eq. (10.10.5-2)}$$

where

C_{sm} = the modal seismic design coefficient determined below

W_m = the effective modal gravity load

w_i = the portion of the total gravity load of the structure at Level i

ϕ_{im} = the displacement amplitude at the i^{th} level of the structure when vibrating in its m^{th} mode

The modal seismic design coefficient (C_{sm}) shall be determined in accordance with the following equation:

$$C_{sm} = \frac{S_{am}}{R/I} \qquad \textbf{(Eq. 10.10.5-3)}$$

where

S_{am} = the design spectral response acceleration at period T_m determined from the general design response spectrum of Section 9.4.5.

R = the response modification factor determined from Table 10.2.

I = the occupancy importance factor determined in accordance with Section 9.5.

T_m = the modal period of vibration (in seconds) of the mth mode of the structure

Exception: When the general design response spectrum of Section 9.4.5 is used for structures where any modal period of vibration (T_m) exceeds 4.0 sec, the modal seismic design coefficient (C_{sm}) for that mode shall be determined by the following equation:

$$C_{sm} = \frac{4S_{D1}}{(R/I)T_m^2} \qquad \textbf{(Eq. 10.10.5-4)}$$

10.10.6 Modal Forces, Deflections, and Drifts. The modal force (F_{xm}) at each level shall be determined by the following equations:

$$F_{xm} = C_{vxm} V_m \qquad \textbf{(Eq. 10.10.6-1)}$$

and

$$C_{vxm} = \frac{w_x \phi_{xm}}{\sum_{i=1}^{n} w_i \phi_{im}} \qquad \textbf{(Eq. 10.10.6-2)}$$

where

C_{vxm} = the vertical distribution factor in the m^{th} mode

V_m = the total design lateral force or shear at the base in the m^{th} mode

w_i and w_x = the portion of the total gravity load of the structure (W) located or assigned to Level i or x

ϕ_{xm} = the displacement amplitude at the x^{th} level of the structure when vibrating in its m^{th} mode

ϕ_{im} = the displacement amplitude at the i^{th} level of the structure when vibrating in its m^{th} mode

The modal deflection at each level (δ_{xm}) shall be determined by the following equations:

$$\delta_{xm} = \frac{C_d \delta_{xem}}{I} \qquad \textbf{(Eq. 10.10.6-3)}$$

and

$$\delta_{xem} = \left(\frac{g}{4\pi^2}\right)\left(\frac{T_m^2 F_{xm}}{w_x}\right) \qquad \textbf{(Eq. 10.10.6-4)}$$

where

C_d = the deflection amplification factor determined from Table 10.2

δ_{xem} = the deflection of Level x in the m^{th} mode at the center of the mass at Level x determined by an elastic analysis

g = the acceleration due to gravity (m^2/sec)

I = the occupancy importance factor deter-mined in accordance with Section 9.5

T_m = the modal period of vibration, in seconds, of the m^{th} mode of the structure

F_{xm} = the portion of the seismic base shear in the m^{th} mode, induced at Level x, and

w_x = the portion of the total gravity load of the structure *(W)* located or assigned to Level x

The modal drift in a story (Δ_m) shall be computed as the difference of the deflections (δ_{xm}) at the top and bottom of the story under consideration.

10.10.7 **Modal Story Shears and Moments.** The story shears, story overturning moments, and the shear forces and overturning moments in vertical elements of the structural system at each level due to the seismic forces determined from the appropriate equation in Section 10.10.6 shall be computed for each mode by linear static methods.

10.10.8 **Design Values.** The design value for the modal base shear (V_t), each of the story shear, moment and drift quantities, and the deflection at each level shall be determined by combining their modal values as obtained from Sections 10.10.6 and 10.10.7. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values or where closely spaced periods in the translational and torsional modes result in significant cross-correlation of the modes, the complete quadratic combination (CQC) method, shall be used.

A base shear (V) shall be calculated using the equivalent lateral force procedure in Section 10.9. For the purpose of this calculation, a fundamental period of the structure (T), in seconds, shall not exceed the coefficient for upper limit on the calculated period (C_u) times the approximate fundamental period of the structure (T_a). Where the design value for the modal base shear (V_t) is less than 85% of the calculated base shear (V) using the equivalent lateral force procedure, the design story shears, moments, drifts, and floor deflections shall be multiplied by the following modification factor:

$$0.85\,\frac{V}{V_t} \qquad \textbf{(Eq. 10.10.8)}$$

where

V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 10.9, and

V_t = the modal base shear, calculated in accordance with this section

10.10.9 **Horizontal Shear Distribution.** The distribution of horizontal shear shall be in accordance with the requirements of Section 10.9.5 except that amplification of torsion per Section 10.9.5.2 is not required for that portion of the torsion, A_x, included in the dynamic analysis model.

10.10.10 **Foundation Overturning.** The foundation overturning moment at the foundation-soil interface may be reduced by 10%.

10.10.11 **P-Delta Effects.** The P-delta effects shall be determined in accordance with Section 10.9.7. The story drifts and base shear used to determine the story shears shall be determined in accordance with Section 10.9.7.1.

SECTION 10.11
DESIGN AND DETAILING REQUIREMENTS

The design and detailing of the components of the seismic force-resisting system shall comply with the requirements of this Section. Foundation design shall conform to the applicable requirements of Section 10.13. The materials and the systems composed of those materials shall conform to the requirements and limitations of Sections 11.1 through 11.4 for the applicable category.

10.11.1 **Seismic Design Category A**. The design and detailing of structures assigned to Category A shall comply with the requirements of this Section.

10.11.1.1 **Load Path Connections.** All parts of the structure between separation joints shall be interconnected to form a continuous path to the seismic force-resisting system, and the connections shall be capable of transmitting the seismic force (F_p) induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having a design strength capable of transmitting a seismic force of 0.133 times the short period design spectral response acceleration coefficient, S_{DS}, times the weight of the smaller portion or 5% of the portion's weight, whichever is greater. This connection force does not apply to the overall design of the lateral-force-resisting system. Connection design forces need not exceed the maxi-mum forces that the structural system can deliver to the connection.

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss to its support. The connection shall have a minimum strength of 5% of the dead plus live load reaction. One means to provide the strength is to use connecting elements such as slabs.

10.11.1.2 **Anchorage of Concrete or Masonry Walls.** Concrete and masonry walls shall be anchored to the roof and all floors and members that provide lateral support for the wall or which are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting the horizontal forces specified in Section 10.11.1.1 but not less than a minimum strength level, horizontal force of 4 kN/m of wall substituted for E in the load combinations.

10.11.2 **Seismic Design Category B.** Structures assigned to Seismic Design Category B shall conform to the requirements of Section 10.11.1 for Seismic Design Category A and the requirements of this Section.

10.11.2.1 **P-Delta Effects.** P-delta effects shall be included where required by Section 10.9.7.2.

10.11.2.2 **Openings.** Where openings occur in shear walls, diaphragms, or other plate-type elements, reinforcement at the edges of the openings shall be designed to transfer the stresses into the structure. The edge reinforcement shall extend into the body of the wall or diaphragm a distance sufficient to develop the force in the reinforcement. The extension must be sufficient in length to allow dissipation or transfer of the force without exceeding the shear and tension capacity of the diaphragm or the wall.

10.11.2.3 **Direction of Seismic Load.** The direction of application of seismic forces used in design shall be that which will produce the most critical load effect in each component. This requirement will be deemed satisfied if the design seismic forces are applied separately and independently in each of two orthogonal directions.

10.11.2.4 **Discontinuities in Vertical System.** Structures with a discontinuity in lateral capacity, vertical irregularity Type 5 as defined in Table 10.3.2.2, shall not be more than 2 stories or 9 m in height where the "weak" story has a calculated strength of less than 65% of the story above.

Exception: The limit does not apply where the "weak" story is capable of resisting a total seismic force equal to Ω_o times the design force prescribed in Section 10.7.

10.11.2.5 **Nonredundant Systems.** The design of a structure shall consider the potentially adverse effect that the failure of a single member, connection, or component of the seismic force-resisting system will have on the stability of the structure; see Section 1.4.

10.11.2.6 **Collector Elements.** Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces.

10.11.2.7 **Diaphragms.** The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

Floor and roof diaphragms shall be designed to resist F_p where F_p is the larger of:

1. The portion of the design seismic force at the level of the diaphragm that depends on the diaphragm for transmission to the vertical elements of the seismic force-resisting system, or
2. $F_p = 0.2S_{DS} I w_p + V_{px}$, **(Eq. 10.11.2.7)**

where

F_p = the seismic force induced by the parts

I = occupancy importance factor (Table 9.5)

S_{DS} = the short period site design spectral response acceleration coefficient, Section 9.4.1

w_p = the weight of the diaphragm and other elements of the structure attached to it

V_{px} = the portion of the seismic shear force at the level of the diaphragm, required to be transferred to the components of the vertical seismic force-resisting system because of the offsets or changes in stiffness of the vertical components above or below the diaphragm.

Diaphragms shall be designed for both the shear and bending stresses resulting from these forces. Diaphragms shall have ties or struts to distribute the wall anchorage forces into the diaphragm. Diaphragm connections shall be positive, mechanical, or welded type connections.

At diaphragm discontinuities, such as openings and re-entrant corners, the design shall ensure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

10.11.2.8 **Anchorage of Concrete or Masonry Walls.** Exterior and interior bearing walls and their anchorage shall be designed for a force normal to the surface equal to 40% of the short period design spectral response acceleration, S_{DS}, times the occupancy importance factor, I, multiplied by the weight of wall (W_c) associated with the anchor, with a minimum force of 10% of the weight of the wall. Interconnection of wall elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces. The connections shall also satisfy Section 10.11.1.2.

The anchorage of concrete or masonry walls to supporting construction shall provide a direct connection capable of resisting the greater of the force $0.4\ S_{DS}I\ W_C$ as given above or 5.84 $S_{DS}I$ kN/m of wall or the force specified in Section 10.11.1.2. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1200 mm.

10.11.2.9 **Inverted Pendulum-Type Structures.** Supporting columns or piers of inverted pendulum-type structures shall be designed for the bending moment calculated at the base determined using the procedures given in Section 10.7 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

10.11.2.10 **Anchorage of Nonstructural Systems.** When required by Chapter 12, all portions or components of the structure shall be anchored for the seismic force, F_p, prescribed therein.

10.11.2.11 **Elements Supporting Discontinuous Walls or Frames.** Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having plan irregularity Type 4 of Table 10.3.2.1 or vertical irregularity Type 4 of Table 10.3.2.2 shall have the design strength to resist the maximum axial force that can develop in accordance with the special seismic loads of Section 10.4.1.

10.11.3 **Seismic Design Category C.** Structures assigned to Category C shall conform to the requirements of Section 10.11.2 for Category B and to the requirements of this Section.

10.11.3.1 **Collector Elements.** Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces. Collector elements, splices, and their connections to resisting elements shall resist the special seismic loads of Section 10.4.1.

Exception: In structures or portions thereof braced entirely by light-frame shear walls, collector elements, splices, and connections to resisting elements need only be designed to resist forces in accordance with Eq. 10.11.4.4.

The quantity Ω_o E in Eq. 10.4.1-1 need not exceed the maximum force that can be transferred to the collector by the diaphragm and other elements of the lateral force-resisting system.

10.11.3.2 **Anchorage of Concrete or Masonry Walls.** Concrete or masonry walls shall be anchored to all floors, roofs, and members that provide out-of-plane lateral support for the wall or that are supported by the wall. The anchorage shall provide a positive direct connection between the wall and floor, roof, or supporting member capable of resisting horizontal forces specified in this Section for structures with flexible diaphragms or with Section 12.1.3 (using a_p of 1.0 and R_p of 2.5) for structures with diaphragms that are not flexible.

Anchorage of walls to flexible diaphragms shall have the strength to develop the out-of-plane force given by Eq. 10.11.3.2:

$$F_p = 0.8 S_{DS} I W_p \qquad \textbf{(Eq. 10.11.3.2)}$$

where

F_p = the design force in the individual anchors

S_{DS} = the design spectral response acceleration at short periods per Section 9.4.4

I = the occupancy importance factor per Section 9.5

W_p = the weight of the wall tributary to the anchor

Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Added chords may be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross ties. The maximum length-to-width ratio of the structural subdiaphragm shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

The strength design forces for steel elements of the wall anchorage system, other than anchor bolts and reinforcing steel, shall be 1.4 times the forces otherwise required by this Section.

In metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this Section in the direction perpendicular to the deck span.

Diaphragm to wall anchorage using embedded straps shall be attached to, or hooked around, the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

When elements of the wall anchorage system are loaded eccentrically or are not perpendicular to the wall, the system shall be designed to resist all components of the forces induced by the eccentricity.

When pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall not be reduced.

10.11.4 **Seismic Design Category D.** Structures assigned to Category D shall conform to the requirements of Section 10.11.3 for Category C and to the requirements of this Section.

10.11.4.1 **Collector Elements.** In addition to the requirements of Section 10.11.3.1, collector elements, splices, and their connections to resisting elements shall resist the forces determined in accordance with Section 10.11.4.4.

10.11.4.2 **Plan or Vertical Irregularities.** When the ratio of the strength provided in any story to the strength required is less than two-thirds of that ratio for the story immediately above, the potentially adverse effect shall be analyzed and the strengths shall be adjusted to compensate for this effect.

For structures having a plan structural irregularity of Type 1, 2, 3, or 4 in Table 10.3.2.1 or a vertical structural irregularity of Type 4 in Table 10.3.2.2, the design forces determined from Section 10.9.2 shall be increased 25% for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for the special seismic loads of Section 10.4.1, in accordance with Section 10.11.3.1.

10.11.4.3 **Vertical Seismic Forces.** The vertical component of earthquake ground motion shall be considered in the design of horizontal cantilever and horizontal prestressed components. The load combinations used in evaluating such components shall include E as defined by Eqs. 10.4-1 and 10.4-2. Horizontal cantilever structural components shall be designed for a minimum net upward force of 0.2 times the dead load in addition to the applicable load combinations of Section 10.4.

10.11.4.4 **Diaphragms.** The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached elements to maintain structural integrity under the individual loading and continue to support the prescribed loads. Floor and roof diaphragms shall be designed to resist design seismic forces determined in accordance with Eq. 10.11.4.4 as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad \textbf{(Eq. 10.11.4.4)}$$

where

F_{px} = the diaphragm design force

F_i = the design force applied to Level i

w_i = the weight tributary to Level i

w_{px} = the weight tributary to the diaphragm at Level x

The force determined from Eq. 10.11.4.4 need not exceed $0.4S_{DS}I\ w_{px}$ but shall not be less than $0.2S_{DS}Iw_{px}$. When the diaphragm is required to transfer design seismic force from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offsets in the placement of the elements or to changes in relative lateral stiffness in the vertical elements, these forces shall be added to those determined from Eq. 10.11.4.4.

SECTION 10.12
DEFLECTION, DRIFT LIMITS, AND BUILDING SEPARATION

10.12.1 **Drift Limits.** The design story drift (Δ) as determined in Section 10.9.7 or 10.10.6, shall not exceed the allowable story drift (Δ_a) as obtained from Table 10.12 for any story. For structures with significant torsional deflections, the maximum drift shall include torsional effects.

10.12.2 **Building Separation.** All structures shall be separated from adjoining structures. Separations shall allow for the displacement (δ_x) as determined in Section 10.9.7.1. Adjacent buildings on the same property shall be separated by at least δ_{xt} where

$$\delta_{xt} = \sqrt{(\delta_{x1})^2 + (\delta_{x2})^2} \qquad \textbf{(Eq. 10.12)}$$

and δ_{x1} and δ_{x2} are total deflections for building 1 & 2 respectively.

TABLE 10.12:
ALLOWABLE STORY DRIFT, Δ_a[a]

Structure	Occupancy Category		
	I, II	III	IV
Structures, other than masonry shear wall or masonry wall frame structures, four stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[b]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[c]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
Masonry wall frame structures	$0.013h_{sx}$	$0.013h_{sx}$	$0.010h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.

[b] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 10.12 is not waived.

[c] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

SECTION 10.13
FOUNDATION DESIGN REQUIREMENTS

10.13.1 **General.** This Section includes only those foundation requirements that are specifically related to seismic-resistant construction. It assumes compliance with other basic requirements, which include, but are not limited to, the extent of the foundation investigation, fills present or to be placed in the area of the structure, slope stability, subsurface drainage, settlement control, and pile, requirements. Except as specifically noted, the term "pile" as used in Sections 10.13.4.4 and 10.13.5.4 includes foundation piers, caissons, and piles, and the term "pile cap" includes the elements to which piles are connected, including grade beams and mats.

10.13.2 **Seismic Design Category A**. There are no special requirements for the foundations of structures assigned to Category A.

10.13.3 **Seismic Design Category B.** The determination of the site coefficient, Section 9.4.3, shall be documented and the resisting capacities of the foundations, subjected to the prescribed seismic forces of Chapters 9 and 10, shall meet the following requirements.

10.13.3.1 **Structural Components.** The design strength of foundation components subjected to seismic forces alone or in combination with other prescribed loads and their detailing requirements shall conform to the requirements of Sections 11.1 through 11.4. The strength of foundation components shall not be less than that required for forces acting without seismic forces.

10.13.3.2 **Soil Capacities.** The capacity of the foundation soil in bearing, or the capacity of the soil interface between pile or pier and the soil, shall be sufficient to support the structure with all prescribed loads, without seismic forces, taking due account of the settlement that the structure can withstand. For the load combination, including earthquake as specified in Section 10.4, the soil capacities must be sufficient to resist loads at acceptable strains considering both the short duration of loading and the dynamic properties of the soil.

10.13.4 **Seismic Design Category C.** Foundations for structures assigned to Category C shall conform to all of the requirements for Categories A and B and to the additional requirements of this Section.

10.13.4.1 **Investigation.** When required by the authority having jurisdiction, a written geotechnical or geologic report shall be submitted. This report shall include, in addition to the requirements of Section 10.13.1 and the evaluations required in Section 10.13.3, the results of an investigation to evaluate the following potential earthquake hazards:

1. Slope instability

2. Liquefaction

3. Lateral spreading

4. Surface rupture

The investigation shall contain recommendations for appropriate foundation designs or other measures to mitigate the effects of the above hazards.

10.13.4.2 **Pole-Type Structures.** When construction employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth is used to resist lateral loads, the depth of embedment required for posts or poles to resist seismic forces shall be determined by means of the design criteria established in the foundation investigation report.

10.13.4.3 **Foundation Ties.** Individual pile caps, drilled piers, or caissons shall be interconnected by ties. All ties shall have a design strength in tension or compression greater than a force equal to 10% of S_{DS} times the larger pile cap or column factored dead plus factored live load unless it is demonstrated that equivalent restraint will be provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils, very dense granular soils, or other approved means.

10.13.4.4 **Special Pile Requirements.** Concrete piles, concrete-filled steel pipe piles, drilled piers, or caissons require minimum bending, shear, tension, and elastic strain capacities. Refer to Section 15.3.1 for supplementary provisions.

10.13.5 **Foundation Requirements for Seismic Design Category D.** Foundations for structures assigned to Seismic Design Category D shall conform to all of the requirements for Seismic Design Category C construction and to the additional requirements of this Section. Design and construction of concrete foundation components shall conform to the requirements of SBC 304 Section 21.8, except as modified by the requirements of this Section.

10.13.5.1 **Investigation.** The owner shall submit to the authority having jurisdiction a written report that includes an evaluation of the items in Section 10.13.4.1 and the determination of lateral pressures on basement and retaining walls due to earthquake motions.

10.13.5.2 **Foundation Ties.** Individual spread footings founded on soil defined in section 14.1.1 as Site class E or F shall be interconnected by ties. Ties shall conform to Section 10.13.4.3.

10.13.5.3 **Liquefaction Potential and Soil Strength Loss.** The geotechnical report required by Section 10.13.5.1 shall assess potential consequences of liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall discuss mitigation measures. Such measures shall be given consideration in the design of the structure and can include, but are not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements, or any combination of these measures.

10.13.5.4 **Special Pile and Grade Beam Requirements.** Piling shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains (without the structure) modified for soil-pile-structure interaction coupled with pile deformations induced by lateral pile resistance to structure seismic forces. Concrete piles in Site class E or F shall be designed and detailed in accordance with SBC 304 Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 within seven pile diameters of the pile cap and the interfaces of soft to medium stiff or liquefiable strata. Refer to Section 15.3.2 for supplementary provisions in addition to those given in Section 15.3.1. Batter piles and their connections shall be capable of resisting forces and moments from the special seismic load combinations of Section 10.4.1. For precast, prestressed concrete piles, detailing provisions as given in Section 15.3.2.4 shall apply.

Section 21.8.3.3 of SBC 304 need not apply when grade beams have the required strength to resist the forces from the special seismic loads of Section 10.4.1. Section 21.8.4.4 (a) of SBC 304 need not apply to concrete piles. Section 21.8.4.4 (b) of SBC 304 need not apply to precast, prestressed concrete piles.

Design of anchorage of piles into the pile cap shall consider the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. For piles required to resist uplift forces or provide rotational restraint, anchorage into the pile cap shall be capable of developing the following:

1. In the case of uplift, the lesser of the nominal tensile strength of the longitudinal reinforcement in a concrete pile, or the nominal tensile strength of a steel pile, or 1.3 times the pile pullout resistance, or the axial tension force resulting from the special seismic loads of Section 10.4.1.

2. In the case of rotational restraint, the lesser of the axial and shear forces and moments resulting from the special seismic loads of Section 10.4.1 or development of the full axial, bending, and shear nominal strength of the pile.

Splices of pile segments shall develop the nominal strength of the pile section, but the splice need not develop the nominal strength of the pile in tension, shear, and bending when it has been designed to resist axial and shear forces and moments from the special seismic loads of Section 10.4.1.

Pile moments, shears, and lateral deflections used for design shall be established considering the interaction of the shaft and soil. Where the ratio of the depth of embedment of the pile-to-the-pile diameter or width is less than or equal to 6, the pile may be assumed to be flexurally rigid with respect to the soil.

Pile group effects from soil on lateral pile nominal strength shall be included where pile center-to-center spacing in the direction of lateral force is less than eight pile diameters or widths. Pile group effects on vertical nominal strength shall be included where pile center-to-center spacing is less than three pile diameters or widths.

SECTION 10.14
SUPPLEMENTARY METHODS OF ANALYSIS

10.14.1 **Linear Response History Analysis.** A linear response history analysis shall consist of an analysis of a linear mathematical model of the structure to determine its response, through methods of numerical integration, to suites of ground motion acceleration histories compatible with the design response spectrum for the site. For purposes of analysis, the structure shall be permitted to be considered to be fixed at the base, or alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations.

10.14.2 **Nonlinear Response History Analysis.** A nonlinear response history analysis shall consist of an analysis of a mathematical model of the structure that directly accounts for the nonlinear hysteretic behavior of the structure's components to determine its response through methods of numerical integration to suites of ground motion acceleration histories compatible with the design response spectrum for the site.

10.14.3 **Soil-Structure Interaction.** Incorporate the effects of soil-structure interaction is optional. The use of this option will decrease the design values of the base shear, lateral forces, and overturning moments but may increase the computed values of the lateral displacements and the secondary forces associated with the P-delta effects.

CHAPTER 11
MATERIAL SPECIFIC SEISMIC DESIGN AND DETAILING REQUIREMENTS

SECTION 11.1
STEEL

11.1.1 **Reference Documents.** The design, construction, and quality of steel components that resist seismic forces shall conform to the requirements of SBC 306 and the references listed (Ref. 11.1-1 through Ref. 11.1-5) except that modifications are necessary to make the references compatible with the provisions of this document. Section 15.4 provides supplementary provisions for this compatibility.

SECTION 11.2
STRUCTURAL CONCRETE

11.2.1 **Reference Documents.** The quality and testing of materials and the design and construction of structural concrete components that resist seismic forces shall conform to the requirements of SBC 304. Section 15.5 provides the supplementary provisions for this compatibility. The load combinations of Section 2.4.1 are not applicable for design of reinforced concrete to resist earthquake loads.

SECTION 11.3
COMPOSITE STRUCTURES

11.3.1 **Reference Documents.** The design, construction, and quality of composite steel and concrete components that resist seismic forces shall conform to the relevant requirements of SBC 304 and SBC 306 and the references listed (Ref. 11.3-1 and Ref. 11.3-2) except as modified by the provisions of this chapter.

SECTION 11.4
MASONRY

11.4.1 **Reference Documents.** The design, construction, and quality assurance of masonry components that resist seismic forces shall conform to the requirements of SBC 304 and Ref. 11.4-1 except that modifications are necessary to make the reference compatible with the provisions of this document. Section 15.6 provides the supplementary provisions for this compatibility.

CHAPTER 12
SEISMIC DESIGN REQUIREMENTS FOR NON-STRUCTURAL COMPONENTS

SECTION 12.1
GENERAL

Chapter 12 establishes minimum design criteria for architectural, mechanical, electrical, and non-structural systems, components, and elements permanently attached to structures including supporting structures and attachments (hereinafter referred to as "components"). The design criteria establish minimum equivalent static force levels and relative displacement demands for the design of components and their attachments to the structure, recognizing ground motion and structural amplification, component toughness and weight, and performance expectations. Seismic Design Categories for structures are defined in Section 9.6. For the purposes of this Section, components shall be considered to have the same Seismic Design Category as that of the structure that they occupy or to which they are attached unless otherwise noted.

This Chapter also establishes minimum seismic design force requirements for nonbuilding structures that are supported by other structures where the weight of the nonbuilding structure is less than 25% of the combined weight of the nonbuilding structure and the supporting structure. Seismic design requirements for nonbuilding structures that are supported by other structures where the weight of the nonbuilding structure is 25% or more of the combined weight of the nonbuilding structure and supporting structure are prescribed in Chapter 13. Seismic design requirements for nonbuilding structures that are supported at grade are prescribed in Chapter 13; however, the minimum seismic design forces for nonbuilding structures that are supported by another structure shall be determined in accordance with the requirements of Section 12.1.3 with R_p equal to the value of R specified in Chapter 13 and $a_p = 2.5$ for nonbuilding structures with flexible dynamic characteristics and $a_p = 1.0$ for nonbuilding structures with rigid dynamic characteristics. The distribution of lateral forces for the supported nonbuilding structure and all nonforce requirements specified in Chapter 13 shall apply to supported nonbuilding structures.

In addition, all components are assigned a component importance factor (I_p) in this chapter. The default value for I_p is 1.00 for typical components in normal service. Higher values for I_p are assigned for components, which contain hazardous substances, must have a higher level of assurance of function, or otherwise require additional attention because of their life safety characteristics. Component importance factors are prescribed in Section 12.1.5.

All architectural, mechanical, electrical, and other non-structural components in structures shall be designed and constructed to resist the equivalent static forces and displacements determined in accordance with this Chapter. The design and evaluation of support structures and architectural components and equipment shall consider their flexibility as well as their strength.

Exception: The following components are exempt from the requirements of this Chapter:

1. All components in Seismic Design Category A.

2. Architectural components in Seismic Design Category B other than parapets supported by bearing walls or shear walls provided that the importance factor (I_p) is equal to 1.0.

3. Mechanical and electrical components in Seismic Design Category B.

4. Mechanical and electrical components in structures assigned to Seismic Design Category C provided that the importance factor (I_p) is equal to 1.0.

5. Mechanical and electrical components in Seismic Design Category D where I_p = 1.0 and flexible connections between the components and associated ductwork, piping, and conduit are provided and that are mounted at (1.25 m) or less above a floor level and weigh (1800 N) or less.

6. Mechanical and electrical components in Seismic Design Category D weighing (100 N) or less where I_p = 1.0 and flexible connections between the components and associated ductwork, piping, and conduit are provided, or for distribution systems, weighing (7 N/m) or less.

The functional and physical interrelationship of components and their effect on each other shall be designed so that the failure of an essential or nonessential architectural, mechanical, or electrical component shall not cause the failure of a nearby essential architectural, mechanical, or electrical component.

12.1.1 Reference Standards.

12.1.1.1 Consensus Standards. The cited references (Ref. 12-1 through 12-13) are consensus standards and are to be considered part of these provisions to the extent referred to in this chapter.

12.1.1.2 Accepted Standards. The cited references (Ref. 12-14 through 12-21) are standards developed within the industry and represent acceptable procedures for design and construction.

12.1.2 Component Force Transfer. Components shall be attached such that the component forces are transferred to the structure. Component seismic attachments shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be provided. Local elements of the supporting structure shall be designed and constructed for the component forces where they control the design of the elements or their connections. The component forces shall be those determined in Section 12.1.3, except that modifications to F_p and R_p due to anchorage conditions need not be considered. The design documents shall include sufficient information relating to the attachments to verify compliance with the requirements of this chapter.

12.1.3 Seismic Forces. Seismic forces (F_p) shall be determined in accordance with Eq. 12.1.3-1:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) \qquad \textbf{(Eq. 12.1.3-1)}$$

F_p is not required to be taken as greater than

$$F_p = 1.6\ S_{DS}\, I_p\, W_p \qquad \textbf{(Eq. 12.1.3-2)}$$

and F_p shall not be taken as less than

$$F_p = 0.3\ S_{DS}\, I_p\, W_p \qquad \textbf{(Eq. 12.1.3-3)}$$

where

F_p	=	seismic design force centered at the component's center of gravity and distributed relative to component's mass distribution
S_{DS}	=	spectral acceleration, short period, as determined from Section 9.4.4
a_p	=	component amplification factor that varies from 1.00 to 2.50 (select appropriate value from Table 12.2.2 or 12.3.2)
I_p	=	component importance factor that varies from 1.00 to 1.50 (see Section 12.1.5)
W_p	=	component operating weight
R_p	=	component response modification factor that varies from 1.50 to 5.00 (select appropriate value from Tables 12.2.2 or 12.3.2)
z	=	height in structure of point of attachment of component with respect to the base. For items at or below the base, z shall be taken as 0. The value of z/h need not exceed 1.0
h	=	average roof height of structure with respect to the base

The force (F_p) shall be applied independently, longitudinally and laterally in combination with service loads associated with the component. Combine horizontal and vertical load effects as indicated in Section 10.4 substituting F_p for the term Q_E. The reliability/redundancy factor, ρ, is permitted to be taken equal to 1.

When positive and negative wind loads exceed F_p for nonbearing exterior wall, these wind loads shall govern the design. Similarly, when the building code horizontal loads exceed F_p for interior partitions, these building code loads shall govern the design.

12.1.4 Seismic Relative Displacements. Seismic relative displacements (D_p) shall be determined in accordance with the following equations:

For two connection points on the same Structure A or the same structural system, one at a height h_x and the other at a height h_y, D_p shall be determined as

$$D_p = \delta_{xA} - \delta_{yA} \qquad \textbf{(Eq. 12.1.4-1)}$$

D_p *is* not required to be taken as greater than

$$D_p = (h_x - h_y)\ \Delta_{aA} / h_{sx} \qquad \textbf{(Eq. 12.1.4-2)}$$

For two connection points on separate Structures A or B or separate structural systems, one at a height h_x and the other at a height h_y, D_p shall be determined as

$$D_p = |\delta_{xA}| + |\delta_{yB}| \qquad \textbf{(Eq. 12.1.4-3)}$$

D_p *is* not required to be taken as greater than

$$D_p = h_x \Delta_{aA} / h_{sx} + h_y \Delta_{aB} / h_{sx} \qquad \textbf{(Eq. 12.1.4-4)}$$

where

D_p = relative seismic displacement that the component must be designed to accommodate

δ_{xA} = deflection at building Level x of Structure A, determined by an elastic analysis as defined in Section 10.9.7.1

δ_{yA} = deflection at building Level y of Structure A, determined by an elastic analysis as defined in Section 10.9.7.1

δ_{yB} = deflection at building Level y of Structure B, determined by an elastic analysis as defined in Section 10.9.7.1

h_x = height of Level x to which upper connection point is attached

h_y = height of Level y to which lower connection point is attached

Δ_{aA} = allowable story drift for Structure A as defined in Table 10.12

Δ_{aB} = allowable story drift for Structure B as defined in Table 10.12

h_{sx} = story height used in the definition of the allowable drift Δ_a in Table 10.12, note that Δ_a/h_{sx} = the drift index

The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.

12.1.5 **Component Importance Factor.** The component importance factor (I_p) shall be selected as follows:

I_p = 1.5 for life safety component required to function after an earthquake (e.g., fire protection sprinkler system)

I_p = 1.5 for component that contains hazardous content

I_p = 1.5 for storage racks in structures open to the public (e.g., warehouse retails stores)

I_p = 1.0 for all other components

In addition, for structures in Occupancy Category IV:

I_p = 1.5 for all components needed for continued operation of the facility or whose failure could impair the continued operation of the facility

12.1.6 **Component Anchorage.** Components shall be anchored in accordance with the following provisions.

12.1.6.1 The force in the connected part shall be determined based on the prescribed forces for the component specified in Section 12.1.3. Where component anchorage is provided by shallow expansion anchors, shallow chemical anchors, or shallow (low deformability) cast-in-place anchors, a value of Rp = 1.5 shall be used in Section 12.1.3 to determine the forces in the connected part.

12.1.6.2 Anchors embedded in concrete or masonry shall be proportioned to carry the least of the following:

a. The design strength of the connected part,

b. 1.3 times the force in the connected part due to the prescribed forces, or

c. The maximum force that can be transferred to the connected part by the component structural system.

12.1.6.3 Determination of forces in anchors shall take into account the expected conditions of installation including eccentricities and prying effects.

12.1.6.4 Determination of force distribution of multiple anchors at one location shall take into account the stiffness of the connected system and its ability to redistribute loads to other anchors in the group beyond yield.

12.1.6.5 Powder driven fasteners shall not be used for tension load applications in Seismic Design Category D unless approved for such loading.

12.1.6.6 The design strength of anchors in concrete shall be determined in accordance with the provisions of Section 11.2.

12.1.7 **Construction Documents.** Construction documents shall be prepared to comply with the requirements indicated in Table 12.1.7.

SECTION 12.2
ARCHITECTURAL COMPONENT DESIGN

12.2.1 **General.** Architectural systems, components, or elements (hereinafter referred to as "components") listed in Table 12.2.2 and their attachments shall meet the requirements of Sections 12.2.2 through 12.2.9.

12.2.2 **Architectural Component Forces and Displacements.** Architectural components shall meet the force requirements of Section 12.1.3 and Table 12.2.2.

Components supported by chains or otherwise suspended from the structural system above are not required to meet the lateral seismic force requirements and seismic relative displacement requirements of this Section provided that they cannot be damaged to become a hazard or cannot damage any other component when subject to seismic motion and they have ductile or articulating connections to the structure at the point of attachment. The gravity design load for these items shall be three times their operating load.

12.2.3 **Architectural Component Deformation.** Architectural components that could pose a life safety hazard shall be designed for the seismic relative displacement requirements of Section 12.1.4. Architectural components shall be designed for vertical deflection due to joint rotation of cantilever structural members.

12.2.4 **Exterior Nonstructural Wall Elements and Connections.**

12.2.4.1 **General.** Exterior nonstructural wall panels or elements that are attached to or enclose the structure shall be designed to resist the forces in accordance with Eq. 12.1.3-1 or 12.1.3-2, and shall accommodate movements of the structure resulting from response to the design basis ground motion, D_p, or temperature changes. Such elements shall be supported by means of positive and direct structural supports or by mechanical connections and fasteners. The support system shall be designed in accordance with the following:

a. Connections and panel joints shall allow for the story drift caused by relative seismic displacements (D_p) determined in Section 12.1.4, or 13 mm, whichever is greatest.

b. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections that provide equivalent sliding or ductile capacity.

c. The connecting member itself shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds.

d. All fasteners in the connecting system such as bolts, inserts, welds, and dowels and the body of the connectors shall be designed for the force (F_p) determined by Eq. 12.1.3-2 with values of R_p and a_p taken from Table 12.2.2 applied at the center of mass of the panel.

e. Anchorage using flat straps embedded in concrete or masonry shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel or to assure that pullout of anchorage is not the initial failure mechanism.

TABLE 12.1.7:
CONSTRUCTION DOCUMENTS

Component Description	**Section Reference**		**Required Seismic Design Categories**
	Quality Assurance	**Design**	
Exterior wall panels, including anchorage	15.2.4.8 No. 1	12.2.4	D
Suspended ceiling system, including anchorage	15.2.4.8 No. 2	12.2.6	D
Access floors, including anchorage	15.2.4.8 No. 2	12.2.7	D
Steel storage racks, including anchorage	15.2.4.8 No. 2	12.2.9	D
Glass in glazed curtain walls, glazed storefronts, and interior glazed partitions, including anchorage	15.2.4.8 No. 3	12.2.10	D
HVAC ductwork containing hazardous materials, including anchorage	15.2.4.9 No. 3	12.3.10	C, D
Piping systems and mechanical units containing flammable, combustible, or highly toxic materials	15.2.4.9 No. 2	12.3.11 12.3.12 12.3.13	C, D
Anchorage of electrical equipment for emergency or standby power systems	15.2.4.9 No. 1	12.3.14	C, D
Project-specific requirements for mechanical and electrical components and their anchorage	15.2.5.5	12.3	C, D

TABLE 12.2.2:
ARCHITECTURAL COMPONENT COEFFICIENTS

Architectural Component or Element	a_p[a]	R_p[b]
Interior Nonstructural Walls and Partitions		
Plain (unreinforced) masonry walls	1	1.5
All other walls and partitions	1	2.5
Cantilever Elements (Unbraced or Braced to Structural Frame Below Its Center of Mass)		
Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys and stacks when laterally braced or supported by the structural frame	2.5	2.5
Cantilever Elements (Braced to Structural Frame Above Its Center of Mass)		
Parapets	1.0	2.5
Chimneys and stacks	1.0	2.5
Exterior nonstructural walls	1.0[b]	2.5
Exterior Nonstructural Wall Elements and Connections		
Wall element	1	2.5
Body of wall panel connections	1	2.5
Fasteners of the connecting system	1.25	1
Veneer		
Limited deformability elements and attachments	1	2.5
Low deformability elements and attachments	1	2.5
Penthouses (Except when Framed by an Extension of the Building Frame)		
Ceilings	2.5	3.5
All	1	2.5
Cabinets		
Storage cabinets and laboratory equipment	1	2.5
Access Floors		
Special access floors (designed in accordance with Section 12.2.7.2)	1	2.5
All other	1	1.5
Appendages and Ornamentations	2.5	2.5
Signs and Billboards	2.5	2.5
Other Rigid Components		
High deformability elements and attachments	1	3.5
Limited deformability elements and attachments	1	2.5
Low deformability materials and attachments	1	1.5
Other Flexible Components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5

[a] A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid and rigidly attached. The value of $a_p = 2.5$ is for equipment generally regarded as flexible or flexibly attached. See Section 9.2 for definitions of rigid and flexible.

[b] Where flexible diaphragms provide lateral support for walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 10.11.

12.2.4.2 **Glass**. Glass in glazed curtain walls and storefronts shall be designed and installed in accordance with Section 12.2.10.

12.2.5 **Out-of-Plane Bending.** Transverse or out of plane bending or deformation of a component or system that is subjected to forces as determined in Section 12.2.2 shall not exceed the deflection capability of the component or system.

12.2.6 **Suspended Ceilings.** Suspended ceilings shall be designed to meet the seismic force requirements of Section 12.2.6.1. In addition, suspended ceilings shall meet the requirements of either industry standard construction as modified in Section 12.2.6.2 or integral construction as specified in Section 12.2.6.3.

12.2.6.1 **Seismic Forces.** Suspended ceilings shall be designed to meet the force requirements of Section 12.1.3.

The weight of the ceiling, W_p, shall include the ceiling grid and panels; light fixtures if attached to, clipped to, or laterally supported by the ceiling grid; and other components which are laterally supported by the ceiling. W_p shall be taken as not less than 20 N/m^2.

The seismic force, F_p, shall be transmitted through the ceiling attachments to the building structural elements or the ceiling-structure boundary.

Design of anchorage and connections shall be in accordance with these provisions.

12.2.6.2 **Industry Standard Construction.** Unless designed in accordance with Section 12.2.6.3, suspended ceilings shall be designed and constructed in accordance with this Section.

12.2.6.2.1 **Seismic Design Category C.** Suspended ceilings in Seismic Design Category C shall be designed and installed in accordance with the CISCA recommendations for seismic Zones 0-2, (Ref. 12-16), except that seismic forces shall be determined in accordance with Sections 12.1.3 and 12.2.6.1.

Sprinkler heads and other penetrations in Seismic Design Category C shall have a minimum of 6 mm clearance on all sides.

12.2.6.2.2 **Seismic Design Category D.** Suspended ceilings in Seismic Design Category D shall be designed and installed in accordance with the CISCA recommendations for seismic Zones 3-4 (Ref. 12-17) and the additional requirements listed in this subsection.

a. A heavy duty T-bar grid system shall be used.

b. The width of the perimeter supporting closure angle shall be not less than 50 mm. In each orthogonal horizontal direction, one end of the ceiling grid shall be attached to the closure angle. The other end in each horizontal direction shall have a 20 mm clearance from the wall and shall rest upon and be free to slide on a closure angle.

c. For ceiling areas exceeding 100 m^2, horizontal restraint of the ceiling to the structural system shall be provided. The tributary areas of the horizontal restraints shall be approximately equal.

Exception: Rigid braces are permitted to be used instead of diagonal splay wires. Braces and attachments to the structural system above shall be adequate to limit relative lateral deflections at point of attachment of ceiling grid to less than 6 mm for the loads prescribed in Section 12.1.3.

d. For ceiling areas exceeding 250 m^2, a seismic separation joint or full height partition that breaks the ceiling up into areas not exceeding 250 m^2 shall be provided unless structural analyses are performed of the ceiling bracing system for the prescribed seismic forces which demonstrate ceiling system penetrations and closure angles provide sufficient clearance to accommodate the additional movement. Each area shall be provided with closure angles in accordance with Item b and horizontal restraints or bracing in accordance with Item c.

e. Except where rigid braces are used to limit lateral deflections, sprinkler heads and other penetrations shall have a 50 mm over-size ring, sleeve, or adapter through the ceiling tile to allow for free movement of at least 25 mm in all horizontal directions. Alternatively, a swing joint that can accommodate 25 mm of ceiling movement in all horizontal directions are permitted to be provided at the top of the sprinkler head extension.

f. Changes in ceiling plan elevation shall be provided with positive bracing.

g. Cable trays and electrical conduits shall be supported independently of the ceiling.

h. Suspended ceilings shall be subject to the special inspection requirements of Section 15.2.4.8.

12.2.6.3 **Integral Ceiling/Sprinkler Construction.** As an alternative to providing large clearances around sprinkler system penetrations through ceiling systems, the sprinkler system and ceiling grid are permitted to be designed and tied together as an integral unit. Such a design shall consider the mass and flexibility of all elements involved, including: ceiling system, sprinkler system, light fixtures, and mechanical (HVAC) appurtenances. The design shall be performed by a registered design professional.

12.2.7 Access Floors.

12.2.7.1 **General.** Access floors shall be designed to meet the force provisions of Section 12.1.3 and the additional provisions of this Section. The weight of the access floor, W_p, shall include the weight of the floor system, 100% of the weight of all equipment fastened to the floor, and 25% of the weight of all equipment supported by, but not fastened to the floor. The seismic force, F_p, shall be transmitted from the top surface of the access floor to the supporting structure.

Overturning effects of equipment fastened to the access floor panels also shall be considered. The ability of "slip on" heads for pedestals shall be evaluated for suitability to transfer overturning effects of equipment.

When checking individual pedestals for overturning effects, the maximum concurrent axial load shall not exceed the portion of W_p assigned to the pedestal under consideration.

12.2.7.2 **Special Access Floors.** Access floors shall be considered to be "special access floors" if they are designed to comply with the following considerations:

1. Connections transmitting seismic loads consist of mechanical fasteners, concrete anchors, welding, or bearing. Design load capacities comply with recognized design codes and/or certified test results.

2. Seismic loads are not transmitted by friction, produced solely by the effects of gravity, powder-actuated fasteners (shot pins), or adhesives.

3. The design analysis of the bracing system includes the destabilizing effects of individual members buckling in compression.

4. Bracing and pedestals are of structural or mechanical shape produced to ASTM specifications that specify minimum mechanical properties. Electrical tubing shall not be used.

5. Floor stringers that are designed to carry axial seismic loads and that are mechanically fastened to the supporting pedestals are used.

12.2.8 Partitions.

12.2.8.1 General. Partitions that are tied to the ceiling and all partitions greater than 1.8 m in height shall be laterally braced to the building structure. Such bracing shall be independent of any ceiling splay bracing. Bracing shall be spaced to limit horizontal deflection at the partition head to be compatible with ceiling deflection requirements as determined in Section 12.2.6 for suspended ceilings and Section 12.2.2 for other systems.

12.2.8.2 Glass. Glass in glazed partitions shall be designed and installed in accordance with Section 12.2.10.

12.2.9 Steel Storage Racks. Steel storage racks sup-ported at the base of the structure shall be designed to meet the force requirements of Chapter 13. Steel storage racks supported above the base of the structure shall be designed to meet the force requirements of Sections 12.1 and 12.2.

12.2.10 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions.

12.2.10.1 General. Glass in glazed curtain walls, glazed storefronts, and glazed partitions shall meet the relative displacement requirement of Eq. 12.2.10.1-1:

$$\Delta_{fallout} \geq 1.25\, D_p\, I \qquad \textbf{(Eq. 12.2.10.1-1)}$$

or 13 mm, whichever is greater; where

$\Delta_{fallout}$ = the relative seismic displacement (drift) causing glass fallout from the curtain wall, storefront wall, or partition (Section 12.2.10.2)

D_p = the relative seismic displacement that the component must be designed to accommodate (Eq. 12.1.4-1). D_p shall be applied over the height of the glass component under consideration

I = the occupancy importance factor (Table 9.5)

Exceptions:

1. Glass with sufficient clearances from its frame such that physical contact between the glass and frame will not occur at the design drift, as demonstrated by Eq. 12.2.10.1-2, shall be exempted from the provisions of Eq. 12.2.10.1-1:

$$D_{clear} \geq 1.25\, D_p \qquad \textbf{(Eq. 12.2.10.1-2)}$$

where

$$D_{clear} = 2c_1 \left(1 + \frac{h_p c_2}{b_p c_1}\right)$$

h_p = the height of the rectangular glass

b_p = the width of the rectangular glass

c_1 = the clearance (gap) between the vertical glass edges and the frame, and

c_2 = the clearance (gap) between the horizontal glass edges and the frame

2. Fully tempered monolithic glass in Occupancy Category I, II or III located no more than 3 m above a walking surface shall be exempted from the provisions of Eq. 12.2.10.1-1.

3. Annealed or heat-strengthened laminated glass in single thickness with interlayer no less than 0.76 mm that is captured mechanically in a wall system glazing pocket, and whose perimeter is secured to the frame by a wet glazed gunable curing elastomeric sealant perimeter bead of 13 mm minimum glass contact width, or other approved anchorage system shall be exempted from the provisions of Eq. 12.2.10.1-1.

12.2.10.2 **Seismic Drift Limits for Glass Components.** $\Delta_{fallout,}$ the drift causing glass fallout from the curtain wall, storefront, or partition shall be determined in accordance with Ref. 12-21, or by engineering analysis.

SECTION 12.3
MECHANICAL AND ELECTRICAL COMPONENT DESIGN

12.3.1 **General.** Attachments and equipment supports for the mechanical and electrical systems, components, or elements (hereinafter referred to as "components") shall meet the requirements of Sections 12.3.2 through 12.3.16.

12.3.2 **Mechanical and Electrical Component Forces and Displacements.** Mechanical and electrical components shall meet the force and seismic relative displacement requirements of Sections 12.1.3, 12.1.4, and Table 12.3.2.
Components supported by chains or otherwise suspended from the structural system above are not required to meet the lateral seismic force requirements and seismic relative displacement requirements of this Section provided they are designed to prevent damage to themselves or causing damage to any other component when subject to seismic motion. Such supports shall have ductile or articulating connections to the structure at the point of attachment. The gravity design load for these items shall be three times their operating load.

12.3.3 **Mechanical and Electrical Component Period.** The fundamental period of the mechanical and electrical component (and its attachment to the building), T_p, shall be determined by the following equation provided that the component and attachment can be reasonably represented analytically by a simple spring and mass single degree of freedom system:

$$T_p = 2\pi \sqrt{\frac{W_p}{K_p g}} \qquad \textbf{(Eq. 12.3.3)}$$

where

T_p = component fundamental period

W_p = component operating weight

g = gravitational acceleration

K_p = stiffness of resilient support system of the component and attachment, determined in terms of load per unit deflection at the center of gravity of the component

Note that consistent units must be used.
Otherwise, determine the fundamental period of the component in seconds (T_p) from experimental test data or by a properly substantiated analysis.

12.3.4 **Mechanical and Electrical Component Attachments.** The stiffness of mechanical and electrical component attachments shall be designed such that the load path for the component performs its intended function.

12.3.5 **Component Supports.** Mechanical and electrical component supports and the means by which they are attached to the component shall be designed for the forces determined in Section 12.1.3 and in conformance with Sections 11.1 through 11.4, as appropriate, for the materials comprising the means of attachment. Such supports include structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, stays, snubbers, and tethers, as well as element forged or cast as a part of the mechanical or electrical component. If standard or proprietary supports are used, they shall be designed by either load rating (i.e., testing) or for the calculated seismic forces. In addition, the stiffness of the support, when appropriate, shall be designed such that the seismic load path for the component performs its intended function.

Component supports shall be designed to accommodate the seismic relative displacements between points of support determined in accordance with Section 12.1.4.

In addition, the means by which supports are attached to the component, except when integral (i.e., cast or forged), shall be designed to accommodate both the forces and displacements determined in accordance with Sections 12.1.3 and 12.1.4. If the value of I_p = 1.5 for the component, the local region of the support attachment point to the component shall be evaluated for the effect of the load transfer on the component wall.

12.3.6 **Component Certification.** Architectural, mechanical, and electrical components shall comply with the force requirements of Chapter 12. Components designated with an I_p greater than 1.0 in Seismic Design Category C and D shall meet additional requirements of Section 15.2.5.5 and in particular, mechanical and electrical equipment which must remain operable following the design earthquake shall demonstrate operability by shake table testing or experience data.
The manufacturer's certificate of compliance indicating compliance with this Section shall be submitted to the authority having jurisdiction when required by the contract documents or when required by the regulatory agency.

12.3.7 **Utility and Service Lines at Structure Interfaces.** At the interface of adjacent structures or portions of the same structure that may move independently, utility lines shall be provided with adequate flexibility to accommodate the anticipated differential movement between the portions that move independently. Differential displacement calculations shall be determined in accordance with Section 12.1.4.

12.3.8 **Site-Specific Considerations.** The possible interruption of utility service shall be considered in relation to designated seismic systems in Occupancy Category IV. Specific attention shall be given to the vulnerability of underground utilities and utility interfaces between the structure and the ground where the seismic coefficient S_{DS} at the underground utility or at the base of the structure is equal to or greater than 0.33.

TABLE 12.3.2:
MECHANICAL AND ELECTRICAL COMPONENTS SEISMIC COEFFICIENTS

Mechanical and Electrical Component or Element[b]	a_p[a]	R_p
General Mechanical Equipment		
Boilers and furnaces	1.0	2.5
Pressure vessels on skirts and free-standing stacks	2.5	2.5
Cantilevered chimneys	2.5	2.5
Other	1.0	2.5
Manufacturing and Process Machinery		
General	1.0	2.5
Conveyors (non-personnel)	2.5	2.5
Piping Systems		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
HVAC Systems		
Vibration isolated	2.5	2.5
Nonvibration isolated	1.0	2.5
Mounted in-line with ductwork	1.0	2.5
Other	1.0	2.5
Elevator Components	1.0	2.5
Escalator Components	1.0	2.5
Trussed Towers (free-standing or guyed)	2.5	2.5
General Electrical		
Distribution systems (bus ducts, conduit, cable tray)	2.5	5.0
Equipment	1.0	2.5
Lighting Fixtures	1.0	1.5

[a] A lower value for a_p shall not be used unless justified by detailed dynamic analyses. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid or rigidly attached. The value of $a_p = 2.5$ is for equipment generally regarded as flexible or flexibly attached. See Section 9.2 for definitions of rigid and flexible.
[b] Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_p$ if the maximum clearance (air gap) between the equipment support frame and restraint is greater than 6 mm. If the maximum clearance is specified on the construction documents to be not greater than 6 mm, the design force may be taken as F_p.

12.3.9 **Storage Tanks Mounted in Structures.** Storage tanks, including their attachments and supports, shall be designed to meet the force requirements of Chapter 13.

12.3.10 **HVAC Ductwork.** Attachments and supports for HVAC ductwork systems shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. In addition to their attachments and supports, ductwork systems designated as having an I_p = 1.5 themselves shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. Where HVAC ductwork runs between structures that could displace relative to one another and for seismically isolated structures where the HVAC ductwork crosses the seismic isolation interface, the HVAC ductwork shall be designed to accommodate the seismic relative displacements specified in Section 12.1.4.

Seismic restraints are not required for HVAC ducts with I_p = 1.0 if either of the following conditions are met:

a. HVAC ducts are suspended from hangers 300 mm or less in length from the top of the duct to the supporting structure. The hangers shall be detailed to avoid significant bending of the hangers and their attachments,

or

b. HVAC ducts have a cross-sectional area of less than 0.6 m^2.

Equipment items installed in-line with the duct system (e.g., fans, heat exchangers, and humidifiers) weighing more than 350 N shall be supported and laterally braced independent of the duct system and shall meet the force requirements of Section 12.1.3. Appurtenances such as dampers, louvers, and diffusers shall be positively attached with mechanical fasteners. Unbraced piping attached to in-line equipment shall be provided with adequate flexibility to accommodate differential displacements.

12.3.11 **Piping Systems.** Attachments and supports for piping systems shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. In addition to their attachments and supports, piping systems designated as having I_p = 1.5 themselves shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. Where piping systems are attached to structures that could displace relative to one another and for seismically isolated structures where the piping system crosses the seismic isolation interface, the piping system shall be designed to accommodate the seismic relative displacements specified in Section 12.1.4.

Seismic effects that shall be considered in the design of a piping system include the dynamic effects of the piping system, its contents, and, when appropriate, its supports. The interaction between the piping system and the supporting structures, including other mechanical and electrical equipment shall also be considered.

12.3.11.1 **Pressure Piping Systems.** Pressure piping systems designed and constructed in accordance with ASME B31 (Ref. 12-5) shall be deemed to meet the force, displacement, and other provisions of this Section. In lieu of specific force and displacement provisions provided in the ASME B31, the force and displacement provisions of Sections 12.1.3 and 12.1.4 shall be used.

12.3.11.2 **Fire Protection Sprinkler Systems.** Fire protection sprinkler systems designed and constructed in accordance with NFPA 13, (Ref. 12-13) shall be deemed to meet the other requirements of this Section, except the force and displacement requirements of Sections 12.1.3 and 12.1.4 shall be satisfied.

12.3.11.3 **Other Piping Systems.** Piping designated as having an I_p = 1.5 but not designed and constructed in accordance with ASME B31 (Ref. 12-5) or NFPA 13 (Ref. 12-13) shall meet the following:

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

1. For piping systems constructed with ductile materials (e.g., steel, aluminum, or copper), 90% of the piping material yield strength.
2. For threaded connections with ductile materials, 70% of the piping

material yield strength.

3. For piping constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25% of the piping material minimum specified tensile strength.
4. For threaded connections in piping constructed with non-ductile materials, 20% of the piping material minimum specified tensile strength.

b. Provisions shall be made to mitigate seismic impact for piping components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications).

c. Piping shall be investigated to ensure that the piping has adequate flexibility between support attachment points to the structure, ground, other mechanical and electrical equipment, or other piping.

d. Piping shall be investigated to ensure that the interaction effects between it and other piping or constructions are acceptable.

12.3.11.4 **Supports and Attachments for Other Piping.** Attachments and supports for piping not designed and constructed in accordance with ASME B31 [Ref. 12-5] or NFPA 13 [Ref. 12-13] shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with a nationally recognized structural code such as, when constructed of steel, SBC 306 or MSS SP-58 (Ref. 12-15).

b. Attachments embedded in concrete shall be suitable for cyclic loads.

c. Rod hangers shall not be used as seismic supports unless the length of the hanger from the supporting structure is 300 mm or less. Rod hangers shall not be constructed in a manner that subjects the rod to bending moments.

d. Seismic supports are not required for:

1. Ductile piping in Seismic Design Category D designated as having an I_p = 1.5 and a nominal pipe size of 25 mm or less when provisions are made to protect the piping from impact or to avoid the impact of larger piping or other mechanical equipment.
2. Ductile piping in Seismic Design Category C designated as having an I_p = 1.5 and a nominal pipe size of 50 mm or less when provisions are made to protect the piping from impact or to avoid the impact of larger piping or other mechanical equipment.
3. Ductile piping in Seismic Design Category D designated as having an I_p = 1.0 and a nominal pipe size of 75 mm or less.
4. Ductile piping in Seismic Design Category A, B, or C designated as having an I_p = 1.0 and a nominal pipe size of 150 mm or less.

e. Seismic supports shall be constructed so that support engagement is maintained.

12.3.12 **Boilers and Pressure Vessels.** Attachments and supports for boilers and pressure vessels shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. In addition to their attachments and supports, boilers and pressure vessels designated as having an I_p = 1.5 themselves shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4.

The seismic design of a boiler or pressure vessel shall include analysis of the following: the dynamic effects of the boiler or pressure vessel, its contents, and its supports; sloshing of liquid contents; loads from attached components such as piping; and the interaction between the boiler or pressure vessel and its support.

12.3.12.1 **ASME Boilers and Pressure Vessels.** Boilers or pressure vessels designed in accordance with the ASME *Code* (Ref. 12-2) shall be deemed to meet the force, displacement, and other requirements of this Section. In lieu of the specific force and displacement provisions provided in the ASME code, the force and displacement provisions of Sections 12.1.3 and 12.1.4 shall be used.

12.3.12.2 **Other Boilers and Pressure Vessels.** Boilers and pressure vessels designated as having an I_p = 1.5 but not constructed in accordance with the provisions of the ASME code (Ref. 12-2) shall meet the following provisions:

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

1. For boilers and pressure vessels constructed with ductile materials (e.g., steel, aluminum, or copper), 90% of the material minimum specified yield strength.
2. For threaded connections in boilers or pressure vessels or their supports constructed with ductile materials, 70% of the material minimum specified yield strength.
3. For boilers and pressure vessels constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25% of the material minimum specified tensile strength.
4. For threaded connections in boilers or pressure vessels or their supports constructed with nonductile materials, 20% of the material minimum specified tensile strength.

b. Provisions shall be made to mitigate seismic impact for boiler and pressure vessel components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications).

c. Boilers and pressure vessels shall be investigated to ensure that the interaction effects between them and other constructions are acceptable.

12.3.12.3 **Supports and Attachments for Other Boilers and Pressure Vessels.** Attachments and supports for boilers and pressure vessels shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with nationally recognized structural code such as, when constructed of steel, SBC 306.

b. Attachments embedded in concrete shall be suitable for cyclic loads.

c. Seismic supports shall be constructed so that support engagement is maintained.

12.3.13 Mechanical Equipment, Attachments, and Supports. Attachments and supports for mechanical equipment not covered in Sections 12.3.8 through 12.3.12 or 12.3.16 shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. In addition to their attachments and supports, such mechanical equipment designated as having an $I_p = 1.5$, itself, shall be designated to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section.

The seismic design of mechanical equipment, attachments, and their supports shall include analysis of the following: the dynamic effects of the equipment, its contents, and, when appropriate, its supports. The interaction between the equipment and the supporting structures, including other mechanical and electrical equipment, shall also be considered.

12.3.13.1 Mechanical Equipment. Mechanical equipment designated as having an $I_p = 1.5$ shall meet the following provisions.

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

1. For mechanical equipment constructed with ductile materials (e.g., steel, aluminum, or copper), 90% of the equipment material minimum specified yield strength.
2. For threaded connections in equipment constructed with ductile materials, 70% of the material minimum specified yield strength.
3. For mechanical equipment constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25% of the equipment material minimum tensile strength.
4. For threaded connections in equipment constructed with nonductile materials, 20% of the material minimum specified yield strength.

b. Provisions shall be made to mitigate seismic impact for equipment components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications).

c. The possibility for loadings imposed on the equipment by attached utility or service lines due to differential motions of points of support from separate structures shall be evaluated.

12.3.13.2 Attachments and Supports for Mechanical Equipment. Attachments and supports for mechanical equipment shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with a nationally recognized structural code such as, when constructed of steel, SBC 306.

b. Friction clips shall not be used for anchorage attachment.

c. Expansion anchors shall not be used for mechanical equipment rated over 10 hp (7.45 kW).

Exception: Undercut expansion anchors.

d. Drilled and grouted-in-place anchors for tensile load applications shall use either expansive cement or expansive epoxy grout.

e. Supports shall be specifically evaluated if weak-axis bending of light-gauge support steel is relied on for the seismic load path.

f. Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_p$. The intent is to prevent excessive movement and to avoid fracture of support springs and any nonductile components of the isolators.

g. Seismic supports shall be constructed so that support engagement is maintained.

12.3.14 **Electrical Equipment, Attachments, and Supports.** Attachments and supports for electrical equipment shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section. In addition to their attachments and supports, electrical equipment designated as having I_p = 1.5, itself, shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 and the additional provisions of this Section.

The seismic design of other electrical equipment shall include analysis of the following: the dynamic effects of the equipment, its contents, and when appropriate, its supports. The interaction between the equipment and the supporting structures, including other mechanical and electrical equipment, shall also be considered. Where conduit, cable trays, or similar electrical distribution components are attached to structures that could displace relative to one another and for seismically isolated structures where the conduit or cable trays cross the seismic isolation interface, the conduit or cable trays shall be designed to accommodate the seismic relative displacement specified in Section 12.1.4.

12.3.14.1 **Electrical Equipment.** Electrical equipment designated as having an I_p = 1.5 shall meet the following provisions:

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

1. For electrical equipment constructed with ductile material (e.g., steel, aluminum, or copper), 90% of the equipment material minimum specified yield strength.
2. For threaded connections in equipment constructed with ductile materials, 70% of the material minimum specified yield strength.
3. For electrical equipment constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25% of the equipment material minimum tensile strength.
4. For threaded connections in equipment constructed with nonductile materials, 20% of the material minimum specified yield strength.

b. Provisions shall be made to mitigate seismic impact for equipment components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low-temperature applications).

c. The possibility for loadings imposed on the equipment by attached utility or service lines due to differential motion of points of support from separate structures shall be evaluated.

d. Batteries on racks shall have wraparound restraints to ensure that the batteries will not fall off the rack. Spacers shall be used between restraints and cells to prevent damage to cases. Racks shall be evaluated for sufficient lateral and longitudinal load capacity.

e. Internal coils of dry type transformers shall be positively attached to their supporting substructure within the transformer enclosure.

f. Slide out components in electrical control panels shall have a latching mechanism to hold contents in place.

g. Structural design of electrical cabinets shall be in conformance with standards of the industry that are acceptable to the authority having jurisdiction. Large cutouts in the lower shear panel shall be specifically evaluated if an evaluation is not provided by the manufacturer.

h. The attachment of additional items weighing more than 450 N shall be specifically evaluated if not provided by the manufacturer.

12.3.14.2 Attachments and Supports for Electrical Equipment. Attachments and supports for electrical equipment shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with a nationally recognized structural code such as, when constructed of steel, SBC 306.

b. Friction clips shall not be used for anchorage attachment.

c. Oversized washers shall be used at bolted connections through the base sheet metal if the base is not reinforced with stiffeners.

d. Supports shall be specifically evaluated if weak-axis bending of light-gauge support steel is relied on for the seismic load path.

e. The supports for linear electrical equipment such as cable trays, conduit, and bus ducts shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4 only if any of the following situations apply:

- Supports are cantilevered up from the floor.
- Supports include bracing to limit deflection,
- Supports are constructed as rigid welded frames,
- Attachments into concrete utilize nonexpanding insets, shot pins, or cast iron embedments, or
- Attachments utilize spot welds, plug welds, or minimum size welds as defined by SBC 306.

f. Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction, and vertical restraints shall be provided where required to resist overturning. Isolator housings and restraints shall not be constructed of cast iron or other materials with limited ductility. (See additional design force requirements in Table 12.3.2.) A visco-elastic pad or similar material of appropriate thickness shall be used between the bumper and equipment item to limit the impact load.

12.3.15 Alternative Seismic Qualification Methods. As an alternative to the analysis methods implicit in the design methodology described above, equipment testing is an acceptable method to determine seismic capacity. Thus, adaptation of a nationally recognized standard for qualification by testing that is acceptable to the authority having jurisdiction is an acceptable alternative, so long as the equipment seismic capacity equals or exceeds the demand expressed in Sections 12.1.3 and 12.1.4.

12.3.16 Elevator Design Requirements.

12.3.16.1 **Reference Document.** Elevators shall meet the force and displacement provisions of Section 12.3.2 unless exempted by either Section 9.1.2.1 or Section 12.1. Elevators designed in accordance with the seismic provisions of (Ref. 12-1) shall be deemed to meet the seismic force requirements of this Section, except as modified below.

12.3.16.2 **Elevators and Hoistway Structural System.** Elevators and hoistway structural systems shall be designed to meet the force and displacement provisions of Sections 12.1.3 and 12.1.4.

12.3.16.3 **Elevator Machinery and Controller Supports and Attachments.** Elevator machinery and controller supports and attachments shall be designed to meet with the force and displacement provisions of Sections 12.1.3 and 12.1.4.

12.3.16.4 **Seismic Controls.** Seismic switches shall be provided for all elevators addressed by Section 12.3.16.1 including those meeting the requirements of the ASME reference, provided they operate with a speed of 46 m/min or greater.

Seismic switches shall provide an electrical signal indicating that structural motions are of such a magnitude that the operation of elevators may be impaired. Upon activation of the seismic switch, elevator operations shall conform to provisions of (Ref. 12-1) except as noted below. The seismic switch shall be located at or above the highest floor serviced by the elevators. The seismic switch shall have two horizontal perpendicular axes of sensitivity. Its trigger level shall be set to 30% of the acceleration of gravity.

In facilities where the loss of the use of an elevator is a life safety issue, the elevator shall only be used after the seismic switch has triggered provided that:

1. The elevator shall operate no faster than the service speed,
2. Before the elevator is occupied, it is operated from top to bottom and back to top to verify that it is operable, and
3. The individual putting the elevator back in service shall ride the elevator from top to bottom and back to top to verify acceptable performance.

12.3.16.5 **Retainer Plates.** Retainer plates are required at the top and bottom of the car and counterweight.

CHAPTER 13
SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURAL

SECTION 13.1
GENERAL

13.1.1 **Nonbuilding Structures.** Nonbuilding structures include all self-supporting structures that carry gravity loads and that may be required to resist the effects of earthquake, with the exception of building structures specifically excluded in Section 9.1.2, and other nonbuilding structures where specific seismic provisions have yet to be developed in Chapter 13. Nonbuilding structures supported by the earth or supported by other structures shall be designed and detailed to resist the minimum lateral forces specified in this section. Design shall conform to the applicable provisions of other sections as modified by this section. Foundation design shall comply with the requirements of Sections 10.1.1, 10.13 and Chapter 11.

13.1.2 **Design.** The design of nonbuilding structures shall provide sufficient stiffness, strength, and ductility consistent with the requirements specified herein for buildings to resist the effects of seismic ground motions as represented by these design forces:

a. Applicable strength and other design criteria shall be obtained from other portions of the seismic provisions of this Code or its reference documents.

b. When applicable strength and other design criteria are not contained in, or referenced by the seismic provisions of this Code, such criteria shall be obtained from reference documents. Where reference documents define acceptance criteria in terms of allowable stresses as opposed to strength, the design seismic forces shall be obtained from this section and used in combination with other loads as specified in Section 2.4 of this Code and used directly with allowable stresses specified in the reference documents. Detailing shall be in accordance with the reference documents.

13.1.3 **Structural Analysis Procedure Selection.** Structural analysis procedures for nonbuilding structures that are similar to buildings shall be selected in accordance with Section 10.6.

Nonbuilding structures that are not similar to buildings shall be designed using either the equivalent lateral force procedure in accordance with Section 10.9, the modal analysis procedure in accordance with Section 10.10. The linear response history analysis and the nonlinear response history analysis as per Section 10.14, or the procedure prescribed in the specific reference document.

SECTION 13.2
REFERENCE STANDARDS

13.2.1 **Consensus Standards.** The cited references (Ref. 13.2-1 through 13.2-22) are consensus standards and are to be considered part of the requirements of Chapters 9 through 12 to the extent referred to in Chapter 13.

13.2.2 **Accepted Standards.** The cited references (Ref. 13.2-23 through 13.2-31) are standards developed within the industry and represent acceptable procedures for design and construction.

13.2.3 **Industry Design Standards and Recommended Practice.** Table 13.2.3 is a cross-reference of consensus standards/accepted standards and the applicable nonbuilding structures.

SECTION 13.3 NONBUILDING STRUCTURES SUPPORTED BY OTHER STRUCTURES

Where nonbuilding structures identified in Table 13.4-2 are supported by other structures, and the nonbuilding structures are not part of the primary seismic force-resisting system, one of the following methods shall be used.

13.3.1 **Less Than 25 % Combined Weight Condition.** For the condition where the weight of the nonbuilding structure is less than 25 % of the combined weight of the nonbuilding structure and supporting structure, the design seismic forces of the nonbuilding structure shall be determined in accordance with Chapter 12 where the values of R_p and a_p shall be determined in accordance to Section 12.1. The supporting structure shall be designed in accordance with the requirements of Chapter 10 or 13.5 as appropriate with the weight of the nonbuilding structure considered in the determination of the seismic weight, W.

13.3.2 **Greater than or Equal to 25 % Combined Weight Condition.** For the condition where the weight of the nonbuilding structure is equal to or greater than 25 percent of the combined weight of the nonbuilding structure and supporting structure, an analysis combining the structural characteristics of both the nonbuilding structure and the supporting structures shall be performed to determine the seismic design forces as follows:

1. Where the nonbuilding structure has rigid component dynamic characteristics (as defined in Section 13.4.2), the nonbuilding structure shall be considered a rigid element with appropriate distribution of its seismic weight. The supporting structure shall be designed in accordance with the requirements of Chapter 10 or 13.5 as appropriate, and the R value of the combined system shall be permitted to be taken as the R value of the supporting structural system. The nonbuilding structure and attachments shall be designed for the forces using the procedures of Chapter 12 where the value of R_p shall be taken as equal to the R value of the nonbuilding structure as set forth in Table 13.4-2 and a_p shall be taken as 1.0.
2. Where the nonbuilding structure has non-rigid characteristics (as defined in Section 13.4.2), the nonbuilding structure and supporting structure shall be modeled together in a combined model with appropriate stiffness and seismic weight distributions. The combined structure shall be designed in accordance with Section 13.5 with the R value of the combined system taken as the R value of the nonbuilding structure with a maximum value of 3.0. The nonbuilding structure and attachments shall be designed for the forces determined for the nonbuilding structure in the combined analysis.

TABLE 13.2.3:
STANDARDS, INDUSTRY STANDARDS, AND REFERENCES

Application	Reference
Steel Storage Racks	RMI [25]
Piers and Wharves	NAVFAC R-939 [27], NAVFAC DM-25.1 [28]
Welded Steel Tanks for Water Storage	ACI 371R [4], ANSI/AWWA D100 [15], Army TM 5-809-10/ NAVFAC P-355/ Air Force AFM 88-3 Chapter 13 [29]
Welded Steel and Aluminum Tanks for Petroleum and Petrochemical Storage	API 620, 9th Edition, Addendum 3 [6], API 650, 10th Edition, Addendum 1 [7], ANSI/API 653, 2nd Edition, Addendum 4 [8], ASME B96.1 [14], Army TM 5-809-10/ NAVFAC P-355/ Air Force AFM 88-3 Chapter 13 [29]
Bolted Steel Tanks for Water Storage	ANSI/AWWA D103 [16], Army TM 5-809-10/ NAVFAC P-355/ Air Force AFM 88-3 Chapter 13 [29]
Bolted Steel Tanks for Petroleum and Petrochemical Storage	API Specification 12B, 14th Edition [10], Army TM 5-809-10/ NAVFAC P-355/ Air Force AFM 88-3 Chapter 13 [29]
Concrete Tanks for Water Storage	ACI 350.3/350.3R [3], ANSI/AWWA D110 [17], ANSI/AWWA D115 [18], Army TM 5-809-10/ NAVFAC P-355/ Air Force AFM 88-3 Chapter 13 [29]
Pressure Vessels	ASME [11]
Refrigerated Liquids Storage:	
Liquid Oxygen, Nitrogen, and Argon	ANSI/NFPA 50 [30], GGA [31]
Liquefied Natural Gas (LNG)	ANSI/NFPA 59A [22], DOT Title 49CFR Part 193 [26]
LPG (Propane, Butane, etc.)	ANSI/API 2510, 7th Edition [9], ANSI/NFPA 30 [19], ANSI/NFPA 58 [20], ANSI/NFPA 59 [21]
Ammonia	ANSI K61.1 [5]
Concrete Silos and Stacking Tubes	ACI 313 [2]
Impoundment Dikes and Walls:	
Hazardous Materials	ANSI K61.1 [5]
Flammable Materials	ANSI/NFPA 30 [19]
Liquefied Natural Gas	ANSI/NFPA 59A [22], DOT Title 49CFR Part 193 [26]
Gas Transmission and Distribution Piping Systems	ASME B31.8 [13]
Cast-in-Place Concrete Stacks and Chimneys	ACI 307 [1]
Steel Stacks and Chimneys	ASME STS-1 [12]
Guyed Steel Stacks and Chimneys	ASME STS-1 [12]
Brick Masonry Liners for Stacks and Chimneys	ASTM C1298 [24]
Amusement Structures	ASTM F1159 [23]

13.3.3 **Architectural, Mechanical, and Electrical Components.** Architectural, mechanical, and electrical components supported by nonbuilding structures shall be designed in accordance with Chapter 12 of this Code.

SECTION 13.4
STRUCTURAL DESIGN REQUIREMENTS

13.4.1 **Design Basis.** Nonbuilding structures having specific seismic design criteria established in reference documents shall be designed using the standards as amended herein. When reference documents are not cited herein, nonbuilding structures shall be designed in compliance with Sections 13.5 and 13.6 to resist minimum seismic lateral forces that are not less than the requirements of Section 10.9 with the following additions and exceptions:

1. The basic seismic-force-resisting system shall be selected as follows:

a. For nonbuilding structures similar to buildings, a system shall be selected from among the types indicated in Table 10.2 or Table 13.4-1 subject to the system limitations and height limits, based on seismic design category indicated in the table. The appropriate values of R, Ω_o, and C_d indicated in 13.4-1 shall be used in determining the base shear, element design forces, and design story drift as indicated in this Code.

b. For nonbuilding structures not similar to buildings, a system shall be selected from among the types indicated in Table 13.4-2 subject to the system limitations and height limits, based on seismic design category indicated in the table. The appropriate values of R, Ω_o, and C_d indicated in Table 13.4-2 shall be used in determining the base shear, element design forces, and design story drift as indicated in this Code.

c. Where neither Table 13.4-1 nor Table 13.4-2 contains an appropriate entry, applicable strength and other design criteria shall be obtained from a reference document that is applicable to the specific type of nonbuilding structure. Design and detailing requirements shall comply with the reference document.

2. For nonbuilding systems that have an R value provided in Table 13.4-2, the seismic response coefficient (C_s) shall not be taken less than:

$$C_s = 0.03 \qquad \textbf{(Eq. 13.4-1)}$$

Exception:
Tanks and vessels that are designed to the following reference documents AWWA 13.2-16, D103-97, Appendix E and Appendix L as modified by this Code, shall be subject to the larger of the minimum base shear values defined by the reference document or the following equation:

$$C_s = 0.01 \qquad \textbf{(Eq. 13.4-2)}$$

Minimum base shear requirements need not apply to the convective (sloshing) component of liquid in tanks.

3. The importance factor, I, shall be as set forth in Section 13.4.1.1.

4. The vertical distribution of the lateral seismic forces in nonbuilding structures covered by this section shall be determined:

a. Using the requirements of Section 10.9.4, or

b. Using the procedures of Section 10.10, or

c. In accordance with reference document applicable to the specific nonbuilding structure.

5. For nonbuilding structural systems containing liquids, gases, and granular solids supported at the base as defined in Section 13.7.1, the minimum seismic design force shall not be less than that required by the reference document for the specific system.

6. Irregular structures per Section 10.3.2 at sites where the S_{DS} is greater than or equal to 0.50 and that cannot be modeled as a single mass shall use the procedures of Section 10.10.

7. Where a reference document provides a basis for the earthquake resistant design of a particular type of nonbuilding structure covered by Chapter 13, such a standard shall not be used unless the following limitations are met:

a. The seismic ground accelerations, and seismic coefficients, shall be in conformance with the requirements of Section 9.4.

b. The values for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the base shear value and overturning moment, each adjusted for the effects of soil structure interaction that is obtained from a substantial analysis using reference documents.

8. The base shear is permitted to be reduced in accordance with Section 10.14 to account for the effects of soil-structure interaction. In no case shall the reduced base shear be less than 0.7V.

9. Unless otherwise noted in Chapter 13, the effects on the nonbuilding structure due to gravity loads and seismic forces shall be combined in accordance with the factored load combinations as presented in Section 2.3.

10. Where specifically required by Chapter 13, the design seismic force on nonbuilding structures shall be as defined in Section 10.4.1.

13.4.1.1 **Importance Factor.** The importance factor (I) and Occupancy Category for nonbuilding structures are based on the relative hazard of the contents and the function. The value of I shall be the largest value determined by the following:

a. Applicable reference document listed in Section 13.2.

b. The largest value as selected from Table 9.5.

c. As specified elsewhere in Chapter 13.

13.4.2 **Rigid Nonbuilding Structures.** Nonbuilding structures that have a fundamental period, T, less than 0.06 sec, including their anchorages, shall be designed for the lateral force obtained from the following:

$$V = 0.30\, S_{DS}\, WI \qquad \textbf{(Eq. 13.4-3)}$$

Where

V = the total design lateral seismic base shear force applied to a nonbuilding structure

S_{DS} = the site design response acceleration as determined from Section 9.4.4

W = nonbuilding structure operating weight

I = the importance factor determined in accordance with Section 13.4.1.1.

The force shall be distributed with height in accordance with Section 10.9.4.

13.4.3 **Loads.** The weight W for nonbuilding structures shall include all dead load as defined for structures in Section 10.7: For purposes of calculating design seismic forces in nonbuilding structures, W also shall include all normal operating contents for items such as tanks, vessels, bins, hoppers, and the contents of piping.

13.4.4 **Fundamental Period.** The fundamental period of the nonbuilding structure shall be determined using the structural properties and deformation characteristics of the resisting elements in a properly substantiated analysis as indicated in Section 10.9.3. Alternatively, the fundamental period T may be computed from the following equation:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} f_i \delta_i}} \qquad \textbf{(Eq. 13.4-4)}$$

The values of f_i represent any lateral force distribution in accordance with the principles of structural mechanics. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i.

Equations 10.9.3.2-1, 10.9.3.2-1a, 10.9.3.2-2 and 10.9.3.2-3 shall not be used for determining the period of a nonbuilding structure.

13.4.5 **Drift Limitations.** The drift limitations of Section 10.12 need not apply to nonbuilding structures if a rational analysis indicates they can be exceeded without adversely affecting structural stability or attached or interconnected components and elements such as walkways and piping. P-delta effects shall be considered when critical to the function or stability of the structure.

13.4.6 **Materials Requirements.** The requirements regarding specific materials in Chapter 11 shall be applicable unless specifically exempted in Chapter 13.

13.4.7 **Deflection Limits and Structure Separation.** Deflection limits and structure separation shall be determined in accordance with this Code unless specifically amended in Chapter 13.

SECTION 13.5
NONBUILDING STRUCTURES SIMILAR TO BUILDINGS

13.5.1 **General.** Nonbuilding structures similar to buildings as defined in Section 9.2 shall be designed in accordance with this Code as modified by this section and the specific reference documents.

This general category of nonbuilding structures shall be designed in accordance with the seismic provisions of this Code and 13.4.

The combination of load effects, E, shall be determined in accordance with Section 10.4.

13.5.2 Pipe Racks

13.5.2.1 **Design Basis.** In addition to the provisions of 13.5.1, pipe racks supported at the base of the structure shall be designed to meet the force requirements of Section 10.9 or 10.10.

Displacements of the pipe rack and potential for interaction effects (pounding of the piping system) shall be considered using the amplified deflections obtained from the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad \textbf{(Eq.13.5-1)}$$

where

C_d = deflection amplification factor in Table 13.4-1.

δ_{xe} = deflections determined using the prescribed seismic design forces of this Code.

I = importance factor determined in accordance with Section 13.4.1.1.

See Section 12.3 for the design of piping systems and their attachments. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

13.5.3 **Steel Storage Racks.** In addition to the provisions of 13.5.1, steel storage racks shall be designed in accordance with the provisions of Sections 13.5.3.1 through 13.5.3.4. Alternatively, steel storage racks shall be permitted to be designed in accordance with the method defined in Section 2.7 "Earthquake Forces" of where the following changes are included:

1. The values of C_a and C_v used shall equal $S_{DS}/2.5$ and S_{D1}, respectively, where S_{DS} and S_{D1} are determined in accordance with Section 9.4.4 of this Code.
2. The importance factor for storage racks in structures open to the public, such as warehouse retail stores, shall be taken equal to 1.5.
3. For storage racks located at or below grade, the value of C_s used shall not be less than 0.14 S_{DS}. For storage racks located above grade, the value of C_s used shall not be less than the value for F_p determined in accordance with Section 12.1.3 of this Code where R_p is taken as equal to R from and a_p is taken as equal to 2.5.

13.5.3.1 **General Requirements.** Steel storage racks shall satisfy the force requirements of this section.

Exception: Steel storage racks supported at the base are permitted to be designed as structures with an R of 4, provided that the seismic requirements of this Code are met. Higher values of R are permitted to be used when the detailing requirements of reference documents listed in 11.1 are met. The importance factor for storage racks in structures open to the public, such as warehouse retail stores, shall be taken equal to 1.5.

13.5.3.2 **Operating Weight.** Steel storage racks shall be designed for each of the following conditions of operating weight W or W_p.

a. Weight of the rack plus every storage level loaded to 67 percent of its rated load capacity.

b. Weight of the rack plus the highest storage level only loaded to 100 percent of its rated load capacity. The design shall consider the actual height of the center of mass of each storage load component.

13.5.3.3 **Vertical Distribution of Seismic Forces.** For all steel storage racks, the vertical distribution of seismic forces shall be as specified in Section 10.9.4 and in accordance with the following:

a. The base shear, V, of the typical structure shall be the base shear of the steel storage rack when loaded in accordance with Section 13.5.3.2.

b. The base of the structure shall be the floor supporting the steel storage rack. Each steel storage level of the rack shall be treated as a level of the structure with heights hi and hx measured from the base of the structure.

c. The factor k shall be permitted to be taken as 1.0.

13.5.3.4 **Seismic Displacements.** Steel storage rack installations shall accommodate the seismic displacement of the storage racks and their contents relative to all adjacent or attached components and elements. The assumed total relative displacement for storage racks shall be not less than 5 percent of the height above the base.

Table 13.4-1:
Seismic Coefficients for Nonbuilding Structures Similar to Buildings.

Nonbuilding Structure Type	R	Ω_o	C_d	Structural System and Height Limits (m)[a]		
				A&B	C	D
Building frame system:						
Special steel concentrically braced frames	5	2	5	NL	NL	50
Ordinary steel concentrically braced frame	4	2	4.5	NL	NL	11
Moment resisting frame system:						
Special steel moment frames	6	3	5.5	NL	NL	NL
Special reinforced concrete moment frames	6	3	5.5	NL	NL	NL
Intermediate steel moment frames	3	3	4	NL	NL	11
Intermediate reinforced concrete moment frames	3	3	4.5	NL	NL	NP
Ordinary moment frames of steel	2.5	3	3	NL	NL	NP[c,d]
Ordinary reinforced concrete moment frames	2	3	2.5	NL	NP	NP

Notes:
a NL = no limit and NP = not permitted. Height shall be measured from the base.
b Steel ordinary braced frames are permitted in pipe racks up to 20 m.
c Steel ordinary moment frames and intermediate moment frames are permitted in pipe racks up to a height of 20 m where the moment joints of field connections are constructed of bolted end plates.
d Steel ordinary moment frames and intermediate moment frames are permitted in pipe racks up to a height of 11 m.

Table 13.4-2:
Seismic Coefficients for Nonbuilding Structures NOT Similar to Buildings.

Nonbuilding Structure Type	R	Ω_o	C_d	Structural System and Height Limits (m)[a]		
				A&B	C	D
Steel Storage Racks	3	2	3.5	NL	NL	NL
Elevated tanks, vessels, bins, or hoppers:						
On symmetrically braced legs	2	2^b	2.5	NL	NL	50
On unbraced legs or asymmetrically braced legs	1.5	2^b	2.5	NL	NL	30
Single pedestal or skirt supported	1.5	2^b	2	NL	NL	NL
- welded steel	1.5	2^b	2	NL	NL	NL
- prestressed or reinforced concrete	1.5	2^b	2	NL	NL	NL
Horizontal, saddle supported welded steel vessels.	2	2^b	2.5	NL	NL	NL
Tanks or vessels supported on structural towers similar to buildings	Use values for the appropriate structure type in the categories for building frame systems and moment resisting frame systems listed in Table 13.4-2.					
Flat bottom ground supported tanks:						
Steel or fiber-reinforced plastics:						
Mechanically anchored	2	2^b	2.5	NL	NL	NL
Self-anchored	2	2^b	2	NL	NL	NL
Reinforced or prestressed concrete:						
Reinforced nonsliding base	1.5	2^b	2	NL	NL	NL
Anchored flexible base	2.5	2^b	2	NL	NL	NL
Unanchored and unconstrained flexible base	1	1.5^b	1.5	NL	NL	NL
All other	1	1.5^b	1.5	NL	NL	NL
Cast-in-place concrete soils, stacks, and chimneys having walls continuous to the foundation	2	1.75	3	NL	NL	NL
All other reinforced masonry structures not similar to buildings	1.5	2	2.5	NL	NL	NL
All other nonreinforced masonry structures not similar to buildings	0.8	2	1.5	NL	NL	15
All other steel and reinforced concrete distribution mass cantilever structures not covered herein including stacks, chimneys, soils, and skirt-supported vertical vessels that are not similar to buildings	2	2	2.5	NL	NL	NL

Table 13.4-2:
Seismic Coefficients for Nonbuilding Structures NOT Similar to Buildings (Contd…..).

Nonbuilding Structure Type		R	Ω_o	C_d	Structural System and Height Limits (m)[a]		
					A&B	C	D
Trussed towers (freestanding or guyed), guyed stacks and chimneys		2	2	2.5	NL	NL	NL
Cooling towers:							
Concrete or steel		2.5	1.75	3	NL	NL	NL
Telecommunication towers:							
Truss:	Steel	2	1.5	3	NL	NL	NL
Pole:	Steel	1	1.5	1.5	NL	NL	NL
	Concrete	1	1.5	1.5	NL	NL	NL
Frame:	Steel	2	1.5	1.5	NL	NL	NL
	Concrete	1.5	1.5	1.5	NL	NL	NL
Amusement structure and monuments		1.5	2	2	NL	NL	NL
Inverted pendulum type structures (except elevated tanks, vessels, bins and hoppers)		1.5	2	2	NL	NL	NL
Signs and billboards		2.5	1.75	3	NL	NL	NL
All other self-supporting structures, tanks, or vessels not covered above or by reference standards that are similar to buildings.		1	2	2.5	NL	NL	15

Notes:
a NL = no limit and NP = not permitted. Height shall be measured from the base.
b See Section 13.7.3 a. for the application of the over-strength factors, Ω_o , for tank and vessels.

13.5.4 Electrical Power Generating Facilities

13.5.4.1 **General.** Electrical power generating facilities are power plants that generate electricity by steam turbines, combustion turbines, diesel generators, or similar turbo machinery.

13.5.4.2 **Design Basis.** In addition to the provisions of 13.5.1, electrical power generating facilities shall be designed using this Code and the appropriate factors contained in Section 13.4.

13.5.5 Structural Towers for Tanks and Vessels

13.5.5.1 **General.** In addition to the provisions of 13.5.1, structural towers which support tanks and vessels shall be designed to meet the provisions of Section 13.3. In addition, the following special considerations shall be included:

a. The distribution of the lateral base shear from the tank or vessel onto the supporting structure shall consider the relative stiffness of the tank and resisting structural elements.

b. The distribution of the vertical reactions from the tank or vessel onto the supporting structure shall consider the relative stiffness of the tank and resisting structural elements. When the tank or vessel is supported on grillage beams, the calculated vertical reaction due to weight and overturning shall be increased at least 20 percent to account for non-uniform support. The grillage beam and vessel attachment shall be designed for this increased design value.

c. Seismic displacements of the tank and vessel shall consider the deformation of the support structure when determining P-delta effects or evaluating required clearances to prevent pounding of the tank on the structure.

13.5.6 Piers and Wharves

13.5.6.1 **General.** Piers and wharves are structures located in waterfront areas that project into a body of water or parallel the shore line.

13.5.6.2 **Design Basis.** In addition to the provisions of 13.5.1, piers and wharves that are accessible to the general public, such as cruise ship terminals and piers with retail or commercial offices or restaurants, shall be designed to comply with this Code.

The design shall account for the effects of liquefaction and soil failure collapse mechanisms, as well as consider all applicable marine loading combinations, such as mooring, berthing, wave and current on piers and wharves as required. Structural detailing shall consider the effects of the marine environment.

SECTION 13.6 GENERAL REQUIREMENTS FOR NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS

Nonbuilding structures that do not have lateral and vertical seismic force-resisting systems that are similar to buildings shall be designed as modified by this section and the specific reference documents. Loads and load distributions shall not be less demanding than those determined in this Code. The combination of load effects, E, shall be determined in accordance with Section 10.4.
Exception: The redundancy factor, ρ, per Section. 10.3.3 shall be taken as 1.

13.6.1 **Earth-Retaining Structures.** This section applies to all earth-retaining walls. The applied seismic forces shall be determined in accordance with Section 10.13.5.1 with a geotechnical analysis prepared by a registered design professional.
The occupancy category shall be determined by the proximity of the retaining wall to buildings and other structures. If failure of the retaining wall would affect an adjacent structure, the occupancy category shall not be less than that of the adjacent structure, as determined from Table 1.6-1. Earth-retaining walls are permitted to be designed for seismic loads as either yielding or non-yielding walls. Cantilevered reinforced concrete retaining walls shall be assumed to be yielding walls and shall be designed as simple flexural wall elements.

13.6.2 **Stacks and Chimneys.** Stacks and chimneys shall be permitted to be either lined or unlined, and shall be constructed from concrete, steel, or masonry. Steel stacks, concrete stacks, steel chimneys, concrete chimneys, and liners shall be designed to resist seismic lateral forces determined from a substantiated analysis using reference documents. Interaction of the stack or chimney with the liners shall be considered. A minimum separation shall be provided between the liner and chimney equal to C_d times the calculated differential lateral drift.

13.6.3 **Amusement Structures.** Amusement structures are permanently fixed structures constructed primarily for the conveyance and entertainment of people. Amusement structures shall be designed to resist seismic lateral forces determined from a substantiated analysis using reference documents.

13.6.4 **Special Hydraulic Structures.** Special hydraulic structures are structures that are contained inside liquid containing structures. These structures are exposed to liquids on both wall surfaces at the same head elevation under normal operating conditions. Special hydraulic structures are subjected to out-of-plane forces only during an earthquake when the structure is subjected to differential hydrodynamic fluid forces. Examples of special hydraulic structures include: separation walls, baffle walls, weirs, and other similar structures.

13.6.4.1 **Design Basis.** Special hydraulic structures shall be designed for out-of-phase movement of the fluid. Unbalanced forces from the motion of the liquid must be applied simultaneously "in front of" and "behind" these elements.

Structures subject to hydrodynamic pressures induced by earthquakes shall be designed for rigid body and sloshing liquid forces and their own inertia force. The height of sloshing shall be determined and compared to the freeboard height of the structure.

Interior elements, such as baffles or roof supports, also shall be designed for the effects of unbalanced forces and sloshing.

13.6.5 **Secondary Containment Systems.** Secondary containment systems such as impoundment dikes and walls shall meet the requirements of the applicable standards for tanks and vessels and the building official. Secondary containment systems shall be designed to withstand the effects of the maximum considered earthquake ground motion when empty and two-thirds of the maximum considered earthquake ground motion when full including all hydrodynamic forces as determined in accordance with the procedures of Section 9.4. When determined by the risk assessment required by Section 1.5.2 or by the authority having jurisdiction that the site may be subject to aftershocks of the same magnitude as the maximum considered motion, secondary containment systems shall be designed to withstand the effects of the maximum considered earthquake ground motion when full including all hydrodynamic forces as determined in accordance with the procedures of Section 9.4.

13.6.5.1 **Freeboard.** Sloshing of the liquid within the secondary containment area shall be considered in determining the height of the impound. Where the primary containment has not been designed with a reduction in the structure category (i.e., no reduction in importance factor I) as permitted by Section 1.5.2, no freeboard provision is required. Where the primary containment has been designed for a reduced structure category (i.e., importance factor I reduced) as permitted by Section 1.5.2, a minimum freeboard, δ_S, shall be provided where

$$\delta_s = 0.50\ DS_{ac} \qquad \textbf{(Eq. 13.6-1)}$$

where S_{ac} is the spectral acceleration of the convective component and is determined according to the procedures of Section 9.4 using 0.5 percent damping. For circular impoundment dikes, D shall be taken as the diameter of the impoundment dike. For rectangular impoundment dikes, D shall be taken as the longer plan dimension of the impoundment dike.

13.6.6 **Telecommunication Towers.** Self-supporting and guyed telecommunication towers shall be designed to resist seismic lateral forces determined from a substantiated analysis using reference documents.

SECTION 13.7
TANKS AND VESSELS

13.7.1 **General.** This section applies to all tanks, vessels, bins and silos, and similar containers storing liquids, gases, and granular solids supported at the base (hereafter referred to generically as tanks and vessels). Tanks and vessels covered herein include reinforced concrete, prestressed concrete, steel, aluminum, and fiber-reinforced plastic materials. Tanks supported on elevated levels in buildings shall be designed in accordance with Section 13.3.

13.7.2 **Design Basis.** Tanks and vessels storing liquids, gases, and granular solids shall be designed in accordance with this Code and shall be designed to meet the requirements of the applicable reference documents listed in Table 13.2. Resistance to seismic forces shall be determined from a substantiated analysis based on the applicable reference documents listed in Table 13.2.

a. Damping for the convective (sloshing) force component shall be taken as 0.5 percent

b. Impulsive and convective components shall be combined by the direct sum or the square root of the sum of the squares (SRSS) method when the modal periods are separated. If significant modal coupling may occur, the complete quadratic combination (CQC) method shall be used.

c. Vertical earthquake forces shall be considered in accordance with the applicable reference document. If the reference document permits the user the option of including or excluding the vertical earthquake force, to comply with this Code, it shall be included. For tanks and vessels not covered by a reference document, the forces due to the vertical acceleration shall be defined as follows:

(1) Hydrodynamic vertical and lateral forces in tank walls: The increase in hydrostatic pressures due to the vertical excitation of the contained liquid shall correspond to an effective increase in density, γ_L, of the stored liquid equal to 0.2 $S_{DS}I\gamma_L$.

(2) Hydrodynamic hoop forces in cylindrical tank walls: In a cylindrical tank wall, the hoop force per unit height, N_h, at level y from the base, associated with the vertical excitation of the contained liquid, shall be computed in accordance with Equation 15.7-1.

$$N_h = 0.2\, S_{DS}I x\ \gamma_L\, (H_L - y)\, (D_i/2) \qquad \textbf{(Eq. 13.7-1)}$$

where

D_i = inside tank diameter (m)

H_L = liquid height inside the tank (m).

y = distance from base of the tank to level being investigated (m).

γ_L = unit weight of stored liquid (kN/m^3)

(3) Vertical inertia forces in cylindrical and rectangular tank walls: Vertical inertia forces associated with the vertical acceleration of the structure itself shall be taken equal to 0.2 $S_{DS}IW$.

13.7.3 **Strength and Ductility.** Structural components and members that are part of the lateral support system shall be designed to provide the following:

a. Connections and attachments for anchorage and other lateral force-resisting components shall be designed to develop the strength of the anchor (e.g., minimum published yield strength, F_y in direct tension, plastic bending moment), or Ω_o times the calculated element design force. The over-strength provisions of Section 10.4.1, and the Ω_o values tabulated in Table 13.4-2, do not apply to the design of walls, including interior walls, of tanks or vessels.

b. Penetrations, manholes, and openings in shell components shall be designed to maintain the strength and stability of the shell to carry tensile and compressive membrane shell forces.

c. Support towers for tanks and vessels with irregular bracing, unbraced panels, asymmetric bracing, or concentrated masses shall be designed using the provisions of Section 10.3.2 for irregular structures. Support towers using chevron or eccentric braced framing shall comply with the seismic requirements of this Code requirement. Support towers using tension only bracing shall be designed such that the full cross-section of the tension element can yield during overload conditions.

d. In support towers for tanks and vessels, compression struts that resist the reaction forces from tension braces shall be designed to resist the lesser of the yield load of the brace (Ag Fy), or Ω_o times the calculated tension load in the brace.

e. The vessel stiffness relative to the support system (foundation, support tower, skirt, etc.) shall be considered in determining forces in the vessel, the resisting components and the connections.

f. For concrete liquid-containing structures, system ductility, and energy dissipation under unfactored loads shall not be allowed to be achieved by inelastic deformations to such a degree as to jeopardize the serviceability of the structure. Stiffness degradation and energy dissipation shall be allowed to be obtained either through limited microcracking, or by means of lateral force resistance mechanisms that dissipate energy without damaging the structure.

13.7.4 **Flexibility of Piping Attachments.** Design of piping systems connected to tanks and vessels shall consider the potential movement of the connection points during earthquakes and provide sufficient flexibility to avoid release of the product by failure of the piping system. The piping system and supports shall be designed so as not to impart significant mechanical loading on the attachment to the tank or vessel shell. Mechanical devices which add flexibility such as bellows, expansion joints, and other flexible apparatus may be used when they are designed for seismic displacements and defined operating pressure.

Unless otherwise calculated, the minimum displacements in Table 13.7-1 shall be assumed. For attachment points located above the support or foundation elevation, the displacements in Table 13.7-1 shall be increased to account for drift of the tank or vessel relative to the base of support.

The piping system and tank connection shall also be designed to tolerate C_d times the displacements given in Table 13.7-1 without rupture, although permanent deformations and inelastic behavior in the piping supports and tank shell is permitted. For attachment points located above the support or foundation elevation, the displacements in Table 13.7-1 shall be increased to account for drift of the tank or vessel.

The values given in Table 13.7-1 do not include the influence of relative movements of the foundation and piping anchorage points due to foundation movements (e.g., settlement, seismic displacements). The effects of the foundation movements shall be included in the piping system design including the determination of the mechanical loading on the tank or vessel, and the total displacement capacity of the mechanical devices intended to add flexibility.

The anchorage ratio, J, for self-anchored tanks shall comply with the criteria shown in Table 13.7-2 and is defined as:

$$J = \frac{M_{rw}}{D^2\left(w_t + w_a\right)} \qquad \textbf{(Eq. 13.7-2)}$$

where

$$w_t = \frac{W_s}{\pi D} + w_r \qquad \textbf{(Eq. 13.7-3)}$$

w_r = roof load acting on the shell in pounds per foot of shell circumference. Only permanent roof loads shall be included. Roof live load shall not be included.

w_a = maximum weight of the tank contents that may be used to resist the shell overturning moment in pounds per foot of shell circumference. Usually consists of an annulus of liquid limited by the bending strength of the tank bottom or annular plate.

M_{rw} = the overturning moment applied at the bottom of the shell due to the seismic design loads in foot-pounds (also known as the ring-wall moment)

D = tank diameter in feet

W_s = total weight of tank shell in pounds

13.7.5 **Anchorage.** Tanks and vessels at grade shall be permitted to be designed without anchorage when they meet the requirements for unanchored tanks in reference documents. Tanks and vessels supported above grade on structural towers or building structures shall be anchored to the supporting structure.

The following special detailing requirements shall apply to steel tank anchor bolts in seismic regions where $S_{DS} > 0.5$, or where the structure is classified as Occupancy Category IV.

a. Hooked anchor bolts (L or J shaped embedded bolts) or other anchorage systems based solely on bond or mechanical friction shall not be used when $S_{DS} \geq 0.33$. Post-installed anchors may be used provided that testing validates their ability to develop yield load in the anchor under cyclic loads in cracked concrete.

b. When anchorage is required, the anchor embedment into the foundation shall be designed to develop the minimum specified yield strength of the anchor.

Table 13.7-1:
Minimum Design Displacements for Piping Attachments.

Condition	Displacements (mm)
Mechanically-Anchored Tanks and Vessels	
Upward vertical displacement relative to support or foundation	25
Downward vertical displacement relative to support or foundation	12
Range of horizontal displacement (radial and tangential) relative to support or foundation	12
Self-anchored Tanks or Vessels (at grade)	
Upward vertical displacement relative to support or foundation	
If designed in accordance with a reference document as modified by this Code:	
Anchorage ratio less than or equal to 0.785 (indicates no uplift)	25
Anchorage ratio greater than 0.785 (indicates uplift)	100
If designed for seismic loads in accordance with this Code but not covered by a reference document:	
For tanks and vessels with a diameter less than 12 m	200
For tanks and vessels with a diameter equal to or greater than 12 m	300
Downward vertical displacement relative to support or foundation	
For tanks with a ring-wall/mat foundation	12
For tanks with a beam foundation	25
Range of horizontal displacement (radial and tangential) relative to support or foundation	50

Table 13.7-2: Anchorage Ratio

J Anchorage Ratio	Criteria
$J < 0.785$	No uplift under the design seismic overturning moment. The tank is self anchored.
$0.785 < J < 1.54$	Tank is uplifting, but the tank is stable for the design load providing the shell compression requirements are satisfied. Tank is self-anchored.
$J > 1.54$	Tank is not stable and shall be mechanical mechanically anchored for the design load.

13.7.6 Ground-Supported Storage Tanks for Liquids

13.7.6.1 General. Ground-supported, flat bottom tanks storing liquids shall be designed to resist the seismic forces calculated using one of the following procedures:

a. The base shear and overturning moment calculated as if tank and the entire contents are a rigid mass system per Section 13.4.2 of this Code, or

b. Tanks or vessels storing liquids in Occupancy Category IV, or with a diameter greater than 6 m shall be designed to consider the hydrodynamic pressures of the liquid in determining the equivalent lateral forces and lateral force distribution per the applicable reference documents listed in

Section 13.2 and the requirements of Section 13.7 of this Code requirement.

c. The force and displacement provisions of Section 13.4 of this Code requirement.

The design of tanks storing liquids shall consider the impulsive and convective (sloshing) effects and their consequences on the tank, foundation, and attached elements. The impulsive component corresponds to the high frequency amplified response to the lateral ground motion of the tank roof, shell, and portion of the contents that moves in unison with the shell. The convective component corresponds to the low frequency amplified response of the contents in the fundamental sloshing mode. Damping for the convective component shall be 0.5 percent for the sloshing liquid unless otherwise defined by the reference document. The following definitions shall apply:

D_i = inside diameter of tank or vessel

H_L = design liquid height inside tank or vessel

L = inside length of a rectangular tank, parallel to the direction of the earthquake force being investigated

N_h = hydrodynamic hoop force per unit height in the wall of a cylindrical tank or vessel

T_c = natural period of the first (convective) mode of sloshing

T_i = fundamental period of the tank structure and impulsive component of the content

V_i = base shear due to impulsive component from weight of tank and contents

V_c = base shear due to the convective component of the effective sloshing mass

y = distance from base of the tank to level being investigated.

γ_L = unit weight of stored liquid

The seismic base shear is the combination of the impulsive and convective components:

$$V = V_i + V_c \qquad \textbf{(Eq. 13.7-4)}$$

where

$$V_i = \frac{S_{ai} W_i}{(R/I)} \qquad \textbf{(Eq. 13.7-5)}$$

$$V_c = \frac{S_{ac} I}{1.5} W_c \qquad \textbf{(Eq. 13.7-6)}$$

S_{ai} = the spectral acceleration as a multiplier of gravity including the site impulsive components at period T_i and 5 percent damping

For $T_i \leq T_s$:

$$S_{ai} = S_{DS} \qquad \textbf{(Eq. 13.7-7)}$$

For $T_s < T_i \leq T_L$

$$S_{ai} = \frac{S_{D1}}{T_i} \qquad \textbf{(Eq. 13.7-8)}$$

For $T_i > T_L$

$$S_{ai} = \frac{S_{D1} T_L}{T_i^2} \qquad \textbf{(Eq. 13.7-9)}$$

Notes:

a. When a reference document is used in which the spectral acceleration for the tank shell, and the impulsive component of the liquid is independent of T_i, then $S_{ai} = S_{DS}$.

b. Eq. 13.7-8 and Eq. 13.7-9 shall not be less than the minimum values required in Section 13.4.1 Item 2 multiplied by R/I.

c. For tanks in Occupancy Category IV, the value of the importance factor (I) used for freeboard determination only shall be taken as 1.0.

d. For tanks in Occupancy Categories I, II and III, the value of T_L used for freeboard determination shall be permitted to be set equal to 4 seconds. The value of the importance factor (I) used for freeboard determination for tanks in Occupancy Categories I, II and III shall be the value determined from Table 9.5.

e. Impulsive and convective seismic forces for tanks are permitted to be combined using the square root of the sum of the squares (SRSS) method in lieu of the direct sum method shown in Section 13.7.6 and its related subsections.

S_{ac} = the spectral acceleration of the sloshing liquid based on the sloshing period T_c and 0.5 percent damping

For $T_c \le T_L$:

$$S_{ac} = \frac{1.5\, S_{D1}}{T_c} \qquad \textbf{(Eq. 13.7-10)}$$

For $T_c > T_L$:

$$S_{ac} = \frac{1.5\, S_{D1} T_L}{T_c^2} \qquad \textbf{(Eq. 13.7-11)}$$

where

$$T_c = 2\pi \sqrt{\frac{D}{3.68\, g\, tan\, h\left(\frac{3.68H}{D}\right)}} \qquad \textbf{(Eq. 13.7-12)}$$

and where

D = the tank diameter in meters, H = liquid height in m, and g = acceleration due to gravity in consistent units

W_i = impulsive weight (impulsive component of liquid, roof and equipment, shell, bottom, and internal components)

W_c = the portion of the liquid weight sloshing

13.7.6.1.1 **Distribution of Hydrodynamic and Inertia Forces.** Unless otherwise required by the appropriate reference document listed in Table 13.2, the method given in ACI 530.3-01 may be used to determine the vertical and horizontal distribution of the hydrodynamic and inertia forces on the walls of circular and rectangular tanks.

13.7.6.1.2 **Sloshing.** Sloshing of the stored liquid shall be taken into account in the seismic design of tanks and vessels in accordance with the following provisions:

a. The height of the sloshing wave, δ_s, shall be computed using Eq. 13.7-13 as follows:

$$\delta_s = 0.5\, D_i I S_{ac} \qquad \textbf{(Eq. 13.7-13)}$$

For cylindrical tanks, D_i shall be the inside diameter of the tank; for rectangular tanks, the term D_i shall be replaced by the longer longitudinal plan dimension of the tank, L.

b. The effects of sloshing shall be accommodated by means of one of the following:

1. A minimum freeboard in accordance with Table 13.7-3.
2. A roof and supporting structure designed to contain the sloshing liquid in accordance with subsection c below.
3. For open-top tanks or vessels only, an overflow spillway around the tank or vessel perimeter.

c. If the sloshing is restricted because the freeboard is less than the computed sloshing height, then the roof and supporting structure shall be designed for an equivalent hydrostatic head equal to the computed sloshing height less the freeboard. In addition, the design of the tank shall use the confined portion of the convective (sloshing) mass as an additional impulsive mass.

13.7.6.1.3 Equipment and Attached Piping. Equipment, piping, and walkways or other appurtenances attached to the structure shall be designed to accommodate the displacements imposed by seismic forces. For piping attachments, see Section 13.7.4.

13.7.6.1.4 Internal Components. The attachments of internal equipment and accessories which are attached to the primary liquid or pressure retaining shell or bottom, or provide structural support for major components (e.g., a column supporting the roof rafters) shall be designed for the lateral loads due to the sloshing liquid in addition to the inertial forces by a substantiated analysis method.

13.7.6.1.5 Sliding Resistance. The transfer of the total lateral shear force between the tank or vessel and the subgrade shall be considered:

a. For unanchored flat bottom steel tanks, the overall horizontal seismic shear force is permitted to be resisted by friction between the tank bottom and the foundation or subgrade. Unanchored storage tanks shall be designed such that sliding will not occur when the tank is full of stored product. The maximum calculated seismic base shear, V, shall not exceed:

$$V < W \tan 30^{o} \qquad \textbf{(Eq. 13.7-14)}$$

W shall be determined using the effective weight of the tank, roof, and contents after reduction for coincident vertical earthquake. Lower values of the friction factor shall be used if the design of the tank bottom to supporting foundation does not justify the friction value above (e.g., leak detection membrane beneath the bottom with a lower friction factor, smooth bottoms, etc).

Table 13.7- 3: Minimum Required Freeboard

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$ g	a	a	δ_s^c
0.167 g $\leq S_{DS} < 0.33$ g	a	a	δ_s^c
0.33 g $\leq S_{DS} < 0.50$ g	a	$0.7\delta_s^b$	δ_s^c
$S_{DS} \geq$ 0.50 g	a	$0.7\delta_s^b$	δ_s^c

a No minimum freeboard is required.

b A freeboard equal to $0.7\,\delta_s$ is required unless one of the following alternatives is provided:
1. Secondary containment is provided to control the product spill.
2. The roof and supporting structure are designed to contain the sloshing liquid.

c Freeboard equal to the calculated wave height, δ_s, is required unless one of the following alternatives is provided:
1. Secondary containment is provided to control the product spill.
2. The roof and supporting structure are designed to contain the sloshing liquid.

b. No additional lateral anchorage is required for anchored steel tanks designed in accordance with reference documents.

c. The lateral shear transfer behavior for special tank configurations (e.g., shovel bottoms, highly crowned tank bottoms, tanks on grillage) can be unique and are beyond the scope of this Code.

13.7.6.1.6 **Local Shear Transfer.** Local transfer of the shear from the roof to the wall and the wall of the tank into the base shall be considered. For cylindrical tanks and vessels, the peak local tangential shear per unit length shall be calculated by:

$$V_{max} = \frac{2V}{\pi D} \qquad \textbf{(Eq. 13.7-15)}$$

a. Tangential shear in flat bottom steel tanks shall be transferred through the welded connection to the steel bottom. This transfer mechanism is deemed acceptable for steel tanks designed in accordance with the reference documents where $S_{DS} < 1.0$ g.

b. For concrete tanks with a sliding base where the lateral shear is resisted by friction between the tank wall and the base, the friction coefficient value used for design shall not exceed tan 30 degrees.

c. Fixed-base or hinged-base concrete tanks, transfer the horizontal seismic base shear shared by membrane (tangential) shear and radial shear into the foundation. For anchored flexible-base concrete tanks, the majority of the base shear is resisted by membrane (tangential) shear through the anchoring system with only insignificant vertical bending in the wall. The connection between the wall and floor shall be designed to resist the maximum tangential shear.

13.7.6.1.7 **Pressure Stability.** For steel tanks, the internal pressure from the stored product stiffens thin cylindrical .shell structural elements subjected to membrane compression forces. This stiffening effect may be considered in resisting seismically induced compressive forces if permitted by the reference document or the building official.

13.7.6.1.8 **Shell Support.** Steel tanks resting on concrete ring walls or slabs shall have a uniformly supported annulus under the shell. Uniform support shall be provided by one of the following methods:

a. Shimming and grouting the annulus

b. Using fiberboard or other suitable padding

c. Using butt-welded bottom or annular plates resting directly on the foundation

d. Using closely spaced shims (without structural grout) provided that the localized bearing loads are considered in the tank wall and foundation to prevent local crippling and spalling.

Anchored tanks shall be shimmed and grouted. Local buckling of the steel shell for the peak compressive force due to operating loads and seismic overturning shall be considered.

13.7.6.1.9 **Repair, Alteration, or Reconstruction.** Repairs, modifications, or reconstruction (i.e., cut down and re-erect) of a tank or vessel shall conform to industry standard practice and this Code. For welded steel tanks storing liquids (see ANSI/API 653-01) and the applicable reference document listed in Section 13.2. Tanks that are relocated shall be re-evaluated for the seismic loads for the new site and the requirements of new construction in accordance with the appropriate reference document and this Code.

13.7.7 Water and Water Treatment Tanks and Vessels

13.7.7.1 **Welded Steel.** Welded steel water storage tanks and vessels shall be designed in accordance with the seismic requirements of ANSI/ AWWA D100-96 except that the design input forces shall be modified as follows: The equations for base shear and overturning moment are defined by the following equations for allowable stress design procedures:

For $T_s < T_c \leq T_L$

$$V_{ACT} = \frac{S_{DS}}{1.4\left(R/I\right)}\left[\left(W_s + W_r + W_f + W_1\right) + 1.5\frac{T_s}{T_c}W_2\right] \qquad \textbf{(Eq 13.7-16)}$$

$$M = \frac{S_{DS}}{1.4\left(R/I\right)}\left[\left(W_s X_s + W_r H_t + W_1 X_1\right) + 1.5\frac{T_s}{T_c}W_2 X_2\right] \qquad \textbf{(Eq 13.7-17)}$$

For $T_c > T_L$

$$V_{ACT} = \frac{S_{DS} I}{1.4\,R}\left[\left(W_s + W_r + W_f + W_1\right) + 1.5\frac{T_s T_L}{T_c^2}W_2\right] \qquad \textbf{(Eq 13.7-18)}$$

$$M = \frac{S_{DS} I}{1.4R}\left[\left(W_s X_s + W_r H_t + W_1 X_1\right) + 1.5\frac{T_s T_L}{T_c^2}W_2 X_2\right] \qquad \textbf{(Eq 13.7-19)}$$

a. Substitute the above equations for Eqs. 13-4 and 13-8 of ANSI/AWWA D100-96 where S_{DS} and T_s are defined in Section 9.4.4, T_L is defined in Section 9.4.5, and R is defined in Table l3.4-2.

b. The hydrodynamic seismic hoop tensile stress is defined in Eqs. 13-20 through 13-25 in ANSI/AWWA D100-96. When using these equations, substitute Eq. 13.7-20 for $\frac{ZI}{R_w}$ directly into the equations.

$$\frac{S_{DS}}{2.5\left[1.4\left(R/I\right)\right]} \qquad \textbf{(Eq. 13.7-20)}$$

c. Sloshing height shall be calculated per Section 13.7.6.1.2 instead of Eq. 13-26 of ANSI/AWWA D100-96.

13.7.7.2 **Bolted Steel.** Bolted steel water storage structures shall be designed in accordance with the seismic requirements of ANSI/AWWA D103-97 except that the design input forces shall be modified in the same manner shown in Section 13.7.7.1 of this Code requirement.

13.7.7.3 **Reinforced and Prestressed Concrete.** Reinforced and prestressed concrete tanks shall be designed in accordance with the seismic requirements of ACI 530.3-01 except that the design input forces for allowable stress design procedures shall be modified as follows:

a. For $T_I < T_o$, and $T_I > T_s$ substitute the term $S_a/[1.4\ (R/I)]$ where S_a is defined in Section 9.4.5, Subsections 1, 2, or 3, for the terms in the appropriate equations as shown below:

For $\frac{ZC_I}{(R_1/I)}$ shear and overturning moment equations of ANSI/AWWA D110-95

For $\frac{ZC_I}{(R_W/I)}$ shear and overturning moment equations of ANSI/AWWA D115-95

For $\frac{ZSC_i}{(R_i/I)}$ in the base shear and overturning moment equations of ACI 530.3-01

b. For $T_o \le T_I \le T_s$, substitute the terms $\frac{S_{DS}}{1.4\,(R/I)}$ for terms $\frac{ZC_I}{(R_i/I)}$ and $\frac{ZSC_i}{(R_i/I)}$

c. For all values of T_c (or T_w), $\frac{ZC_c}{(R_c/I)}$, $\frac{ZC_c}{(R_W/I)}$ and $\frac{ZSC_c}{(R_c/I)}$ are replaced by $\frac{1.5\,S_{D1} I\,T_L}{T_c^2}$ or $\frac{1.5\,S_{DS} I T_S T_L}{T_c^2}$

where

S_a, S_{D1}, S_{DS}, T_o, T_s and T_L are defined in Section 9.4.5 of this Code.

13.7.8 Petrochemical and Industrial Tanks and Vessels Storing Liquids

13.7.8.1 **Welded Steel.** Welded steel petrochemical and industrial tanks and vessels storing liquids shall be designed in accordance with the seismic requirements of API 620-02 and API 650-01 except that the design input forces for allowable stress design procedures shall be modified as follows:

a. When using the equations in Section E.3 of API 650-01, substitute into the equation for overturning moment M (where S_{DS}, T_s and T_L are defined in Section 9.4.5 of this Code requirement. Thus,

In the range $Ts < T_c \le T_L$,

$$M = S_{DS} I[0.24(W_s X_s + W_t H_t + W_1 X_1) + 0.80\ C_2 T_s W_2 X_2] \qquad \textbf{(Eq. 13.7-21)}$$

where

$$C_2 = \frac{0.75\,S}{T_c} \quad \text{and } S = 1.0$$

In the range $T_w > T_L$, and

$$M = S_{DS}I[0.24(W_sX_s + W_tH_t + W_1X_1) + 0.71\,C_2T_sW_2X_2] \qquad \textbf{(Eq. 13.7-22)}$$

where

$$C_2 = \frac{0.8438\,ST_L}{T_c^2} \quad \text{and } S = 1.0$$

13.7.8.2 **Bolted Steel.** Bolted steel tanks used for storage of production liquids. API 12B-95 covers the material, design, and erection requirements for vertical, cylindrical, aboveground bolted tanks in nominal capacities of 100 to 10,000 barrels for production service. Unless required by the building official having jurisdiction, these temporary structures need not be designed for seismic loads. If design for seismic load is required, the loads may be adjusted for the temporary nature of the anticipated service life.

13.7.8.3 **Reinforced and Prestressed Concrete.** Reinforced concrete tanks for the storage of petrochemical and industrial liquids shall be designed in accordance with the force requirements of Section 13.7.7.3.

13.7.9 Ground-Supported Storage Tanks for Granular Materials

13.7.9.1 **General.** The inter-granular behavior of the material shall be considered in determining effective mass and load paths, including the following behaviors:

a. Increased lateral pressure (and the resulting hoop stress) due to loss of the inter-granular friction of the material during the seismic shaking.

b. Increased hoop stresses generated from temperature changes in the shell after the material has been compacted.

c. Inter-granular friction, which can transfer seismic shear directly to the foundation.

13.7.9.2 **Lateral Force Determination.** The lateral forces for tanks and vessels storing granular materials at grade shall be determined by the requirements and accelerations for short period structures (i.e., S_{DS}).

13.7.9.3 **Force Distribution to Shell and Foundation**

13.7.9.3.1 **Increased Lateral Pressure.** The increase in lateral pressure on the tank wall shall be added to the static design lateral pressure but shall not be used in the determination of pressure stability effects on the axial buckling strength of the tank shell.

13.7.9.3.2 **Effective Mass.** A portion of a stored granular mass will act with the shell (the effective mass). The effective mass is related to the physical characteristics of the product, the height-to-diameter (H/D) ratio of the tank, and the intensity of the seismic event. The effective mass shall be used to determine the shear and overturning loads resisted by the tank.

13.7.9.3.3 **Effective Density.** The effective density factor (that part of the total stored mass of product which is accelerated by the seismic event) shall be determined in accordance ACI 313-97.

13.7.9.3.4 Lateral Sliding. For granular storage tanks that have a steel bottom and are supported such that friction at the bottom to foundation interface can resist lateral shear loads, no additional anchorage to prevent sliding is required. For tanks without steel bottoms (i.e., the material rests directly on the foundation), shear anchorage shall be provided to prevent sliding.

13.7.9.3.5 Combined Anchorage Systems. If separate anchorage systems are used to prevent over-turning and sliding, the relative stiffness of the systems shall be considered in determining the load distribution.

13.7.9.4 Welded Steel Structures. Welded steel granular storage structures shall be designed in accordance with the seismic provisions of this Code. Component allowable stresses and materials shall be per ANSI/AWWA D100-96, except the allowable circumferential membrane stresses and material requirements in API 650-01 shall apply.

13.7.9.5 Bolted Steel Structures. Bolted steel granular storage structures shall be designed in accordance with the seismic provisions of this section. Component allowable stresses and materials shall be per ANSI/AWWA D103-97.

13.7.9.6 Reinforced Concrete Structures. Reinforced concrete structures for the storage of granular materials shall be designed in accordance with the seismic force requirements of this Code and the requirements of ACI 313-97.

13.7.9.7 Prestressed Concrete Structures. Prestressed concrete structures for the storage of granular materials shall be designed in accordance with the seismic force provisions of this Code and the requirements of ACI 313-97.

13.7.10 Elevated Tanks and Vessels for Liquids and Granular Materials

13.7.10.1 General. This section applies to tanks, vessels, bins, and hoppers that are elevated above grade where the supporting tower is an integral part of the structure, or where the primary function of the tower is to support the tank or vessel. Tanks and vessels that are supported within buildings, or are incidental to the primary function of the tower, are considered mechanical equipment and shall be designed in accordance with Chapter 12 of this Code requirement.

Elevated tanks shall be designed for the force and displacement requirements of the applicable reference document, or Section 13.4.

13.7.10.2 Effective Mass. The design of the supporting tower or pedestal, anchorage, and foundation for seismic over-turning shall assume the material stored is a rigid mass acting at the volumetric center of gravity. The effects of fluid-structure interaction may be considered in determining the forces, effective period, and mass centroids of the system if the following requirements are met:

- **a.** The sloshing period, T_c is greater than 3T where T = natural period of the tank with confined liquid (rigid mass) and supporting structure.
- **b.** The sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid-structure interaction analysis or testing.

Soil-structure interaction may be included in determining T Section 10.14.

13.7.10.3 P-Delta Effects. The lateral drift of the elevated tank shall be considered as follows:

a. The design drift, the elastic lateral displacement of the stored mass center of gravity shall be increased by the factor, C_d for evaluating the additional load in the support structure.

b. The base of the tank shall be assumed to be fixed rotationally and laterally.

c. Deflections due to bending, axial tension, or compression shall be considered. For pedestal tanks with a height-to-diameter ratio less than 5, shear deformations of the pedestal shall be considered.

d. The dead load effects of roof mounted equipment or platforms shall be included in the analysis.

e. If constructed within the plumbness tolerances specified by the reference document, initial tilt need not be considered in the P-delta analysis.

13.7.10.4 **Transfer of Lateral Forces into Support Tower.** For post supported tanks and vessels which are cross-braced:

a. The bracing shall be installed in such a manner as to provide uniform resistance to the lateral load (e.g., pretensioning or tuning to attain equal sag).

b. The additional load in the brace due to the eccentricity between the post to tank attachment and the line of action of the bracing shall be included.

c. Eccentricity of compression strut line of action (elements that resist the tensile pull from the bracing rods in the seismic force-resisting systems) with their attachment points shall be considered.

d. The connection of the post or leg with the foundation shall be designed to resist both the vertical and lateral resultant from the yield load in the bracing assuming the direction of the lateral load is oriented to produce the maximum lateral shear at the post to foundation interface. Where multiple rods are connected to the same location, the anchorage shall be designed to resist the concurrent tensile loads in the braces.

13.7.10.5 **Evaluation of Structures Sensitive to Buckling Failure.** Shell structures that support substantial loads may exhibit a primary mode of failure from localized or general buckling of the support pedestal or skirt during seismic loads. Such structures may include single pedestal water towers, skirt-supported process vessels, and similar single member towers. Where the structural assessment concludes that buckling of the support is the governing primary mode of failure, structures and components in Occupancy Category IV shall be designed to resist the seismic forces as follows:

a. The seismic response coefficient for this evaluation shall be per Section 10.9.2.1 of this Code with I/R set equal to 1.0. Soil-structure and fluid-structure interaction may be utilized in determining the Structural response. Vertical or orthogonal combinations need not be considered.

b. The resistance of the structure or component shall be defined as the critical buckling resistance of the element; i.e., a factor of safety set equal to 1.0.

c. The anchorage and foundation shall be designed to resist the load determined in (a). The foundation shall be proportioned to provide a stability ratio of at least 1.2 for the overturning moment. The maximum toe pressure under the foundation shall not exceed the lesser of the ultimate bearing capacity or 3 times the allowable bearing capacity. All structural components and elements of the foundation shall be designed to resist the combined loads with a load factor of 1.0 on all loads including dead load, live load, and earthquake load. Anchors shall be permitted to yield.

13.7.10.6 **Welded Steel Water Storage Structures.** Welded steel elevated water storage structures shall be designed and detailed in accordance with the seismic requirements of ANSI/AWWA D100-96 and this Code except that the design input forces for allowable stress design procedures shall be modified by substituting the following terms for $\frac{ZC}{(R_w/I)}$ into Eqs. 13-1 and 13-3 of ANSI/AWWA D100-96 and set the value for S = 1.0.

For $T \leq T_s$ substitute the term

$$\frac{S_{DS}}{1.4(R/I)} \quad \textbf{(Eq. 13.7-22)}$$

For $T_s < T \leq 4.0$ sec, substitute the term

$$\frac{S_{D1}}{T[1.4(R/I)]} \quad \textbf{(Eq. 13.7-23)}$$

13.7.10.6.1 **Analysis Procedures.** The equivalent lateral force procedure shall be permitted. A more rigorous analysis shall also be permitted. Analysis of single pedestal structures shall be based on a fixed-base, single degree-of-freedom model. All mass, including the liquid, shall be considered rigid unless the sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid-structure interaction analysis or testing. The inclusion of soil-structure interaction shall be permitted.

13.7.10.6.2 **Structure Period.** The fundamental period of vibration of the structure shall be established using the structural properties and deformational characteristics of the resisting elements in a substantiated analysis. The period used to calculate the seismic response coefficient shall not exceed 4.0 sec. See ANSI/AWWA D100-96 for guidance on computing the fundamental period of cross-braced structures.

13.7.10.7 **Concrete Pedestal (Composite) Tanks.** Concrete pedestal (composite) elevated water storage structures shall be designed in accordance with the requirements of ACI 371R-98 and except that the design input forces shall be modified as follows:

In Eq. 4-8a of ACI 371R-98,

For $T_s < T \leq 2.5$ sec, replace the term $\frac{1.2\,C_v}{RT^{2/3}}$ with

$$\frac{S_{D1}}{T(R/I)} \quad \textbf{(Eq. 13.7-24)}$$

In Eq. 4-8b of ACI 371R-98, replace the term $\frac{2.5\,C_a}{R}$ with

$$\frac{S_{DS}}{(R/I)} \quad \textbf{(Eq. 13.7-25)}$$

In Eq. 4-9 of ACI 371R-98, replace the term $0.5C_a$ with

$$0.2\,S_{DS} \quad \textbf{(Eq. 13.7-26)}$$

13.7.10.7.1 **Analysis Procedures.** The equivalent lateral force procedure shall be permitted for all concrete pedestal tanks and. shall be based on a fixed-base, single degree-of-freedom model. All mass, including the liquid, shall be considered rigid unless the sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid-structure interaction analysis or testing. Soil structure interaction may be included as per Section 10.14. A more rigorous analysis shall be permitted.

13.7.10.7.2 **Structure Period.** The fundamental period of vibration of the structure shall be established using the uncracked structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The period used to calculate the seismic response coefficient shall not exceed 2.5 sec.

13.7.11 Boilers and Pressure Vessels

13.7.11.1 **General.** Attachments to the pressure boundary, supports, and lateral force-resisting anchorage systems for boilers and pressure vessels shall be designed to meet the force and displacement requirements of Sections 15.3 or 15.4 and the additional requirements of this section. Boilers and pressure vessels categorized as Occupancy Category III or IV shall be designed to meet the force and displacement requirements of Sections 13.3 or 13.4.

13.7.11.2 **ASME Boilers and Pressure Vessels.** Boilers or pressure vessels designed and constructed in accordance with ASME BPVC-03 shall be deemed to meet the requirements of this section provided that the force and displacement requirements of Sections 13.3 or 13.4 are used with appropriate scaling of the force and displacement requirements to the working stress design basis.

13.7.11.3 **Attachments of Internal Equipment and Refractory.** Attachments to the pressure boundary for internal and external ancillary components (refractory, cyclones, trays, etc.) shall be designed to resist the seismic forces specified in this Code to safeguard against rupture of the pressure boundary. Alternatively, the element attached may be designed to fail prior to damaging the pressure boundary provided that the consequences of the failure do not place the pressure boundary in jeopardy. For boilers or vessels containing liquids, the effect of sloshing on the internal equipment shall be considered if the equipment can damage the integrity of the pressure boundary.

13.7.11.4 **Coupling of Vessel and Support Structure.** Where the mass of the operating vessel or vessels supported is greater than 25 percent of the total mass of the combined structure, the structure and vessel designs shall consider the effects of dynamic coupling between each other. Coupling with adjacent, connected structures such as multiple towers shall be considered if the structures are interconnected with elements that will transfer loads from one structure to the other.

13.7.11.5 **Effective Mass.** Fluid-structure interaction (sloshing) shall be considered in determining the effective mass of the stored material providing sufficient liquid surface exists for sloshing to occur and the T_c is greater than 3T. Changes to or variations in material density with pressure and temperature shall be considered.

13.7.11.6 **Other Boilers and Pressure Vessels.** Boilers and pressure vessels designated as Occupancy Category IV but are not designed and constructed in accordance with the requirements of ASME BPVC-03 shall meet the following requirements:

The seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the material strength shown in Table 13.7-4.

Consideration shall be made to mitigate seismic impact loads for boiler or vessel components constructed of non-ductile materials or vessels operated in such a way that material ductility is reduced (e.g., low temperature applications).

Table 13.7- 4: Maximum Material Strength

Material	Minimum Ratio F_u/F_y	Max. Material Strength Vessel Material	Max. Material Strength Threaded Material[a]
Ductile (e.g., steel, aluminum, copper)	1.33[b]	90%[d]	70%[d]
Semi-ductile	1.2[c]	70%[d]	50%[d]
Nonductile (e.g., cast iron, ceramics, fiberglass)	NA	25%[e]	20%[e]

a Threaded connection to vessel or support system.
b Minimum 20% elongation per the ASTM material specification.
c Minimum 15% elongation per the ASTM material specification.
d Based on material minimum specified yield strength.
e Based on material minimum specified tensile strength.

13.7.11.7 **Supports and Attachments for Boilers and Pressure Vessels.** Attachments to the pressure boundary and support for boilers and pressure vessels shall meet the following requirements:

a. Attachments and supports transferring seismic loads shall be constructed of ductile materials suitable for the intended application and environmental conditions.

b. Seismic anchorages embedded in concrete shall be ductile and detailed for cyclic loads.

c. Seismic supports and attachments to structures shall be designed and constructed so that the support or attachment remains ductile throughout the range of reversing seismic lateral loads and displacements.

d. Vessel attachments shall consider the potential effect on the vessel and the support for uneven vertical reactions based on variations in relative stiffness of the support members, dissimilar details, non-uniform shimming, or irregular supports. Uneven distribution of lateral forces shall consider the relative distribution of the resisting elements, the behavior of the connection details, and vessel shear distribution.

The requirements of Sections 13.4 and 13.7.10.5 shall also be applicable to this section.

13.7.12 Liquid and Gas Spheres

13.7.12.1 **General.** Attachments to the pressure or liquid boundary, supports, and lateral force-resisting anchorage systems for liquid and gas spheres shall be designed to meet the force and displacement requirements of Sections 13.3 or 13.4 and the additional requirements of this section. Spheres categorized as Occupancy Category III or IV shall themselves be designed to meet the force and displacement requirements of Sections 13.3 or 13.4.

13.7.12.2 **ASME Spheres.** Spheres designed and constructed in accordance with Section VIII of ASME BPVC-03 shall be deemed to meet the requirements of this section providing the force and displacement requirements of Sections 13.3 or 13.4 are used with appropriate scaling of the force and displacement requirements to the working stress design basis.

13.7.12.3 **Attachments of Internal Equipment and Refractory.** Attachments to the pressure or liquid boundary for internal and external ancillary components (refractory, cyclones, trays, etc.) shall be designed to resist the seismic forces specified in this Code to safeguard against rupture of the pressure boundary. Alternatively, the element attached to the sphere could be designed to fail prior to damaging the pressure or liquid boundary providing the consequences of the failure does not place the pressure boundary in jeopardy. For spheres containing liquids, the effect of sloshing on the internal equipment shall be considered if the equipment can damage the pressure boundary.

13.7.12.4 **Effective Mass.** Fluid-structure interaction (sloshing) shall be considered in determining the effective mass of the stored material providing sufficient liquid surface exists for sloshing to occur and the T_c is greater than 3T. Changes to or variations in fluid density shall be considered.

13.7.12.5 **Post and Rod Supported.** For post supported spheres that are cross-braced:

- **a.** The requirements of Section 13.7.10.4 shall also be applicable to this section.
- **b.** The stiffening effect of (reduction in lateral drift) from pretensioning of the bracing shall be considered in determining the natural period.
- **c.** The slenderness and local buckling of the posts shall be considered.
- **d.** Local buckling of the sphere shell at the post attachment shall be considered.
- **e.** For spheres storing liquids, bracing connections shall be designed and constructed to develop the minimum published yield strength of the brace. For spheres storing gas vapors only, bracing connection shall be designed for Ω_0 times the maximum design load in the brace. Lateral bracing connections directly attached to the pressure or liquid boundary are prohibited.

13.7.12.6 **Skirt Supported.** For skirt-supported spheres, the following requirements shall apply:

- **a.** The provisions of Section 13.7.10.5 shall also apply.
- **b.** The local buckling of the skirt under compressive membrane forces due to axial load and bending moments shall be considered.
- **c.** Penetration of the skirt support (manholes, piping, etc.) shall be designed and constructed to maintain the strength of the skirt without penetrations.

13.7.13 Refrigerated Gas Liquid Storage Tanks and Vessels

13.7.13.1 **General.** The seismic design of the tanks and facilities for the storage of liquefied hydrocarbons and refrigerated liquids is beyond the scope of this section. The design of such tanks is addressed in part by various reference documents listed in

Section 13.2.

Exception: Low pressure, welded steel storage tanks for liquefied hydrocarbon gas (e.g., LPG, butane, etc.) and refrigerated liquids (e.g., ammonia) shall be designed in accordance with .the requirements of Section 13.7.8 and API 620-02.

13.7.14 Horizontal, Saddle Supported Vessels for Liquid or Vapor Storage

13.7.14.1 **General.** Horizontal vessels supported on saddles (sometimes referred to as blimps) shall be designed to meet the force and displacement requirements of Sections 13.3 or 13.4.

13.7.14.2 **Effective Mass.** Changes to or variations in material density shall be considered. The design of the supports, saddles, anchorage, and foundation for seismic overturning shall assume the material stored is a rigid mass acting at the volumetric center of gravity.

13.7.14.3 **Vessel Design.** Unless a more rigorous analysis is performed,

a. Horizontal vessels with a length-to-diameter ratio of 6 or more may be assumed to be a simply supported beam spanning between the saddles for determining the natural period of vibration and global bending moment.

b. Horizontal vessels with a length-to-diameter ratio of less than 6, the effects of "deep beam shear" shall be considered when determining the fundamental period and stress distribution.

c. Local bending and buckling of the vessel shell at the saddle supports due to seismic load shall be considered. The stabilizing effects of internal pressure shall not be considered to increase the buckling resistance of the vessel shell.

d. If the vessel is a combination of liquid and gas storage, the vessel and supports shall be designed both with and without gas pressure acting (assume piping has ruptured and pressure does not exist).

CHAPTER 14
SITE CLASSIFICATION PROCEDURE FOR SEISMIC DESIGN

SECTION 14.1
GENERAL

14.1.1 **Site Class Definitions.** The site shall be classified as one of the following classes:

A = Hard rock with measured shear wave velocity, $\overline{\upsilon_s}$ > 1500 m/s

B = Rock with 760 m/s < $\overline{\upsilon_s}$ ≤ 1500 m/s

C = Very dense soil and soft rock with 370 m/s ≤ $\overline{\upsilon_s}$ ≤ 760 m/s or $\overline{N}$ or $\overline{N}_{ch}$ > 50 or $\overline{s_u}$ ≥ 100 kPa

D = Stiff soil with 180 m/s ≤ $\overline{\upsilon_s}$ ≤ 370 m/s or with 15 ≤ $\overline{N}$ or $\overline{N}_{ch}$ ≤ 50 or 50 kPa ≤ $\overline{s_u}$ ≤ 100 kPa

E = A soil profile with $\overline{\upsilon_s}$ < 180 m/s or any profile with more than 3 m of soft clay. Soft clay is defined as soil with PI > 20, w ≥ 40%, and $\overline{s_u}$ < 25 kPa

F = Soils requiring site-specific evaluations:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.

   Exception: For structures having fundamental periods of vibration equal to or less than 0.5-sec, site-specific evaluations are not required to determine spectral accelerations for liquefiable soils. Rather, the Site Class may be determined in accordance with Section 14.1.2 and the corresponding values of F_a and F_v determined from Tables 9.4.3a and 9.4.3b.
2. Peats and/or highly organic clays (H > 3 m of peat and/or highly organic clay where *H* = thickness of soil).
3. Very high plasticity clays *(H* > 8 m with *PI > 75)*.
4. Very thick soft/medium stiff clays (H > 37 m).

Exception: When the soil properties are not known in sufficient detail to determine the Site Class, Class D shall be used. Site Class E shall be used when the authority having jurisdiction determines that Site Class E is present at the site or in the event that Site E is established by geotechnical data.

14.1.1.1 **Referenced Standards.** The following standards are referenced in the provisions for determining the seismic coefficients:

[1] ASTM. "Test Method for Penetration Test and Split-Barrel Sampling of Soils." ASTM D1586-84, 1984.

[2] ASTM. "Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils." ASTM D4318-93, 1993.

[3] ASTM. "Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock." ASTM D2216-92, 1992.

[4] ASTM. "Test Method for Unconfined Compressive Strength of Cohesive Soil." ASTM D2166-91, 1991.

[5] ASTM. "Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression." ASTM D2850-87, 1987.

TABLE 14.1.1: SITE CLASSIFICATION

<table>
<tr><th>Site Class</th><th>$\overline{\upsilon}_s$</th><th>$\overline{N}$ or $\overline{N}_{ch}$</th><th>$\overline{s}_u$</th></tr>
<tr><td>A
Hard rock</td><td>> 1500 m/s</td><td>not applicable</td><td>not applicable</td></tr>
<tr><td>B
Rock</td><td>760 to 1500 m/s</td><td>not applicable</td><td>not applicable</td></tr>
<tr><td>C
Very dense soil and soft rock</td><td>370 to 760 m/s</td><td>> 50</td><td>> 100 kPa</td></tr>
<tr><td>D
Stiff soil</td><td>180 to 370 m/s</td><td>15 to 50</td><td>50 to 100 kPa</td></tr>
<tr><td rowspan="2">E
Soil</td><td><180 m/s</td><td>< 15</td><td><50 kPa</td></tr>
<tr><td colspan="3">Any profile with more than 3 m of soil having the following characteristics:
- Plasticity index PI > 20,
- Moisture content w $\geq$ 40%, and
- Undrained shear strength $\overline{s}_u$ < 25 kPa</td></tr>
<tr><td colspan="2">F Soils requiring site-specific evaluation</td><td colspan="2">1. Soils vulnerable to potential failure or collapse
2. Peats and/or highly organic clays
3. Very high plasticity clays
4. Very thick soft/medium clays</td></tr>
</table>

Note: When the soil properties are not known in sufficient detail to determine the Site Class, Class D or E shall be used.

14.1.2 **Steps for Classifying a Site.** The Site Class of a site shall be determined using the following steps:

Step 1: Check for the four categories of Site Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.

Step 2: Check for the existence of a total thickness of soft clay > 3 m where a soft clay layer is defined by $\overline{s}_u$ < 25 kPa, $w \geq 40\%$, and $PI > 20$. If this criterion is satisfied, classify the site as Site Class E.

Step 3: Categorize the site using one of the following three methods with $\overline{\upsilon}_s$, $\overline{N}$, and $\overline{s}_u$ computed in all cases as specified by the definitions in Section 14.1.3.

a. The $\overline{\upsilon_s}$ method:

Determine $\overline{\upsilon_s}$ for the top 30 m of soil. Compare the value of $\overline{\upsilon_s}$ with those given in Section 14.1.1 and Table 14.1.1 and assign the corresponding Site Class.

$\overline{\upsilon_s}$ for rock, Site Class B, shall be measured on-site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering.

$\overline{\upsilon_s}$ for softer and more highly fractured and weathered rock shall be measured on-site or shall be classified as Site Class C.

The classification of hard rock, Site Class A, shall be supported by on-site measurements of $\overline{\upsilon_s}$ or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of at least 30 m, surficial measurements of υ_s are not prohibited from being extrapolated to assess $\overline{\upsilon_s}$.

The rock categories, Site Classes A and B, shall not be assigned to a site if there is more than 3 m of soil between the rock surface and the bottom of the spread footing or mat foundation.

b. The $\overline{N}$ method:

Determine $\overline{N}$ for the top 30 m of soil. Compare the value of $\overline{N}$ with those given in Section 14.1.1 and Table 14.1.1 and assign the corresponding Site Class.

c. The $\overline{s_u}$ method:

For cohesive soil layers, determine $\overline{s_u}$ for the top 30 m of soil. For cohesionless soil layers, determine $\overline{N}_{ch}$ for the top 30 m of soil. Cohesionless soil is defined by a PI < 20 where cohesive soil is defined by a PI > 20. Compare the values of $\overline{s_u}$ and $\overline{N}_{ch}$ with those given in Section 14.1.1 and Table 14.1.1 and assign the corresponding Site Class. When the $\overline{N}_{ch}$ and $\overline{s_u}$ criteria differ, assign the category with the softer soil. (Site Class E soil is softer than D).

14.1.3 **Definitions of Site Class Parameters.** The definitions presented below apply to the upper 30 m of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to *n* at the bottom where there are a total of *n* distinct layers in the upper 30 m. Where some of the *n* layers are cohesive and others are not, *k* is the number of cohesive layers and *m* is the number of cohesionless layers. The symbol *i* refers to any one of the layers between 1 and *n*.

v_{si} is the shear wave velocity in m/s.

d_i is the thickness of any layer between 0 and 30 m.

$\overline{\upsilon_s}$ is

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{\upsilon_{si}}} \qquad \textbf{(Eq. 14.1.3-1)}$$

whereby $\sum_{i=1}^{n} d_i$ is equal to 30 m

N_i is the standard penetration resistance, ASTM D1586-84 not to exceed 100 blows/300 mm as directly measured in the field without corrections.

$\overline{N}$ is:

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \textbf{(Eq. 14.1.3-2)}$$

$\overline{N}_{ch}$ is:

$$\overline{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \qquad \textbf{(Eq. 14.1.3-3)}$$

whereby $\sum_{i=1}^{m} d_i = d_s$. (Use only d_i and N_i for cohesionless soils.)

d_s is the total thickness of cohesionless soil layers in the top 30 m.

$\overline{s_{ui}}$ is the undrained shear strength in kPa, not to exceed 240 kPa, ASTM D2166-91 or ASTM D2850-87.

$\overline{s_u}$ is

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \qquad \textbf{(Eq. 14.1.3-4)}$$

whereby $\sum_{i=1}^{k} d_i = d_c$

d_c is the total thickness (30 - d_s) of cohesive soil layers in the top 30 m

PI is the plasticity index, ASTM D4318-93

ω is the moisture content in percent, ASTM D2216-92

CHAPTER 15
QUALITY ASSURANCE AND SUPPLEMENTAL PROVISIONS

SECTION 15.1
PURPOSE

These provisions are not directly related to computation of earthquake loads, but they are deemed essential for satisfactory performance in an earthquake when designing with the loads determined from Chapters 9 through 13, due to the substantial cyclic inelastic strain capacity assumed to exist by the load procedures given in the aforementioned chapters. These supplemental provisions form an integral part of SBC 301.

SECTION 15.2
QUALITY ASSURANCE

This section provides minimum requirements for quality assurance for seismic force-resisting systems and other designated seismic systems. These requirements supplement the testing and inspection requirements contained in the reference standards given in Chapters 11 through 13.

15.2.1 **Scope.** As a minimum, the quality assurance provisions apply to the following:

1. The seismic force-resisting systems in structures assigned to Seismic Design Categories C and D.
2. Other designated seismic systems in structures assigned to Seismic Design Category D that are required in Table 12.1.7.

   Exception: Structures that comply with the following criteria are exempt from the preparation of a quality assurance plan but those structures are not exempt from special inspection(s) or testing requirements:

   a. The structure is constructed of light-gauge cold-formed steel framing, S_{DS} does not exceed 0.50 g, the height of the structure does not exceed 10 m above grade, and the structure meets the requirements in Items c and d below,

   or

   b. The structure is constructed using a reinforced masonry structural system or reinforced concrete structural system, S_{DS} does not exceed 0.50 g, the height of the structure does not exceed 8 m above grade, and the structure meets the requirements in Items c and d below.

   c. The structure is classified as Occupancy Category I.

   d. The structure does not have any of the following plan irregularities as defined in Table 10.3.2.1 or any of the following vertical irregularities as defined in Table 10.3.2.2:

      1. Torsional irregularity
      2. Extreme torsional irregularity
      3. Nonparallel systems

4. Stiffness irregularity, soft story
5. Stiffness irregularity, extreme soft story
6. Discontinuity in capacity, weak story

15.2.2 Reference Standards.
The following standards (Ref. 15.2.2-1 through Ref. 15.2.2-3) are referenced in the provisions for inspection and testing. See Reference Chapter SBC 301.

15.2.3 Quality Assurance Plan. A quality assurance plan shall be submitted to the authority having jurisdiction.

15.2.3.1 Details of Quality Assurance Plan. The quality assurance plan shall specify the designated seismic systems or seismic force-resisting system in accordance with Section 15.2 that are subject to quality assurance. The registered design professional in responsible charge of the design of a seismic force-resisting system and a designated seismic system shall be responsible for the portion of the quality assurance plan applicable to that system. The special inspections and special tests needed to establish that the construction is in conformance with these provisions shall be included in the portion of the quality assurance plan applicable to the designated seismic system. The quality assurance plan shall include:

1. The seismic force-resisting systems and designated seismic systems in accordance with this chapter that are subject to quality assurance.

2. The special inspections and testing to be provided as required by these provisions and the reference standards in Chapter 11 through 13.

3. The type and frequency of testing.

4. The type and frequency of special inspections.

5. The frequency and distribution of testing and special inspection reports.

6. The structural observations to be performed.

7. The frequency and distribution of structural observation reports.

15.2.3.2 Contractor Responsibility. Each contractor responsible for the construction of a seismic force-resisting system, designated seismic system, or component listed in the quality assurance plan shall submit a written contractor's statement of responsibility to the regulatory authority having jurisdiction and to the owner prior to the commencement of work on the system or component. The contractor's statement of responsibility shall contain the following:

1. Acknowledgment of awareness of the special requirements contained in the quality assurance plan.

2. Acknowledgment that control will be exercised to obtain conformance with the design documents approved by the authority having jurisdiction.

3. Procedures for exercising control within the contractor's organization, the method and frequency of reporting, and the distribution of the reports.

4. Identification and qualifications of the person(s) exercising such control and their position(s) in the organization.

15.2.4 **Special Inspection.** The building owner shall employ a special inspector(s) to observe the construction of all designated seismic systems in accordance with the quality assurance plan for the following construction work:

15.2.4.1 **Foundations.** Continuous special inspection is required during driving of piles and placement of concrete in piers or piles. Periodic special inspection is required during construction of drilled piles, piers, and caisson work, the placement of concrete in shallow foundations, and the placement of reinforcing steel.

15.2.4.2 **Reinforcing Steel.**

15.2.4.2.1 Periodic special inspection during and on completion of the placement of reinforcing steel in intermediate and special moment frames of concrete and concrete shear walls.

15.2.4.2.2 Continuous special inspection during the welding of reinforcing steel resisting flexural and axial forces in intermediate and special moment frames of concrete, in boundary members of concrete shear walls, and welding of shear reinforcement.

15.2.4.3 **Structural Concrete.** Periodic special inspection during and on completion of the placement of concrete in intermediate and special moment frames, and in boundary members of concrete shear walls.

15.2.4.4 **Prestressed Concrete.** Periodic special inspection during the placement and after the completion of placement of prestressing steel and continuous special inspection is required during all stressing and grouting operations and during the placement of concrete.

15.2.4.5 **Structural Masonry.**

15.2.4.5.1 Periodic special inspection during the preparation of mortar, the laying of masonry units, and placement of reinforcement; and prior to placement of grout.

15.2.4.5.2 Continuous special inspection during welding of reinforcement, grouting, consolidation and reconsolidation, and placement of bent-bar anchors as required by Section 15.6.

15.2.4.6 **Structural Steel.**

15.2.4.6.1 Continuous special inspection is required for all structural welding.

Exception: Periodic special inspection for single-pass fillet or resistance welds and welds loaded to less than 50% of their design strength shall be the minimum requirement, provided the qualifications of the welder and the welding electrodes are inspected at the beginning of the work and all welds are inspected for compliance with the approved construction documents at the completion of welding.

15.2.4.6.2 Periodic special inspection is required for installation and tightening of fully tensioned high-strength bolts in slip-critical connections and in connections subject to direct tension. Bolts in connections identified as not being slip-critical or subject to direct tension need not be inspected for bolt tension other than to ensure that the plies of the connected elements have been brought into snug contact.

15.2.4.7 **Cold-Formed Steel Framing.**

15.2.4.7.1 Periodic special inspection is required during all welding operations of elements of the seismic force-resisting system.

15.2.4.7.2 Periodic special inspection is required for screw attachment, bolting, anchoring, and other fastening of components within the seismic force-resisting system including struts, braces, and hold-downs.

15.2.4.8 **Architectural Components.** Special inspection for architectural components shall be as follows:

1. Periodic special inspection during the erection and fastening of exterior cladding, interior and exterior nonbearing walls, and interior and exterior veneer in Seismic Design Category D.

   Exceptions:

   a. Architectural components less than 9 m above grade or walking surface.

   b. Cladding and veneer weighing 250 N/ m^2 or less.

   c. Interior nonbearing walls weighing 700 N/m^2 or less.

2. Periodic special inspection during the anchorage of access floors, suspended ceilings, and storage racks 2.5 m or greater in height in Seismic Design Category D.

3. Periodic special inspection during erection of glass 9 m or more above an adjacent grade or walking surface in glazed curtain walls, glazed storefronts, and interior glazed partitions in Seismic Design Category D.

15.2.4.9 **Mechanical and Electrical Components.** Special inspection for mechanical and electrical components shall be as follows:

1. Periodic special inspection during the anchorage of electrical equipment for emergency or standby power systems in Seismic Design Categories C and D.

2. Periodic special inspection during the installation for flammable, combustible, or highly toxic piping systems and their associated mechanical units in Seismic Design Categories C, and D.

3. Periodic special inspection during the installation of HVAC ductwork that will contain hazardous materials in Seismic Design Categories C and D.

4. Periodic special inspection during the installation of vibration isolation systems when the construction documents indicate a maximum clearance (air gap) between the equipment support frame and restraint less than or equal to 6 mm.

15.2.5 **Testing.** The special inspector(s) shall be responsible for verifying that the special test requirements are performed by an approved testing agency for the types of work in designated seismic systems listed below.

15.2.5.1 **Reinforcing and Prestressing Steel.** Special testing of reinforcing and prestressing steel shall be as follows:

15.2.5.1.1 Examine certified mill test reports for each shipment of reinforcing steel used to resist flexural and axial forces in reinforced concrete intermediate and special moment frames and boundary members of reinforced concrete shear walls or reinforced masonry shear walls and determine conformance with construction documents.

15.2.5.1.2 Where ASTM A615 or Saudi equivalent reinforcing steel is used to resist earthquake-induced flexural and axial forces in special moment frames and in wall boundary elements of shear walls in structures of Seismic Design Category D, verify that the requirements of Section 21.2.5.1 of SBC 304 have been satisfied.

15.2.5.1.3 Where ASTM A615 reinforcing steel or Saudi equivalent is to be welded, verify that chemical tests have been performed to determine weldability in accordance with Section 3.5.2 of SBC 304.

15.2.5.2 **Structural Concrete.** Samples of structural concrete shall be obtained at the project site and tested in accordance with the requirements of SBC 304.

15.2.5.3 **Structural Masonry.** Quality assurance testing of structural masonry shall be in accordance with the requirements of Ref. 11.4-1.

15.2.5.4 **Structural Steel.** The testing needed to establish that the construction is in conformance with these provisions shall be included in a quality assurance plan. The minimum testing contained in the quality assurance plan shall be as required in Ref. 11.1-1 and the following requirements:

15.2.5.4.1 **Base Metal Testing.** Base metal thicker than 38 mm, when subject to through-thickness weld shrinkage strains, shall be ultrasonically tested for discontinuities behind and adjacent to such welds after joint completion. Any material discontinuities shall be accepted or rejected on the basis of ASTM A435, *Specification for Straight Beam Ultrasound Examination of Steel Plates,* or ASTM A898, *Specification for Straight Beam Ultrasound Examination for Rolled Steel Shapes (Level 1 Criteria),* and criteria as established by the registered design professional(s) in responsible charge and the construction documents.

15.2.5.5 **Mechanical and Electrical Equipment.** As required to ensure compliance with the seismic design provisions herein, the registered design professional in responsible charge shall clearly state the applicable requirements on the construction documents. Each manufacturer of these designated seismic system components shall test or analyze the component and its mounting system or anchorage as required and shall submit a certificate of compliance for review and acceptance by the registered design professional in responsible charge of the design of the designated seismic system and for approval by the authority having jurisdiction. The basis of certification shall be by actual test on a shaking table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance), or by more rigorous analysis providing for equivalent safety. The special inspector shall examine the designated seismic system and shall determine whether its anchorages and label conform with the certificate of compliance.

15.2.6 **Structural Observations.** Structural observations shall be provided for those structures included in Seismic Design Category D, when one or more of the following conditions exist:

1. The structure is included in Occupancy Category III or IV, or

2. The height of the structure is greater than 25 m above the base.

Observed deficiencies shall be reported in writing to the owner and the authority having jurisdiction.

15.2.7 **Reporting and Compliance Procedures.** Each special inspector shall furnish to the authority having jurisdiction, the registered design professional in responsible charge, the owner, the persons preparing the quality assurance plan, and the contractor copies of regular weekly progress reports of his observations, noting therein any uncorrected deficiencies and corrections of previously reported deficiencies. All deficiencies shall be brought to the immediate attention of the contractor for correction. At completion of construction, each special inspector shall submit a final report to the authority having jurisdiction certifying that all inspected work was completed substantially in accordance with approved construction documents. Work not in compliance shall be described in the final report. At completion of construction, the building contractor shall submit a final report to the authority having jurisdiction certifying that all construction work incorporated into the seismic force-resisting system and other designated seismic systems was constructed substantially in accordance with the approved construction documents and applicable workmanship requirements. Work not in compliance shall be described in the final report. The contractor shall correct all deficiencies as required.

SECTION 15.3
SUPPLEMENTARY FOUNDATION REQUIREMENTS

15.3.1 **Special Pile Requirements for Category C.** All concrete piles and concrete-filled pipe piles shall be connected to the pile cap by embedding the pile reinforcement in the pile cap for a distance equal to the development length as specified in SBC 304 as modified by Section 15.5 of this Code or by the use of field-placed dowels anchored in the concrete pile. For deformed bars, the development length is the full development length for compression or tension, in the case of uplift, without reduction in length for excess area.

Hoops, spirals, and ties shall be terminated with seismic hooks as defined in Section 21.1 of SBC 304.

Where required for resistance to uplift forces, anchorage of steel pipe (round HSS sections), concrete-filled steel pipe or H piles to the pile cap shall be made by means other than concrete bond to the bare steel section.

> **Exception:** Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile.

Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pile, provisions shall be made so that those specified lengths or extents are maintained after pile cutoff.

15.3.1.1 **Uncased Concrete Piles.** Reinforcement shall be provided where required by analysis. As a minimum, longitudinal reinforcement ratio of 0.0025 shall be provided for uncased cast-in-place concrete drilled or augered piles, piers, or caissons in the top one-third of the pile length or a minimum length of 3 m below the ground. There shall be a minimum of four longitudinal bars with closed ties (or equivalent spirals) of a minimum of a 10 mm diameter provided at 16 longitudinal-bar-diameter maximum spacing. Transverse confinement reinforcing with a maximum spacing of 150 mm or 8 longitudinal-bar-diameters, whichever is less, shall be provided in the pile within three pile diameters of the bottom of the pile cap.

15.3.1.2 **Metal-Cased Concrete Piles.** Reinforcement requirements are the same as for uncased concrete piles.

Exception: Spiral welded metal casing of a thickness not less than No. 14 gauge can be considered as providing concrete confinement equivalent to the closed ties or equivalent spirals required in an uncased concrete pile, provided that the metal casing is adequately protected against possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

15.3.1.3 **Concrete-Filled Pipe.** Minimum reinforcement 0.01 times the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment anchorage into the pile cap.

15.3.1.4 **Precast Nonprestressed Concrete Piles.** A minimum longitudinal steel reinforcement ratio of 0.01 shall be provided for precast nonprestressed concrete piles. The longitudinal reinforcing shall be confined with closed ties or equivalent spirals of a minimum 10 mm diameter. Transverse confinement reinforcing shall be provided at a maximum spacing of eight times the diameter of the smallest longitudinal bar, but not to exceed 150 mm, within three pile diameters of the bottom of the pile cap. Outside of the confinement region, closed ties or equivalent spirals shall be provided at a 16 longitudinal-bar-diameter maximum spacing, but not greater than 200 mm. Reinforcement shall be full length.

15.3.1.5 **Precast Prestressed Piles.** For the upper 6 m of precast prestressed piles, the minimum volumetric ratio of spiral reinforcement shall not be less than 0.007 or the amount required by the following formula:

$$\rho_s = \frac{0.12\, f_c'}{f_{yh}} \qquad \textbf{(Eq. 15.3.1.5-1)}$$

ρ_s = volumetric ratio (vol. spiral/vol. core)

f_c' = specified compressive strength of concrete, (MPa)

f_{yh} = specified yield strength of spiral reinforcement, which shall not be taken greater than 580 MPa

A minimum of one-half of the volumetric ratio of spiral reinforcement required by Eq. 15.3.1.5-1 shall be provided for the remaining length of the pile.

15.3.2 **Special Pile Requirements for Category D.**

15.3.2.1 **Uncased Concrete Piles.** A minimum longitudinal reinforcement ratio of 0.005

shall be provided for uncased cast-in-place drilled or augered concrete piles, piers, or caissons in the top one-half of the pile length, or a minimum length of 3m below ground, or throughout the flexural length of the pile, whichever length is greatest. The flexural length shall be taken as the length of pile to a point where the concrete section cracking moment multiplied by the resistance factor 0.4 exceeds the required factored moment at that point. There shall be a minimum of four longitudinal bars with transverse confinement reinforcement in the pile in accordance with Sections 21.4.4.1, 21.4.4.2, and 21.4.4.3 of SBC 304. Such transverse confinement reinforcement shall extend the full length of the pile in Site Classes E or F, a minimum of seven times the least pile dimension above and below the interfaces of soft to medium stiff clay or liquefiable strata, and three times the least pile dimension below the bottom of the pile cap in Site Classes other than E or F.

In other than Site Classes E or F, it shall be permitted to use a transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.4.4.1(a) of SBC 304 throughout the remainder of the pile length. Tie spacing throughout the remainder of the pile length shall not exceed 12 longitudinal-bar-diameters, one-half the diameter of the section, or 300 mm. Ties shall be a minimum of 10 mm bars for up to diameter 500 mm piles and 12 mm bars for piles of larger diameter.

15.3.2.2 **Metal-Cased Concrete Piles.** Reinforcement requirements are the same as for uncased concrete piles.

> **Exception:** Spiral welded metal-casing of a thickness not less than No. 14 gauge can be considered as providing concrete confinement equivalent to the closed ties or equivalent spirals required in an uncased concrete pile, provided that the metal casing is adequately protected against possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

15.3.2.3 **Precast Concrete Piles.** Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.4.4.1, 21.4.4.2, and 21.4.4.3 of SBC 304 for the full length of the pile.

> **Exception:** In other than Site Classes E or F, the specified transverse confinement reinforcement shall be provided within three pile diameters below the bottom of the pile cap, but it shall be permitted to use a transverse reinforcing ratio of not less than one-half of that required in Section 21.4.4.1(a) of SBC 304 throughout the remainder of the pile length.

15.3.2.4 **Precast Prestressed Piles.** In addition to the requirements for Seismic Design Category C, the following requirements shall be met:

1. Requirements of SBC 304, Chapter 21, need not apply.
2. Where the total pile length in the soil is 10 m or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 10 m, the ductile pile region shall be taken as the greater of 10 m or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.
3. In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six

times the diameter of the longitudinal strand, or 200 mm, whichever is smaller.

4. Spiral reinforcement shall be spliced by lapping one full turn by welding or by the use of a mechanical connector. Where spiral reinforcement is lap spliced, the ends of the spiral shall terminate in a seismic hook in accordance with SBC 304, except that the bend shall be not less than 135 degrees. Welded splices and mechanical connectors shall comply with Section 12.14.3 of SBC 304.

5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with:

$$\rho_s = 0.25 \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_{ch}} - 1.0 \right] \times \left[0.5 + \frac{1.4\,P}{f'_c\,A_g} \right]$$

but not less than

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \left[0.5 + \frac{1.4\,P}{f'_c\,A_g} \right]$$

and not to exceed $\rho_s = 0.021$

where

ρ_s = volumetric ratio (vol. spiral/vol. core

$f'_c \leq$ 40 MPa

f_{yh} = yield strength of spiral reinforcement 580 MPa

A_g = pile cross-sectional area, mm^2

A_{ch} = core area defined by spiral outside diameter, mm^2

P = axial load on pile resulting from the load combination 1.2D + 0.5L + 1.0E, kN

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.

6. When transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacings, and perpendicular to dimension, h_c, shall conform to:

$$A_{sh} = 0.3\, sh_c \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_{ch}} - 1.0 \right] \times \left[0.5 + \frac{1.4\,P}{f'_c\,A_g} \right]$$

but not less than

$$A_{sh} = 0.12\, sh_c \frac{f'_c}{f_{yh}} \left[0.5 + \frac{1.4\,P}{f'_c\,A_g} \right]$$

where

s = spacing of transverse reinforcement measured along length of pile, mm

h_c = cross-sectional dimension of pile core measured center-to-center of hoop reinforcement, mm

$f_{yh} \leq 480$ MPa

The hoops and cross ties shall be equivalent to deformed bars not less than 10 mm in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

Outside of the length of the pile requiring transverse reinforcement, the spiral or hoop reinforcement with a volumetric ratio not less than one-half of that required for transverse confinement reinforcement shall be provided.

15.3.2.5 **Steel Piles.** The connection between the pile cap and steel piles or unfilled steel pipe piles shall be designed for a tensile force equal to 10% of the pile compression capacity.

> **Exceptions:** Connection tensile capacity need not exceed the strength required to resist the special seismic loads of Section 10.4. Connections need not be provided where the foundation or supported structure does not rely on the tensile capacity of the piles for stability under the design seismic forces.

SECTION 15.4
SUPPLEMENTARY PROVISIONS FOR STEEL

15.4.1 **General.** The design, construction, and quality of steel components that resist seismic forces shall conform to the requirements of the references listed in Section 11.1 except as modified by the requirements of this section.

15.4.2 **Seismic Requirements for Steel Structures.** Steel structures and structural elements therein that resist seismic forces shall be designed in accordance with the requirements of Sections 15.4.3 and 15.4.4 for the appropriate Seismic Design Category.

15.4.3 **Seismic Design Categories A, B, and C.** Steel structures assigned to Seismic Design Categories A, B, and C shall be of any construction permitted by the references in Section 15.4.1. An R factor as set forth in Table 10.2 shall be permitted when the structure is designed and detailed in accordance with the requirements of Ref. 11.1-1 for structural steel buildings as modified by this chapter and Section 15.4.6 for light-framed walls. Systems not detailed in accordance with Ref. 11.1-1 shall use the R factor designated for "Systems not detailed for seismic."

15.4.4 **Seismic Design Category D.** Steel structures assigned to Seismic Design Category D shall be designed and detailed in accordance with Ref. 11.1-1 Part I or Section 15.4.6 for light-framed cold-formed steel wall systems.

15.4.5 **Cold-Formed Steel Seismic Requirements.** The design of cold-formed carbon or low-alloy steel to resist seismic loads shall be in accordance with the provisions of Ref. 11.1-2, and the design of cold-formed stainless steel structural members to resist seismic loads shall be in accordance with the provisions of Ref. 11.1-3, except as modified by this section. The references to section and paragraph numbers are to those of the particular specification modified.

15.4.5.1 Ref. 11.1-2-Revised Section A5.1.3 of Ref. 11.1-2 by deleting the reference to earthquake or seismic loads in the sentence permitting the 0.75 factor. Seismic load combinations shall be as determined by this Code.

15.4.6 **Light-Framed Wall Requirements.** Cold-formed steel stud wall systems designed in accordance with Ref. 11.1-2 or 11.1-3 shall, when required by the provisions of Sections 15.4.3 or 15.4.4, also comply with the requirements of this section.

15.4.6.1 **Boundary Members.** All boundary members, chords, and collectors shall be designed to transmit the axial force induced by the specified loads of Chapters 9 through 13.

15.4.6.2 **Connections.** Connections of diagonal bracing members, top chord splices, boundary members, and collectors shall have a design strength equal to or greater than the nominal tensile strength of the members being connected or Ω_o times the design seismic forces. The pullout resistance of screws shall not be used to resist seismic forces.

15.4.6.3 **Braced Bay Members.** In stud systems where the lateral forces are resisted by braced frames, the vertical and diagonal members of braced bays shall be anchored such that the bottom tracks are not required to resist tensile forces by bending of the track or track web. Both flanges of studs in a bracing bay shall be braced to prevent lateral torsional buckling. In braced shear walls, the vertical boundary members shall be anchored so the bottom track is not required to resist uplift forces by bending of the track web.

15.4.6.4 **Diagonal Braces.** Provision shall be made for pre-tensioning or other methods of installation of tension-only bracing to prevent loose diagonal straps.

15.4.6.5 **Shear Walls.** Nominal shear values for wall sheathing materials are given in Table 15.4.6.5. Design shear values shall be determined by multiplying the nominal values therein by a factor of 0.55. In structures over 1 story in height, the assemblies in Table 15.4.6.5 shall not be used to resist horizontal loads contributed by forces imposed by masonry or concrete construction.

Panel thicknesses shown in Table 15.4.6.5 shall be considered to be minimums. No panels less than 50 mm. wide shall be used. Plywood or oriented strand board structural panels shall be of a type that is manufactured using exterior glue. Framing members, blocking, or strapping shall be provided at the edges of all sheets. Fasteners along the edges in shear panels shall be placed not less than 10 mm in from panel edges. Screws shall be of sufficient length to ensure penetration into the steel stud by at least two full diameter threads.

The height-to-length ratio of wall systems listed in Table 15.4.6.5 shall not exceed 2:1.

Perimeter members at openings shall be provided and shall be detailed to distribute the shearing stresses. Wood sheathing shall not be used to splice these members.

Wall studs and track shall have a minimum uncoated base thickness of not less than 0.85 mm and shall not have an uncoated base metal thickness greater than 1.22 mm. Panel end studs and their uplift anchorage shall have the design strength to resist the forces determined by the seismic loads.

15.4.7 **Seismic Requirements for Steel Deck Diaphragms.** Steel deck diaphragms shall be made from materials conforming to the requirements of Ref. 11.1-2 or 11.1-3. Nominal strengths shall be determined in accordance with approved analytical procedures or with test procedures prepared by a registered design professional experienced in testing of cold-formed steel assemblies and approved by the authority having jurisdiction. Design strengths shall be determined by multiplying the nominal strength by a resistance factor, ϕ equal to 0.60 for mechanically connected diaphragms and equal to 0.50 for welded diaphragms. The steel deck installation for the building, including fasteners, shall comply with the test assembly arrangement. Quality standards established for the nominal strength test shall be the minimum standards required for the steel deck installation, including fasteners.

15.4.8 **Steel Cables.** The design strength of steel cables shall be determined by the provisions of Ref. 11.1-5 except as modified by this section. Ref. 11.1-5, Section 5d, shall be modified by substituting $1.5(T_4)$ when T_4 is the net tension in cable due to dead load, prestress, live load, and seismic load. A load factor of 1.1 shall be applied to the prestress force to be added to the load combination of Section 3.1.2 of Ref. 11.1-5.

SECTION 15.5
SUPPLEMENTAL PROVISIONS FOR CONCRETE

The supplemental provisions for Concrete have already been incorporated in SBC 304.

SECTION 15.6
SUPPLEMENTARY PROVISIONS FOR MASONRY

15.6.1 For the purposes of design of masonry structures using the earthquake loads given in this Code, several amendments to the reference standard are necessary.

15.6.2 The references to "Seismic Performance Category" in Section 1.13 and elsewhere of the reference standard shall be replaced by "Seismic Design Category".

15.6.3 The anchorage forces given in Section 1.13.3.2 of the reference standard shall not be interpreted to replace the anchorage forces given in Chapters 9 through 13 of this Code.

15.6.4 To qualify for the R factors given in Chapters 9 through 13 of this Code, the requirements of the reference standard shall be satisfied and amended as follows: Ordinary and detailed plain masonry shear walls shall be designed according to Sections 2.1 and 2.2 of the reference standard. Detailed plain masonry shear walls shall be reinforced as a minimum as required in Section 1.13.5.3.3 of the reference standard.

Ordinary, intermediate, and special reinforced masonry shear walls shall be designed according to Sections 2.1 and 2.3 of the reference standard. Ordinary and intermediate reinforced masonry shear walls shall be reinforced as a minimum as required in Section 1.13.5.3.3 of the reference standard. In addition, intermediate reinforced masonry shear walls shall have vertical reinforcing bars spaced no farther apart than 1200 mm. Special reinforced masonry shear walls shall be reinforced as a minimum as required in Sections 1.13.6.3 and 1.13.6.4 of the reference standard.

Special reinforced masonry shear walls shall not have a ratio of reinforcement ρ, that exceeds that given by either Method A or B below:

15.6.4.1 **Method A**. Method A is permitted to be used where the story drift does not exceed 0.010h, as given in Table 10.12 and if the extreme compressive fiber strains are less than 0.0035 mm/mm for clay masonry and 0.0025 mm/mm for concrete masonry.

1. When walls are subjected to in-plane forces, and for columns and beams, the critical strain condition corresponds to a strain in the extreme tension reinforcement equal to five times the strain associated with the reinforcement yield stress, f_y.
2. When walls are subjected to out-of-plane forces, the critical strain condition corresponds to a strain in the reinforcement equal to 1.3 times the strain associated with reinforcement yield stress, f_y.

The strain at the extreme compression fiber shall be assumed to be 0.0035 mm/mm for clay masonry and 0.0025 mm/mm for concrete masonry.

The calculation of the maximum reinforcement ratio shall include factored gravity axial loads. The stress in the tension reinforcement shall be assumed to be 1.25 f_y. Tension in the masonry shall be neglected. The strength of the compressive zone shall be calculated as 80% of the area of the compressive zone. Stress in reinforcement in the compression zones shall be based on a linear strain distribution.

15.6.4.2 **Method B.** Method B is permitted to be used where story drift does not exceed $0.013h_{sx}$ as given in Table 10.12.

1. Boundary members shall be provided at the boundaries of shear walls when the compressive strains in the wall exceed 0.002. The strain shall be determined using factored forces and R equal to 1.5.
2. The minimum length of the boundary member shall be three times the thickness of the wall, but shall include all areas where the compressive strain per Section 15.6.4.2 item 1, is greater than 0.002.
3. Lateral reinforcement shall be provided for the boundary elements. The lateral reinforcement shall be a minimum of 10 mm closed ties at a maximum spacing of 200 mm on center within the grouted core, or equivalent approved confinement, to develop an ultimate compressive strain of at least 0.006.
4. The maximum longitudinal reinforcement ratio shall not exceed $0.15\frac{f'_m}{f_y}$.

15.6.5 Where allowable stress design is used for load combinations including earthquake, Eq. 2.4.1-3 and Eq. 2.4.1-5 of Section 2.4.1 of this Code shall replace combinations (c) and (e) of Section 2.1.1.1.1 of the reference standard.

TABLE 15.4.6.5: NOMINAL SHEAR VALUES FOR SEISMIC FORCES FOR SHEAR WALLS FRAMED WITH COLD-FORMED STEEL STUDS (IN kN/m)[a.b]

Assembly Description	Fastener Spacing at Panel Edges[c] (mm)				Framing Spacing (mm o.c.)
	150	100	75	50	
12 mm rated structural I sheathing (4-ply) plywood one side[d]	11.4	14.4	21.4	23.7	600
12 mm oriented strand board one side[d]	10.2	13.3	18.6	23.7	600

[a] Nominal shear values shall be multiplied by the appropriate strength reduction factor to determine design strength as set forth in 15.4.6.5.

[b] Studs shall be a minimum 40 mm x 90 mm with a 10 mm return lip. Track shall be a minimum 30 mm x 90 mm. Both studs and track shall have a minimum uncoated base metal thickness of 0.85 mm and shall be ASTM A653 SS Grade 33, ASTM A792 SS Grade 33, or ASTM A875 SS Grade 33. Framing screws shall be No. 8 x 16 mm wafer head self-drilling. Plywood and OSB screws shall be a minimum No. 8 x 25 mm bugle head. Where horizontal straps are used to provide blocking, they shall be a minimum 40 mm wide and of the same material and thickness as the stud and track.

[c] Screws in the field of the panel shall be installed 300 mm on center unless otherwise shown.

[d] Both flanges of the studs shall be braced in accordance with Section 15.4.6.3.

CHAPTER 16
EXISTING BUILDING PROVISIONS

SECTION 16.1
ADDITIONS TO EXISTING STRUCTURES

16.1.0 Additions shall be made to existing structures only as follows:

16.1.1 Additions shall not be permitted unless the existing structures have been designed according to this code.

16.1.2 An addition that is structurally independent from an existing structure shall be designed and constructed in accordance with the seismic requirements for new structures.

16.1.3 An addition that is not structurally independent from an existing structure shall be designed and constructed such that the entire structure conforms to the seismic force-resistance requirements for new structures unless the following three conditions are complied with:

1. The addition shall comply with the requirements for new structures.
2. The addition shall not increase the seismic forces in any structural element of the existing structure by more than 5% unless the capacity of the element subject to the increased forces is still in compliance with these provisions.
3. The addition shall not decrease the seismic resistance of any structural element of the existing structure unless the reduced resistance is equal to or greater than that required for new structures.

SECTION 16.2
CHANGE OF USE

When a change of use results in a structure being reclassified to a higher Occupancy Category, the structure shall conform to the seismic requirements for new construction.

Exceptions:

1. When a change of use results in a structure being reclassified from Occupancy Category I or II to Occupancy Category III and the structure is located in a seismic map area where S_{DS} < 0.33, compliance with these provisions is not required.
2. Specific seismic detailing provisions of Chapter 15 required for a new structure are not required to be met when it can be shown that the level of performance and seismic safety is equivalent to that of a new structure. Such analysis shall consider the regularity, over-strength, redundancy, and ductility of the structure within the context of the existing and retrofit (if any) detailing provided.

SECTION 16.3
MULTIPLE USES

Structures having multiple uses shall be assigned the classification of the use having the highest Occupancy Category except in structures having two or more portions which are structurally separated in accordance with Section 10.12, each portion shall be separately classified. Where a structurally separated portion of a structure provides access to, egress from, or shares life safety components with another portion having a higher Occupancy Category, both portions shall be assigned the higher Occupancy Category.

REFERENCED STANDARDS

These are the standards referenced within SBC 301. The standards are listed herein by the promulgating agency of the standard, the standard identification, the effective date and title. The application of the referenced standards shall be as specified in SBC.

Ref. 4-1 ANSI. (1988). "American National Standard Practice for the Inspection of Elevators, Escalators, and Moving Walks (Inspectors' Manual)." ANSI A17.2.

Ref. 4-2 ANSI/ASME. (1993). "American National Standard Safety Code for Elevators and Escalators." ANSI/ASME A17.1.

Ref. 5-1 ASCE. (1998). "Flood Resistant Design and Construction." SEI/ASCE 24-98.

Ref. 7.4-1 Standard Test Method for Performance of Exterior Windows, Curtain Walls, Doors and Storm Shutters Impacted by Missile(s) and Exposed to Cyclic Pressure Differentials, ASTM E1886-97, ASTM Inc., West Conshohocken, PA, 1997.

Ref. 7.4-2 Specification Standard for Performance of Exterior Windows, Glazed Curtain Walls, Doors and Storm Shutters Impacted by Windborne Debris in Hurricanes, ASTM E 1996-99, ASTM Inc., West Conshohocken, PA, 1999.

Ref. 11.1-1 American Institute of Steel Construction, Seismic Provisions for Structural Steel Buildings, Part I, 1997, including Supplement 2, November 10, 2000.

Ref. 11.1-2 American Iron and Steel Institute (AISI), Specification for the Design of Cold-Formed Steel Structural Members, 1996, including Supplement No. 1, July 30, 1999.

Ref. 11.1-3 ASCE, Specification for the Design of Cold-Formed Stainless Steel Structural Members, ASCE 8-90, 1990.

Ref. 11.1-4 Steel Joist Institute, Standard Specification, Load Tables and Weight Tables for Steel Joists and Joist Girders, 1994.

Ref. 11.1-5 ASCE, Structural Applications for Steel Cables for Buildings, ASCE 19-95, 1995.

Ref. 11.3-1 American Institute of Steel Construction (AISC), Seismic Provisions for Structural Steel Buildings, Including Supplement No. 1 (February 15, 1999, July 1997, Parts I and II.

Ref. 11.3-2 American Iron and Steel Institute (AISI), Specification for the Design of Cold-Formed Steel Structural Members, 1996, including Supplement 2000.

Ref. 11.4-1 American Concrete Institute, Building Code Requirements for Masonry Structures, ACI 530-99/ASCE 5-99/TMS 402-99, 1999 and Specifications for Masonry Structures, ACI 530.1-99/ ASCE 6-99/TMS 602-99, 1999.

Ref. 12-1 American Society of Mechanical Engineers (ASME), ASME A17.1, Safety Code For Elevators and Escalators, 1996.

Ref. 12-2 American Society of Mechanical Engineers (ASME), Boiler And Pressure Vessel Code, including addendums through 1997.

Ref. 12-3 American Society For Testing and Materials (ASTM), ASTM C635, Standard Specification for the Manufacture, Performance, and Testing of Metal Suspension Systems For Acoustical Tile And Lay-in Panel Ceilings, 1997.

Ref. 12-4 American Society For Testing And Materials (ASTM), ASTM C636, Standard Practice for Installation of Metal Ceiling Suspension Systems for Acoustical Tile And Lay-in Panels, 1996.

Ref. 12-5 American National Standards Institute/American Society :of Mechanical Engineers, ASME B31.1-98, Power Piping.

Ref. 12-6 American Society of Mechanical Engineers, ASME B31.3-96, Process Piping.

Ref. 12-7 American Society of Mechanical Engineers, ASME B31.4-92, Liquid Transportation Systems for Hydrocarbons, Liquid Petroleum Gas, Anhydrous Ammonia, and Alcohols.

Ref. 12-8 American Society of Mechanical Engineers, ASME B31.5-92, Refrigeration Piping.

Ref. 12-9 American Society of Mechanical Engineers, ASME B31.9-96, Building Services Piping.

Ref. 12-10 American Society of Mechanical Engineers, ASME B31.11-89 (Reaffirmed 1998), Slurry Transportation Piping Systems.

Ref. 12-11 American Society of Mechanical Engineers, ASME B31.8-95, Gas Transmission and Distribution Piping Systems.

Ref. 12-12 Institute of Electrical and Electronic Engineers (IEEE), Standard 344, Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations, 1987.

Ref. 12-13 National Fire Protection Association (NFPA), NFPA-13, Standard for the Installation of Sprinkler Systems, 1999.

Ref. 12-14 American Society of Heating, Ventilating, and Air Conditioning (ASHRAE), "Seismic Restraint Design," 1999.

Ref. 12-15 Manufacturer's Standardization Society of the Valve and Fitting Industry (MSS). SP-58, "Pipe-hangers and Supports - Materials, Design, and Manufacture," 1988.

Ref. 12-16 Ceilings and Interior Systems Construction Association (CISCA), "Recommendations for Direct-Hung Acoustical Tile and Lay-in Panel Ceilings," Seismic Zones 0-2, 1991.

Ref. 12-17 Ceilings and Interior Systems Construction Association (CISCA), "Recommendations for Direct-Hung Acoustical Tile and Lay-in Panel Ceilings," Seismic Zones 3-4, 1991.

Ref. 12-18 Sheet Metal and Air Conditioning Contractors National Association (SMACNA), HVAC Duct Construction Standards, Metal and Flexible, 1995.

Ref. 12-19 Sheet Metal and Air Conditioning Contractors National Association (SMACNA), Rectangular Industrial Duct Construction Standards, 1980.

Ref. 12-20 Sheet Metal and Air Conditioning Contractors National Association (SMACNA), Seismic Restraint Manual Guidelines for Mechanical Systems, 1991, including Appendix B, 1998.

Ref. 12-21 American Architectural Manufacturers Association (AAMA), "Recommended Dynamic Test Method for Determining the Seismic Drift Causing Glass Fallout from a Wall System," Publication No. AAMA 501.6-2001.

Ref. 13.2-1 ACI. (1995). "Standard Practice for the Design and Construction of Cast-In-Place Reinforced Concrete Chimneys." ACI 307.

Ref. 13.2-2 ACI. (1997). "Standard Practice for the Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials." ACI 313.

Ref. 13.2-3 ACI. (1999). "Standard Practice for the Seismic Design of Liquid-Containing Concrete Structures." ACI 350.3/350.3R.

Ref. 13.2-4 ACI. (1998). "Guide to the Analysis, Design, and Construction of Concrete-Pedestal Water Towers." ACI 371R.

Ref. 13.2-5 ANSI "Safety Requirements for the Storage and Handling of Anhydrous

Ammonia." ANSI K61.1.

Ref. 13.2-6 American Petroleum Institute (API). (December 1998). "Design and Construction of Large, Welded, Low Pressure Storage Tanks." API 620, 9th Edition, Addendum 3.

Ref. 13.2-7 API. (March 2000). "Welded Steel Tanks For Oil Storage." API 650, 10th Edition, Addendum 1.

Ref. 13.2-8 API. (December 1999). "Tank Inspection, Repair, Alteration, and Reconstruction." ANSI/API 653, 2nd Edition, Addendum 4.

Ref. 13.2-9 API. (May 1995). "Design and Construction of Liquefied Petroleum Gas Installation." ANSI/API 2510, 7th Edition.

Ref. 13.2-10 API. (February 1995). "Bolted Tanks for Storage of Production Liquids." Specification 12B, 14th Edition.

Ref. 13.2-11 ASME. (2000). "Boiler and Pressure Vessel Code." Including Addenda through 2000.

Ref. 13.2-12 ASME. (1992). "Steel Stacks." ASME STS-1.

Ref. 13.2-13 ASME. (1995). "Gas Transmission and Distribution Piping Systems" ASME B31.8.

Ref. 13.2-14 ASME. (1999). "Welded Aluminum-Alloy Storage Tanks." ASME B96.1.

Ref. 13.2-15 American Water Works Association (AWWA). (1997). "Welded Steel Tanks for Water Storage." ANSI/AWWA D100.

Ref. 13.2-16 AWWA. (1997). "Factory-Coated Bolted Steel Tanks for Water Storage." ANSI/AWWA D103.

Ref. 13.2-17 AWWA. (1995). "Wire- and Strand-Wound Circular for Prestressed Concrete Water Tanks." ANSI/AWWA D110.

Ref. 13.2-18 AWWA. (1995). "Circular Prestressed Concrete Tanks with Circumferential Tendons." ANSI/AWWA DI15.

Ref. 13.2-19 National Fire Protection Association (NFPA). (2000). "Flammable and Combustible Liquids Code." ANSI/NFPA 30.

Ref. 13.2-20 NFPA. (2001). "Storage and Handling of Liquefied Petroleum Gas." ANSI/NFPA 58.

Ref. 13.2-21 NFPA. (2001). "Storage and Handling of Liquefied Petroleum Gases at Utility Gas Plants." ANSI/NFPA 59.

Ref. 13.2-22 NFPA. (2001). "Production, Storage, and Handling of Liquefied Natural Gas (LNG)." ANSI/NFPA 59A.

Ref. 13.2-23 ASTM. (1992). "Standard Practice for the Design and Manufacture of Amusement Rides and Devices." ASTM F1159.

Ref. 13.2-24 ASTM. (1998). "Standard Guide for Design and Construction of Brick Liners for Industrial Chimneys." ASTM C1298

Ref. 13.2-25 Rack Manufacturers Institute (RMI). (1997). "Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks."

Ref. 13.2-26 U.S. Department of Transportation (DOT). "Pipeline Safety Regulations." Title 49CFR Part 193.

Ref. 13.2-27 U.S. Naval Facilities Command (NAVFAC). "The Seismic Design of Waterfront Retaining Structures." NAVFAC R-939.

Ref. 13.2-28 U.S. Naval Facilities Engineering Command. "Piers and Wharves." NAVFAC DM-25.1

Ref. 13.2-29 U.S. Army Corps of Engineers (USACE). (1992). "Seismic Design for Buildings." Army TM 5-809-10/ NAVFAC P-355/Air Force AFM 88-3, Chapter 13.

Ref. 13.2-31 Compressed Gas Association (CGA). (1999). "Guide for Flat-Bottomed LOX/LIN/LAR Storage Tank Systems." 1st Edition.

Ref. 15.2.2-1 ANSI. (1998). "Structural Welding Code-Steel." ANSI/AWS D1.1-98.

Ref. 15.2.2-2 ASTM. (1990). "Specification for Straight Beam Ultrasound Examination of Steel Plates." ASTM A435-90.

Ref. 15.2.2-3 ASTM. (1991). "Specification for Straight Beam Ultrasound Examination for Rolled Steel Shapes." ASTM A898-91.

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Gratitude

In appreciation and gratitude
to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence
& Aviation and Inspector General

For their continuous support and gracious consideration,
the Saudi Building Code National Committee (SBCNC)
is honored to present the first issue of
the Saudi Building Code (SBC).

Saudi Building Code Requirements

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PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11th June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Structural Requirements for Testing and Inspection (SBC 302) were developed based on ICC code in addition to American Concrete Institute (ACI) materials. ACI grants permission to the SBCNC to include ACI materials in the SBC, and ACI is not responsible for any modifications or changes that SBCNC has made to accommodate local conditions.

The development process of SBC 302 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were implemented. These changes range from replacing sections with new ones, inserting additional sections such as precast concrete constructions, and special inspections for wind requirements, removing irrelevant topics such as wood construction, and wall panels and veneers.

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** Partially.

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CHAPTER 1
GENERAL

SECTION 1.1
SCOPE

1.1.0 The Saudi Building Code for Structural Tests and Inspections referred to as SBC 302, provides minimum requirements for Structural Tests and Inspections. SBC 302 shall govern the quality, workmanship and requirements for:

1. Approval procedures of inspection agencies, fabricators and others required in SBC 302,
2. Special inspection of the materials, installation, fabrication, erection or placement of components and connections requiring special expertise to ensure compliance with approved construction documents and referenced standards,
3. Seismic and wind resistant constructions,
4. New building materials or method of construction not provided for in the SBC.

Materials of construction and tests shall conform to the applicable standards listed in SBC 301 through SBC 306.

SECTION 1.2
DEFINITIONS

1.2.0 The following words and terms shall, for the purposes of this code and as used elsewhere in the SBC, have the meanings shown herein.

Approved agency. An established and recognized agency regularly engaged in conducting tests or furnishing inspection services, when such agency has been approved.

Approved fabricator. An established and qualified person, firm or corporation approved by the building official pursuant to this code.

Certificate of compliance. A certificate stating that materials and products meet specified standards or that work was done in compliance with approved construction documents.

Fabricated item. Structural, load-bearing or lateral load-resisting assemblies consisting of materials assembled prior to installation in a building or structure, or subjected to operations such as heat treatment, thermal cutting, cold working or reforming after manufacture and prior to installation in a building or structure. Materials produced in accordance with standard specifications referenced by this code, such as rolled structural steel shapes, steel-reinforcing bars, masonry units and plywood sheets, shall not be considered "fabricated items."

Inspection certificate. An identification applied on a product by an approved agency containing the name of the manufacturer, the function and performance characteristics, and the name and identification of an approved agency that indicates that the product or material has been inspected and evaluated by an approved agency (see 1.5.5 and "Label", "Manufacturer's designation" and "Mark").

Label. An identification applied on a product by the manufacturer that contains the name of the manufacturer, the function and performance characteristics of the product or material, and the name and identification of an approved agency and that indicates that the representative sample of the product or material has been tested and evaluated by an approved agency (see 1.5.5 and "Inspection certificate", "Manufacturer's designation" and "Mark").

Manufacturer's designation. An identification applied on a product by the manufacturer indicating that a product or material complies with a specified standard or set of rules (see also "Inspection certificate", "Label" and "Mark").

Mark. An identification applied on a product by the manufacturer indicating the name of the manufacturer and the function of a product or material (see also "Inspection certificate," "Label" and "Manufacturer's designation").

Special conditions not covered by the structural code. New building materials, equipment, appliances, systems or methods of construction not covered by the SBC, and any material of questioned suitability proposed for use in the construction of a building or structure.

Special inspection. Inspection as herein required of the materials, installation, fabrication, erection or placement of components and connections requiring special expertise to ensure compliance with approved construction documents and referenced standards. The special inspections shall be carried out by certified special inspectors or approved agencies.

Special inspection, continuous. The full-time observation of work requiring special inspection by an approved special inspector who is present in the area where the work is being performed.

Special inspection, periodic. The part-time or intermittent observation of work requiring special inspection by an approved special inspector who is present in the area where the work has been or is being performed and at the completion of the work.

Sprayed fire-resistant materials. Cementitious or fibrous materials that are spray applied to provide fire-resistant protection of the substrates.

SECTION 1.3
INSPECTIONS

1.3.1 **General.** Construction or work for which a permit is required shall be subject to inspection by the building official and such construction or work shall remain accessible and exposed for inspection purposes until approved. Approval as a result of an inspection shall not be construed to be an approval of a violation of the provisions of this code or of other ordinances of the jurisdiction. Inspections presuming to give authority to violate or cancel the provisions of this code or of other ordinances of the jurisdiction shall not be valid. It shall be the duty of the permit applicant to cause the work to remain accessible and exposed for inspection purposes. Neither the building official nor the jurisdiction shall be liable for expense entailed in the removal or replacement of any material required to allow inspection.

1.3.2 **Preliminary inspection.** Before issuing a permit, the building official is authorized to examine or cause to be examined buildings, structures, sites and approved drawings and specifications for which an application has been filed.

1.3.3 **Required inspections.** The building official, upon notification, shall make the inspections set forth in Sections 1.3.3.1 through 1.3.3.6.

1.3.3.1 **Footing and foundation inspection.** Footing and foundation inspections shall be made after excavations for the foundations are complete and any required reinforcing steel is in place. Inspection shall include shoring of adjacent building structures, dewatering, and compaction if required. For concrete foundations, any required forms shall be in place prior to inspection. Materials for the foundation shall be on the job, except where concrete is ready mixed in accordance with ASTM C 94, the concrete need not be on the job.

1.3.3.2 **Concrete slab and under-floor inspection.** Concrete slab and under-floor inspections shall be made after in-slab or under-floor reinforcing steel and building service equipment, conduit, piping accessories and other ancillary equipment items are in place, but before any concrete is placed or floor sheathing installed, including the sub floor.

1.3.3.3 **Fire-resistant penetrations.** Protection of joints and penetrations in fire-resistance-rated assemblies shall not be concealed from view until inspected and approved.

1.3.3.4 **Energy efficiency inspections.** Inspections shall be made to determine compliance with SBC 601 and shall include, but not be limited to, inspections for: envelope insulation *R* and *U* values, fenestration *U* value, duct system *R* value, and HVAC and water-heating equipment efficiency.

1.3.3.5 **Special inspections.** For special inspections, see Chapter 2.

1.3.3.6 **Final inspection.** The final inspection shall be made after all work required by the building permit is completed.

1.3.4 **Inspection agencies.** The building official is authorized to accept reports of approved inspection agencies, provided such agencies satisfy the requirements as to qualifications and reliability.

1.3.5 **Inspection requests.** It shall be the duty of the holder of the building permit or their duly authorized agent to notify the building official when work is ready for inspection. It shall be the duty of the permit holder to provide access to and means for inspections of such work that are required by this code.

1.3.6 **Approval required.** Work shall not be done beyond the point indicated in each successive inspection without first obtaining the approval of the building official. The building official, upon notification, shall make the requested inspections and shall either indicate the portion of the construction that is satisfactory as completed, or notify the permit holder or his or her agent wherein the same fails to comply with this code. Any portions that do not comply shall be corrected and such portion shall not be covered or concealed until authorized by the building official.

SECTION 1.4
ALTERNATIVE MATERIALS, DESIGN AND METHODS OF CONSTRUCTION AND EQUIPMENT

1.4.1 **General.** The provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety.

1.4.1.1 **Research reports.** Supporting data, where necessary to assist in the approval of materials or assemblies not specifically provided for in this code, shall consist of valid research reports from approved sources.

1.4.1.2 **Tests.** Whenever there is insufficient evidence of compliance with the provisions of this code, or evidence that a material or method does not conform to the requirements of this code, or in order to substantiate claims for alternative materials or methods, the building official shall have the authority to require tests as evidence of compliance to be made at no expense to the jurisdiction. Test methods shall be as specified in this code or by other recognized test standards. In the absence of recognized and accepted test methods, the building official shall approve the testing procedures. Tests shall be performed by an approved agency. Reports of such tests shall be retained by the building official for the period required for retention of public records.

SECTION 1.5
APPROVALS

1.5.1 **Approved agency.** An approved agency shall provide all information as necessary for the building official to determine that the agency meets the applicable requirements.

1.5.1.1 **Independent.** An approved agency shall be objective and competent. The agency shall also disclose possible conflicts of interest so that objectivity can be confirmed.

1.5.1.2 **Equipment.** An approved agency shall have adequate equipment to perform required tests. The equipment shall be periodically calibrated.

1.5.1.3 **Personnel.** An approved agency shall employ (experienced professional engineers and qualified technicians) educated in conducting, supervising and evaluating tests and/or inspections.

1.5.2 **Written approval.** Any material, appliance, equipment, system or method of construction meeting the requirements of SBC shall be approved in writing after satisfactory completion of the required tests and submission of required test reports.

1.5.3 **Approved record.** For any material, appliance, equipment, system or method of construction that has been approved, a record of such approval, including the conditions and limitations of the approval, shall be kept on file in the building official's office and shall be open to public inspection at appropriate times.

1.5.4 **Performance.** Specific information consisting of test reports conducted by an approved testing agency in accordance with standards sited in the Saudi Building Code, or other such information as necessary, shall be provided for the building official to determine that the material meets the applicable code requirements.

1.5.4.1 **Research and investigation.** Sufficient technical data shall be submitted to the building official to substantiate the proposed use of any material or assembly. If it is determined that the evidence submitted is satisfactory proof of performance for the use intended, the building official shall approve the use of the material or assembly subject to the requirements of SBC. The cost offsets, reports and investigations required under these provisions shall be paid by the permit applicant.

1.5.4.2 **Research reports.** Supporting data, where necessary to assist in the approval of materials or assemblies not specifically provided for in SBC, shall consist of valid research reports from approved sources.

1.5.5 **Labeling.** Where materials or assemblies are required by SBC to be labeled, such materials and assemblies shall be labeled by an approved agency in accordance with 1.5.1. Products and materials required to be labeled shall be labeled in accordance with the procedures set forth in Sections 1.5.5.1 through 1.5.5.3.

1.5.5.1 **Testing.** An approved agency shall test a representative sample of the product or material being labeled to the relevant standard or standards. The approved agency shall maintain a record of the tests performed. The record shall provide sufficient details to verify compliance with the test standard.

1.5.5.2 **Inspection and identification.** The approved agency shall periodically perform an inspection, which shall be in-plant if necessary, of the product or material that is to be labeled. The inspection shall verify that the labeled product or material is representative of the product or material tested.

1.5.5.3 **Label information.** The label shall contain the manufacturer's or distributor's identification, model number, serial number or definitive information describing the product or material's performance characteristics and approved agency's identification.

1.5.6 **Evaluation and follow-up inspection services.** Where structural components or other items regulated by SBC are not visible for inspection after completion of a prefabricated assembly, the permit applicant shall submit a report of each prefabricated assembly. The report shall indicate the complete details of the assembly, including a description of the assembly and its components, the basis upon which the assembly is being evaluated, test results and similar information and other data as necessary for the building official to determine conformance to SBC. Such a report shall be approved by the building official.

1.5.6.1 **Follow-up inspection.** The permit applicant shall provide for special inspections of fabricated items in accordance with Section 2.2.

1.5.6.2 **Test and inspection records.** Records of test and special inspection shall be preserved by the special inspector (inspection agency) for 5 years after completion of the project. Copies of necessary test and inspection records shall be filed with the building official.

CHAPTER 2
SPECIAL INSPECTIONS

SECTION 2.1
GENERAL

2.1.0 Where application is made for construction as described in this chapter, the owner or the registered design professional in responsible charge acting as the owner's agent shall employ one or more special inspectors to provide inspections during construction on the types of work listed under Chapter 2. The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for inspection of the particular type of construction or operation requiring special inspection. These inspections are in addition to the inspections specified in Section 1.3.

Exceptions:

1. Special inspections are not required for work of a minor nature as approved by the building official.
2. Unless otherwise required by the building official, special inspections are not required for occupancies in Group R-3 and occupancies in Group U that are accessory to a residential occupancy including, but not limited to, those listed in SBC 201.

2.1.1 **Building permit requirement.** The permit applicant shall submit a statement of special inspections prepared by the registered design professional in responsible charge in accordance with construction documents and other related provisions, as a condition for permit issuance. This statement shall include a complete list of materials and work requiring special inspections by this chapter, the inspections to be performed and a list of the individuals, approved agencies or firms intended to be retained for conducting such inspections.

2.1.2 **Report requirement.** Special inspectors shall keep records of inspections. The special inspector shall furnish inspection reports to the building official, and to the registered design professional in responsible charge. Reports shall indicate that work inspected was done in conformance to approved construction documents. Discrepancies shall be brought to the immediate attention of the contractor for correction. If the discrepancies are not corrected, the discrepancies shall be brought to the attention of the building official and to the registered design professional in responsible charge prior to the completion of that phase of the work. A final report documenting required special inspections and correction of any discrepancies noted in the inspections shall be submitted no later than 3 months of the completion of the work.

SECTION 2.2
INSPECTION OF FABRICATORS

2.2.0 Where fabrication of structural load-bearing members and assemblies is being performed on the premises of a fabricator's shop, special inspection of the fabricated items shall be required by this chapter and as required elsewhere in SBC.

2.2.1 **Fabrication and implementation procedures.** The special inspector shall verify that the fabricator maintains detailed fabrication and quality control procedures that provide a basis for inspection control of the workmanship and the fabricator's ability to conform to approved construction documents and referenced standards. The special inspector shall review the procedures for completeness and adequacy relative to the code requirements for the fabricator's scope of work.

Exception: Special inspections as required by Section 2.2 shall not be required where the fabricator is approved in accordance with Section 2.2.2.

2.2.2 **Fabricator approval.** Special inspections required by this code are not required where the work is done on the premises of a fabricator registered and approved to perform such work without special inspection. Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an approved special inspection agency. At completion of fabrication, the approved fabricator shall submit a certificate of compliance to the building official stating that the work was performed in accordance with the approved construction documents.

SECTION 2.3
STEEL CONSTRUCTION

2.3.0 The special inspections for steel elements of buildings and structures shall be as required by this section and Table 2.3.

Exceptions:

1. Special inspection of the steel fabrication process shall not be required where the fabricator does not perform any welding, thermal cutting or heating operation of any kind as part of the fabrication process. In such cases, the fabricator shall be required to submit a detailed procedure for material control that demonstrates the fabricator's ability to maintain suitable records and procedures such that, at any time during the fabrication process, the material specification, grade and mill test reports for the main stress-carrying elements are capable of being determined.
2. The special inspector need not be continuously present during welding of the following items, provided the materials, welding procedures and qualifications of welders are verified prior to the start of the work; periodic inspections are made of the work in progress and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding.
   a. Single-pass fillet welds not exceeding 8 mm in size.
   b. Floor and roof deck welding.
   c. Welded studs when used for structural diaphragm.
   d. Welded sheet steel for cold-formed steel framing members such as studs and joists.
   e. Welding of stairs and railing systems.

2.3.1 **Welding.** Welding inspection shall be in compliance with AWS D1.1. The basis for welding inspector qualification shall be AWS D1.1.

2.3.2 **Details.** The special inspector shall perform an inspection of the steel frame to verify compliance with the details shown on the approved construction documents, such as bracing, stiffening, member locations and proper application of joint details at each connection.

2.3.3 **High-strength bolts.** Installation of high-strength bolts shall be periodically inspected in accordance with AISC specifications.

2.3.3.1 **General.** While the work is in progress, the special inspector shall determine that the requirements for bolts, nuts, washers and paint; bolted parts and installation and tightening in such standards are met. For bolts requiring pretensioning, the special inspector shall observe the pre-installation testing and calibration procedures when such procedures are required by the installation method or by project plans or specifications; determine that all plies of connected materials have been drawn together and properly snugged and monitor the installation of bolts to verify that the selected procedure for installation is properly used to tighten bolts. For joints required to be tightened only to the snug-tight condition, the special inspector need only verify that the connected materials have been drawn together and properly snugged.

2.3.3.2 **Periodic monitoring.** Monitoring of bolt installation for pretensioning is permitted to be performed on a periodic basis when using the turn-of-nut method with match marking techniques, the direct tension indicator method or the alternate design fastener (twist-off bolt) method. Joints designated as snug tight need be inspected only on a periodic basis.

2.3.3.3 **Continuous monitoring.** Monitoring of bolt installation for pretensioning using the calibrated wrench method or the turn-of-nut method without match marking shall be performed on a continuous basis.

TABLE 2.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS during task listed	PERIODIC during task listed	REFERENCE STANDARD
1. Material verification of high-strength bolts, nuts and washers: a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material specifications; AISC 335, Section A3.4; SBC 306, Section 1.3.3
b. Manufacturer's certificate of compliance required.	—	X	
2. Inspection of high-strength bolting. a. Bearing-type connections. b. Slip-critical connections.	— X	X	SBC 306, Section 13.2.5& SBC 302 (2.3.3)
3. Material verification of structural steel: a. Identification markings to conform to ASTM standards specified in the approved construction documents. b. Manufacturers' certified mill test reports.	— —	X X	ASTM A 6 or ASTM A 568 & SBC 302 (3.3.4) ASTM A 6 or ASTM A 568
4. Material verification of weld filler materials: a. Identification markings to conform to AWS specification in the approved construction documents. b. Manufacturer's certificate of compliance required.	— —	X X	AISC ASD, Section A3.6; SBC 306, Section 1.3.5
5. Inspection of welding: a. Structural steel: 1) Complete and partial penetration groove welds 2) Multipass fillet welds. 3) Single-pass fillet welds > 8 mm 4) Single-pass fillet welds ≤ 8 mm 5) Floor and deck welds.	X X X — —	— — — X X	AWS D1.1 & SBC 302 (2.3.1) AWS D1.3
6. Inspection of steel frame joint details for compliance with approved construction documents: a. Details such as bracing and stiffening. b. Member locations. c. Application of joint details at each connection.	— — —	— — —	SBC 302 (2.3.2)

SECTION 2.4
CONCRETE CONSTRUCTION

2.4.0 **Concrete construction.** The special inspections and verifications for concrete construction for compliance with approved design drawings and with SBC requirements shall be as required by this section and Table 2.4.

Exception: Special inspections shall not be required for minor constructions such as fence walls, driveways and nonstructural concrete elements.

2.4.1 **Materials.** In the absence of sufficient data or documentation providing evidence of conformance to quality standards for materials, the building official shall require testing of materials in accordance with the appropriate standards and criteria for the material in Section 3.0 SBC 304.

TABLE 2.4
REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS during task listed	PERIODIC during task listed	REFERENCE STANDARD
1. Inspection of structural layout and members dimensions for compliance with approved design drawings and construction tolerances	–	X	–
2. Inspection of reinforcing steel, including prestressing tendons, and placement.		X	SBC 304 (3.5, 7.1-7.7)
3. Inspection of formwork		X	SBC 304 (6.1-6.2)
4. Inspection of concrete mix a. Verify the use of required design mix b. Verify that the delivery is from certified batch plant		 X X	 SBC 304 (4, 5.2-5.4)
5. At the time fresh concrete is sampled to fabricate specimens for strength tests, perform slump and determine the temperature of the concrete.	X		ASTM C 172 ASTM C 31 SBC 304 (5.6, 5.8)
6. Inspection of concrete placement for proper application techniques.	X		SBC 304 (5.9, 5.10)
7. Inspection of curing techniques and hot weather requirements.		X	SBC 304 (5.11-5.13)
8. Inspection of prestressed concrete: a. Application of prestressing forces. b. Grouting of bonded prestressing tendons in the seismic-force-resisting system.	 X X		 SBC 304 (18.20) SBC 304 (18.18.4)
9. Verification of in-situ concrete strength, prior to stressing of tendons in post-tensioned concrete and prior to removal of shores and forms from beams and structural slabs.		X	SBC 304 (6.2)

SECTION 2.5
PRECAST CONCRETE CONSTRUCTION

2.5.0 The special inspections and verifications for the precast concrete elements for compliance with the approved design drawings, shop drawings, the approved manufacturer procedure and SBC requirements shall be as required by this section and Table 2.5.

Exception: Special inspection shall not be required for non-structural precast elements such as curb stones, and pavers.

TABLE 2.5
REQUIRED VERIFICATION AND INSPECTION OF PRECAST CONCRETE CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS during task listed	PERIODIC during task listed	REFERENCE STANDARD
1. Shop drawings shall be reviewed for compliances with the contract specifications and drawings prior to fabrication.		X	
2. Form work, rebar, location of lifting hooks and embedded items including pre-stressed tendons and post tension duct.		X	SBC 304 (3.5, 7.1-7.7, 16.7)
3. Concrete mix design approval and verify the use of the approved concrete mix.		X	SBC 304 (4, 5.2-5.4)
4.			
a. Conveying and placing concrete techniques.		X	SBC 304 (5.9 & 5.10)
b. Samples of the concrete to determine slump test, temperature, air and the strength of the mix.		X	ACI 301, Section 1.6.4.2.d, e, f & g
5. Precast with pretension			
a. Verification of tendons for prestressed reinforcement.		X	SBC 304 (3.5)
b. Verification tensile stresses in the tendons and measurement of tendon elongation.		X	SBC 304 (18.18)
c. Verification of anchorage end blocks strength to support prestressing forces.		X	SBC 304 (18.13.2)
6. Curing techniques including curing by high pressure steam to accelerate strength gain and reduce time of curing.		X	SBC 304 (5.11 & 5.13)
7. Finishing		X	ACI 301 Section 5.3.3 & 5.3.4
8. Precast with post-tension			
a. Verification of the concrete strength before the post tension takes place.		X	
b. Verification of ducts and end anchorage materials.		X	ACI 301, Section 9.2
c. Verification of the profile of ducts in the member as per design drawings.		X	SBC 304 (18.15)
d. Verification of prestressing force and measurement of tendon elongation.		X	SBC 304 (18.18)
e. Grouting of ducts.		X	SBC 304 (18.16)
f. Verification of anchorage and couplers.		X	SBC 304 (18.19)
9. Transportation			
a. Verification of precast elements transported position consistent with their shape and design to avoid excessive handling stress.		X	SBC 304 (16.6)
b. Verification of concrete strength before transportation. Member should have attained 85% of the concrete compressive strength.		X	SBC 304 (16.6)
10. Storage and handling at the job site.		X	SBC 304 (16.9)
11. Lifting and erection of the elements.		X	
12. Connection type and technique. Refer to Table 2.3	X		SBC 306, AWS (D1.1), ASTM A 325, ASTM A490,H-strength, ASTM A 307 Regular

SECTION 2.6
MASONRY CONSTRUCTION

2.6.0 The special inspection and verifications for masonry construction shall be as required by this section and table 2.6.

Exception: Special inspections shall not be required for minor construction such as fence walls.

TABLE 2.6
REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION

VERIFICATION AND INSPECTION	Continuous during task listed	Periodic during task listed	REFERENCE STANDARD	
			SBC 305	ACI 530.1/ASCE 6/TMS 602
1-As masonry construction begins, the following shall be verified to ensure compliance with the approved submittals				
a. Certificate for materials used in masonry construction including but not limited Masonry units, Mortar and grout material, Anchors, ties, fasteners, and metal accessories, Reinforcement.	–	X	3.	Art. 1.5
b. Proportions of site-prepared mortar.	–	X	–	Art. 2.6A
2-The inspection program shall verify compliance with construction documents and code provisions.				
a. Size, location, and alignment of structural elements and construction.	–	X	–	Art. 3.3G
b. Placement of masonry units and mortar.	–	X	4.1.2	Art 3.3
c. Placement of reinforcement, tie and anchor.	–	X	4.8	Art. 3.4
d. Protection of masonry during cold weather or hot weather.	–	X	4.3, 4.4	Art.1.8C, 1.8D
3-Prior to grouting, the following shall be verified to ensure compliance with code.				
a. Grout space is clean.	–	X	–	Art. 3.2D
b. Placement of reinforcement and connectors.	–	X	–	Art. 3.4
c. Proportions of site-prepared grout.	–	X	–	Art. 2.6B
4-Grout placement shall be verified to ensure compliance with code.		X	–	Art 3.5
5-Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed.		X	5.2.2, 5.3	Art. 1.4

SECTION 2.7
SOIL AND FOUNDATIONS

2.7.0 The special inspections for existing site soil conditions, fill placement and load-bearing requirements shall be as required by this section and Table 2.7. The approved soils report, required by Section 2.2 SBC 303, shall be used to determine compliance.

TABLE 2.7
REQUIRED VERIFICATION AND INSPECTION OF SOIL AND FOUNDATIONS

VERIFICATION AND INSPECTION	Continuous during task listed	Periodic during task listed	REFERENCE STANDARD SBC 303	REFERENCE STANDARD ASTM
1. Geotechnical Investigation			Chapter 2	D-420
a. Particle Size Analysis of Soils		X		D-422, D-1140
b. Liquid Limit, Plastic Limit		X		D4318
c. Standard Penetration Test		X		D-1586
d. Soil Classification		X		D-2487
2. Earthwork			Chapter 3	
a. Excavation		X		
b. Backfill - Compaction				
b.1 Moisture Density Relations	X			D-698, D-1557
b.2 Density by Nuclear Methods	X			D-2922
c. Drainage		X	Chapter 13	
3. Dewatering		X	Chapter 13	
4. Bearing Capacity			Chapter 4	
a. Plate Load Test		X		D-1194
b. Field Vane Test in Cohesive Soils	X			D-2573
c. Direct Shear Strength	X			D-3080
5. Footings and Foundations		X	Chapters 5, 6, 8, 9, 10, 11, 12	
6. Retaining Walls		X	Chapter 7	
7. Damp proofing & Waterproofing		X	Chapter 13	
8. Pier & Pile Foundations			Chapter 14	
a. Friction Cone Penetration Test	X			D-3441
b. (Testing) Pile under Axial Load	X			D-1143
9. Expansive & Collapsible Soils		X	Chapter 2	
10. Sabkha Soils		X	SBC 302 (2.7.5)	
11. Random & Artificial Backfill		X	SBC 302 (2.7.4)	

2.7.1 **Site preparation**. Prior to placement of the prepared fill, the special inspector shall determine that the site has been prepared in accordance with the approved soils report.

2.7.2 **During fill placement**. During placement and compaction of the fill material, the special inspector shall determine that the material being used and the maximum lift thickness comply with the approved report, as specified in Section 3.10 SBC 303.

2.7.3 **Evaluation of in-place density.** The special inspector shall determine, at the approved frequency, that the in-place dry density of the compacted fill complies with the approved report.

2.7.4 **Special conditions.** Random backfill or artificial backfill when encountered in reclaimed land (along shorelines, wadi slopes, etc.) shall be treated with utmost care, to make sure that soil conditions are satisfactory, otherwise soil improvement techniques should be used to avoid future structural damage.

2.7.5 **Sabkha soils.** Building foundations built over Sabkha soils, encountered in many locations in the Kingdom of Saudi Arabia, should be protected. Special precaution shall be taken regarding the control of humidity in the ground. Replacement of Sabkha soils shall be considered where possible.

2.7.6 **Pile foundations.** A special inspector shall be present when pile foundations are being installed and during tests. The special inspector shall make and submit to the building official records of installation of each pile and results of load tests.

Records shall include the cutoff and tip elevation of each pile relative to a permanent reference.

2.7.7 **Pier foundations.** Special inspection is required for pier foundations for buildings assigned to Seismic Design Category C or D.

SECTION 2.8
SPRAYED FIRE-RESISTANT MATERIALS

2.8.0 Special inspections for sprayed fire-resistant materials applied to structural elements and decks shall be in accordance with Section 2.8.1 through 2.8.5. Special inspections shall be based on the fire-resistance design as designated in the approved construction documents.

2.8.1 **Structural member surface conditions.** The surfaces shall be prepared in accordance with the approved fire-resistance design and the approved manufacturer's written instructions. The prepared surface of structural members to be sprayed shall be inspected before the application of the sprayed fire-resistant material.

2.8.2 **Application.** The substrate shall have a minimum ambient temperature before and after application as specified in the approved manufacturer's written instructions. The area for application shall be ventilated during and after application as required by the approved manufacturer's written instructions.

2.8.3 **Thickness.** The average thickness of the sprayed fire-resistant materials applied to structural elements shall not be less than the thickness required by the approved fire-resistant design. Thickness shall be determined in accordance with ASTM E 605. Samples of the sprayed fire-resistant materials shall be selected in accordance with Section 2.8.3.1 and 2.8.3.2.

2.8.3.1 **Floor, roof and wall assemblies.** The thickness of the sprayed fire-resistant material applied to floor, roof and wall assemblies shall be determined in accordance with ASTM E 605, taking the average of not less than four measurements for each 100 m^2 of the sprayed area on each floor or part thereof.

2.8.3.2 **Structural framing members.** The thickness of the sprayed fire-resistant material applied to structural members shall be determined in accordance with ASTM E 605. Thickness testing shall be performed on not less than 25 percent of the structural members on each floor.

2.8.4 **Density.** The density of the sprayed fire-resistant material shall not be less than the density specified in the approved fire-resistant design. Density of the sprayed fire-resistant material shall be determined in accordance with ASTM E 605.

2.8.5 **Bond strength.** The cohesive/adhesive bond strength of the cured sprayed fire-resistant material applied to structural elements shall not be less than 7 kN/m^2. The cohesive/adhesive bond strength shall be determined in accordance with the field test specified in ASTM E 736 by testing in-place samples of the sprayed fire-resistant material selected in accordance with 2.8.5.1 and 2.8.5.2.

2.8.5.1 **Floor, roof and wall assemblies.** The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from each floor, roof and wall assembly at the rate of not less than one sample for every 1000 m^2 or part thereof of the sprayed area in each story.

2.8.5.2 **Structural framing members.** The test samples for determining the cohesive/ adhesive bond strength of the sprayed fire-resistant materials shall be selected from beams, girders, joists, trusses and columns at the rate of not less than one sample for each type of structural framing member for each 1000 m^2 of floor area or part thereof in each story.

CHAPTER 3
SEISMIC AND WIND RESISTANT CONSTRUCTIONS

SECTION 3.1
SCOPE

The provisions of this chapter apply to Quality assurance, Special inspections and Testing for Seismic Resistance and Special Inspections for Wind Requirements. Provisions in Chapter 15 SBC 301 shall apply when not in conflict with the seismic provisions of this chapter.

SECTION 3.2
QUALITY ASSURANCE FOR SEISMIC RESISTANCE

3.2.1 **Scope.** A quality assurance plan for seismic requirements shall be provided in accordance with Section 3.2.2 for the following:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C or D.

2. Designated seismic systems in structures assigned to Seismic Design Category D.

3. The following additional systems in structures assigned to Seismic Design Category C:

   a. Heating, ventilating and air-conditioning (HVAC) ductwork containing hazardous materials and anchorage of such ductwork.

   b. Piping systems and mechanical units containing flammable, combustible or highly toxic materials.

   c. Anchorage of electrical equipment used for emergency or standby power systems.

4. The following additional systems in structures assigned to Seismic Design Category D:

   a. Systems required for Seismic Design Category C.

   b. Exterior wall panels and their anchorage.

   c. Suspended ceiling systems and their anchorage.

   d. Access floors and their anchorage.

   e. Steel storage racks and their anchorage, where the factor, *Ip*, determined in Section 12.15, SBC 301, is equal to 1.5.

Exceptions:

A quality assurance plan is not required for structures designed and constructed in accordance with the following:

1. The structure is constructed of light framed cold-formed steel; the design spectral response acceleration at short periods, S_{DS}, as determined in Section 9.4 SBC 301, does not exceed 0.5 g, and the height of the structure does not exceed 10000 mm above grade plane; or

2. The structure is constructed using a reinforced masonry structural system or reinforced concrete structural system; the design spectral response acceleration at short periods, S_{DS}, as determined in section 9.4 SBC 301, does not exceed 0.5 g, and the height of the structure does not exceed 8000 mm above grade plane; or

3. The structure is a detached one- or two-family dwelling not exceeding two stories in height; and

   a. The structure is classified as Occupancy Category I, as determined in Section 9.5, SBC 301; and

   b. The structure does not have any of the following plan or vertical irregularities as defined in Section 10.3.2, SBC 301:

      i. Torsional irregularity.

      ii. Nonparallel systems.

      iii. Stiffness irregularity–extreme soft story and soft story.

      iv. Discontinuity in capacity–weak story.

3.2.2 **Quality assurance plan preparation.** The design of each designated seismic system shall include a quality assurance plan prepared by a registered professional structural engineer. The quality assurance plan shall identify the following:

1. The designated seismic systems and seismic-force-resisting systems that are subject to quality assurance in accordance with Section 3.2.1.
2. The special inspections and testing to be provided as required by Chapter 2 and section 3.4 and other applicable sections of SBC, including the applicable standards referenced by SBC.
3. The type and frequency of testing required.
4. The type and frequency of special inspections required.
5. The required frequency and distribution of testing and special inspection reports.

3.2.3 **Contractor responsibility.** Each contractor responsible for the construction of a seismic-force-resisting system, designated seismic system, or component listed in the quality assurance plan shall submit a written contractor's statement of responsibility to the building official and to the owner prior to the commencement of work on the system or component. The contractor's statement of responsibility shall contain the following:

1. Acknowledgment of awareness of the special requirements contained in the quality assurance plan.
2. Acknowledgment that control will be exercised to obtain conformance with the construction documents approved by the building official.
3. Procedures for exercising control within the contractor's organization, the method and frequency of reporting and the distribution of the reports.
4. Identification and qualifications of the person(s) exercising such control and their position(s) in the organization.

SECTION 3.3
SPECIAL INSPECTIONS FOR SEISMIC RESISTANCE

3.3.1 **General.** Special inspection as specified in this section is required for the following, where required in Section 2.1. Special inspections itemized in Section 3.3.2 through 3.3.6 are required for the following:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C or D, as determined in Section 9.6, SBC 301.
2. Designated seismic systems in structures assigned to Seismic Design Category D.
3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C or D that are required in Section 3.3.5 and 3.3.6.

3.3.2 **Structural steel.** Continuous special inspection for structural welding in accordance with AISC 341.

Exceptions:

1. Single-pass fillet welds not exceeding 8 mm in size.
2. Floor and roof deck welding.

3.3.3 **Cold-formed steel framing.** Periodic special inspections during welding operations of elements of the seismic-force-resisting system. Periodic special inspections for screw attachment, bolting, anchoring and other fastening of components within the seismic-force-resisting system, including struts, braces, and hold-downs.

3.3.4 **Storage racks and access floors.** Periodic special inspection during the anchorage of access floors and storage racks 2500 mm or greater in height in structures assigned to Seismic Design Category D.

3.3.5 **Architectural components.** Periodic special inspection during the erection and fastening of exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer in structures assigned to Seismic Design Category D.

Exceptions:

1. Special inspection is not required for architectural components in structures 9000 mm or less in height.
2. Special inspection is not required for cladding and veneer weighing 250 N/m^2 or less.
3. Special inspection is not required for interior nonbearing walls weighing 750 N/m^2 or less.

3.3.6 **Mechanical and electrical components.** Periodic special inspection is required during the anchorage of electrical equipment for emergency or standby power systems in structures assigned to Seismic Design Category C or D. Periodic special inspection is required during installation of piping systems intended to carry flammable, combustible or highly toxic contents and their associated mechanical units in structures assigned to Seismic Design Category C or D. Periodic special inspection is required during the installation of HVAC ductwork that will contain hazardous materials in structures assigned to Seismic Design Category C or D.

3.3.6.1 **Component inspection.** Special inspection required for the installation of the following components, where the component has a Component Importance Factor of 1.0 or 1.5 in accordance with Section 12.1.5 SBC 301, shall maintain an approved quality control program. Evidence of the quality control program shall be permanently identified on each piece of equipment by a label.

1. Equipment using combustible energy sources.
2. Electrical motors, transformers, switchgear unit substations and motor control centers.
3. Reciprocating and rotating-type machinery.
4. Piping distribution systems 75 mm and larger.
5. Tanks, heat exchangers and pressure vessels.

3.3.6.2 **Component and attachment testing.** The component manufacturer shall test or analyze the component and the component mounting system or anchorage for the design forces in SBC 301 for those components having a Component Importance Factor of 1.0 or 1.5 in accordance with SBC 301. The manufacturer shall submit a certificate of compliance for review and acceptance by the registered design professional responsible for the design, and for approval by the building official. The basis of certification shall be by test on a shaking table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces from SBC 301 or by more rigorous analysis. The special inspector shall inspect the component and verify that the label, anchorage or mounting conforms to the certificate of compliance.

SECTION 3.4
STRUCTURAL TESTING FOR SEISMIC RESISTANCE

3.4.1 **Masonry.** Testing and verification of masonry materials and assemblies prior to construction shall comply with the requirements of Section 3.4.1, and Table 3.4.

TABLE 3.4
MINIMUM TESTS AND SUBMITTALS

Certificates of compliance used in masonry construction.
Verification of f'_m prior to construction and every 500 square meters during construction.
Verification of proportions of materials in mortar and grout as delivered to the site.

3.4.2 **Testing for seismic resistance.** The tests specified in Section 3.4.3 through 3.4.5 are required for the following:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C or D, as determined in SBC 301.
2. Designated seismic systems in structures assigned to Seismic Design

Category D.

3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C or D that are required in Section 3.4.5.

3.4.3 **Reinforcing and prestressing steel.** Certified mill test reports shall be provided for each shipment of reinforcing and prestressing steel used to resist flexural, shear and axial forces in reinforced concrete intermediate frames, special moment frames and boundary elements of special reinforced concrete or reinforced masonry shear walls.

3.4.4 **Structural steel.** The testing shall be as required by AISC 341 and the additional requirements herein. The acceptance criteria for nondestructive testing shall be as required in AWS D1.1 or specified by the registered design professional.

3.4.5 **Mechanical and electrical equipment.** Each manufacturer of designated seismic system components shall test or analyze the component and its mounting system or anchorage and submit a certificate of compliance for review and acceptance by the registered design professional in responsible charge of the design of the designated seismic system and for approval by the building official. The evidence of compliance shall be by actual test on a shake table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance) or by more rigorous analysis providing for equivalent safety. The special inspector shall examine the designated seismic system and determine whether the anchorages and label conform with the evidence of compliance.

SECTION 3.5
SPECIAL INSPECTIONS FOR WIND REQUIREMENTS

3.5.0 Inspection of Structural Connections in wind exposure Categories C and D where the 3-second gust basic wind speed is 175 km/h or greater, shall be in accordance to Table 3.5.

Exception: Fabrication of manufactured components and assemblies that have a label indicating compliance with the wind-load and impact-resistance requirements of this code.

TABLE 3.5
REQUIRED VERIFICATION AND INSPECTION OF STRUCTURAL CONNECTIONS

VERIFICATION AND INSPECTION		Continuous during task listed	Periodic during task listed	References Standard
1.	Roof connections and roof framing connections		X	SBC 306
2.	Wall connections to roof and floor diaphragms and framing.		X	SBC 306
3.	Roof and floor diaphragm systems, including collectors, drag struts and boundary elements.		X	SBC 306
4.	Vertical windforce-resisting systems, including braced frames, moment frames and shear walls.		X	SBC 306
5.	Windforce-resisting system connections to the foundation.		X	SBC 306
6.	Fabrication and installation of components and assemblies required to meet the impact-resistance requirements.		X	SBC 306
7.	Precast concrete wall elements during the erection.		X	SBC 306

CHAPTER 4
SPECIAL CONDITIONS

SECTION 4.1
ALTERNATIVE MATERIALS AND TEST PROCEDURES

4.1.1 **Conformance to standards.** The design strengths and permissible stresses of any structural material that are identified by a manufacturer's designation as to manufacture and grade by mill tests, or the strength and stress grade is otherwise confirmed to the satisfaction of the building official, shall conform to the specifications and methods of design of accepted engineering practice or the approved rules in the absence of applicable standards.

4.1.2 **New materials.** For new materials that are not specifically provided for in this code, the design strengths and permissible stresses shall be established by tests as provided for in Section 4.1.3.

4.1.3 **Test procedure.** In the absence of approved rules or other approved standards, the building official shall make, or cause to be made, the necessary tests and investigations; or the building official shall accept duly authenticated reports from approved agencies in respect to the quality and manner of use of new materials or assemblies as provided for in Section 1.4. The cost of all tests and other investigations required under the provisions of this code shall be borne by the permit applicant.

SECTION 4.2
TEST SAFE LOAD

4.2.0 Where proposed construction is not capable of being designed by approved engineering analysis, or where proposed construction design method does not comply with the applicable material design standard, the system of construction or the structural unit and the connections shall be subjected to the tests prescribed in Section 4.4. The building official shall accept certified reports of such tests conducted by an approved testing agency, provided that such tests meet the requirements of SBC and approved procedures.

SECTION 4.3
IN-SITU LOAD TESTS

4.3.1 **General.** Whenever there is a reasonable doubt as to the stability or load-bearing capacity of a completed building, structure or portion thereof for the expected loads, an engineering assessment shall be required. The engineering assessment shall involve either a structural analysis or an in-situ load test, or both. The structural analysis shall be based on actual material properties and other as-built conditions that affect stability or load-bearing capacity, and shall be conducted in accordance with the applicable design standard. If the structural assessment determines that the load-bearing capacity is less than that required by the code, load tests shall be conducted in accordance with Section 4.3.2. If the building, structure or portion thereof is found to have inadequate stability or load-bearing capacity for the expected loads, modifications to ensure structural adequacy or the removal of the inadequate construction shall be required.

4.3.2 **Test standards.** Structural components and assemblies shall be tested in accordance with the appropriate material standards listed in SBC. In the absence of a standard that contains an applicable load test procedure, the test procedure shall be developed by a registered design professional and approved. The test procedure shall simulate loads and conditions of application that the completed structure or portion thereof will be subjected to in normal use.

4.3.3 **In-situ load tests.** In-situ load tests shall be conducted in accordance with Section 4.3.3.1 or 4.3.3.2 and shall be supervised by a registered design professional. The test shall simulate the applicable loading conditions specified in SBC 301 as necessary to address the concerns regarding structural stability of the building, structure or portion thereof.

4.3.3.1 **Load test procedure specified.** Where a specialized code or standard listed in SBC contains an applicable load test procedure and acceptance criteria, the test procedure and acceptance criteria in the standard shall apply. In the absence of specific load factors or acceptance criteria, the load factors and acceptance criteria in Section 4.3.3.2 shall apply.

4.3.3.2 **Load test procedure not specified.** In the absence of applicable load test procedures contained within a standard referenced by this code or acceptance criteria for a specific material or method of construction, such existing structure shall be subjected to a test procedure developed by a registered design professional that simulates applicable loading and deformation conditions. For components that are not a part of the seismic-load-resisting system, the test load shall be equal to two times the unfactored design loads. The test load shall be left in place for a period of 24 hours. The structure shall be considered to have successfully met the test requirements where the following criteria are satisfied:

1. Under the design load, the deflection shall not exceed the limitations specified in SBC 301.
2. Within 24 hours after removal of the test load, the structure shall have recovered not less than 75 % of the maximum deflection.
3. During and immediately after the test, the structure shall not show evidence of failure.

SECTION 4.4
PRECONSTRUCTION LOAD TESTS

4.4.1 **General.** In evaluating the physical properties of materials and methods of construction that are not capable of being designed by approved engineering analysis or do not comply with applicable material design standards listed in SBC, the structural adequacy shall be predetermined based on the load test criteria established in Section 4.4.

4.4.2 **Load test procedures specified.** Where specific load test procedures, load factors and acceptance criteria are included in the applicable design standards listed in SBC, such test procedures, load factors and acceptance criteria shall apply. In the absence of specific test procedures, load factors or acceptance criteria, the corresponding provisions in Section 4.4.3 shall apply.

4.4.3 **Load test procedures not specified.** Where load test procedures are not specified in the applicable design standards listed in SBC, the load-bearing and deformation capacity of structural components and assemblies shall be determined on the basis of a test procedure developed by a registered design professional that simulates applicable loading and deformation conditions. For components and assemblies that are not a part of the seismic-load-resisting system, the test shall be as specified in Section 4.4.3.1. Load tests shall simulate the applicable loading conditions specified in SBC 301.

4.4.3.1 **Test procedure.** The test assembly shall be subjected to an increasing superimposed load equal to not less than two times the superimposed design load. The test load shall be left in place for a period of 24 hours. The tested assembly shall be considered to have successfully met the test requirements if the assembly recovers not less than 75 % of the maximum deflection within 24 hours after the removal of the test load. The test assembly shall then be reloaded and subjected to an increasing superimposed load until either structural failure occurs or the superimposed load is equal to two and one-half times the load at which the deflection limitations specified in Section 4.4.3.2 were reached, or the load is equal to two and one-half times the superimposed design load. In the case of structural components and assemblies for which deflection limitations are not specified in Section 4.4.3.2, the test specimen shall be subjected to an increasing superimposed load until structural failure occurs or the load is equal to two and one-half times the desired superimposed design load. The allowable superimposed design load shall be taken as the lesser of:

1. The load at the deflection limitation given in Section 4.4.3.2.
2. The failure load divided by 2.5.
3. The maximum load applied divided by 2.5.

4.4.3.2 **Deflection.** The deflection of structural members under the design load shall not exceed the limitations in SBC 301.

4.4.4 **Wall and partition assemblies.** Load-bearing wall and partition assemblies shall sustain the test load both with and without window framing. The test load shall include all design load components. Wall and partition assemblies shall be tested both with and without door and window framing.

4.4.5 **Test specimens.** Test specimens and construction shall be representative of the materials, workmanship and details normally used in practice. The properties of the materials used to construct the test assembly shall be determined on the basis of tests on samples taken from the load assembly or on representative samples of the materials used to construct the load test assembly. Required tests shall be conducted or witnessed by an approved agency.

REFERENCED STANDARDS

These are the standards referenced within SBC 302. The standards are listed herein by the promulgating agency of the standard, the standard identification, the effective date and title. The application of the referenced standards shall be as specified in SBC.

1. American Concrete Institute (ACI), "Specifications for Masonry Structures", ACI 530.1-02, Farmington Hills, MI 483333-9094.

2. American Concrete Institute (ACI), "Specifications for Structural Concrete", ACI 301, Farmington Hills, MI 483333-9094.

3. American Institute of Steel Construction, Inc. (AISC), "Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design, including Supplement No. 1", 2001, AISC 335-89S1 Chicago, IL 60601-2001.

4. American Institute of Steel Construction, Inc. (AISC), "Load and Resistance Factor Design Specification for Structural Steel Buildings" LRFD 1999, Chicago, IL 60601-2001.

5. American Institute of Steel Construction, Inc. (AISC), "Seismic Provisions for Structural Steel Buildings", AISC 341-02 Chicago, IL 60601-2001.

6. American Society of Civil Engineers (ASCE/SEI), Structural Engineering Institute, "Specification for Masonry Structures", ASCE 6-02, Reston, VA 20191-4400.

7. American Society for Testing and Materials (ASTM A 6/A 6M-01b Specification for General Requirements for Rolled Steel, Structural Steel Bars, Plates, Shapes, and Sheet Piling), West Conshohocken, PA 19428-2959.

8. American Society for Testing and Materials (ASTM A 307-00 Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength).

9. American Society for Testing and Materials (ASTM A 568/A 568M—01 Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements).

10. American Society for Testing and Materials (ASTM C 31/31M—98 Practice for Making and Curing Concrete Test Specimens in the Field).

11. American Society for Testing and Materials (ASTM C 172—99 Practice for Sampling Freshly Mixed Concrete).

12. American Society for Testing and Materials (ASTM D 422—98 Test Method for Particle-size Analysis of Soils).

13. American Society for Testing and Materials (ASTM D1143—81 (1994) E01 Test Method for Piles Under Static Axial Compressive Load).

14. American Society for Testing and Materials (ASTM D1557-00 Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (2,700kN m/m^3).

15. American Society for Testing and Materials (ASTM D1586-99 Specification for Penetration Test and Split-barrel Sampling of Soils).

16. American Society for Testing and Materials (ASTM D2487-00 Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

17. American Society for Testing and Materials (ASTM E 605-00 Test Method for Thickness and Density of Sprayed Fire-resistive Material (SFRM) Applied to Structural Members)

18. American Society for Testing and Materials (ASTM E 736—00 Test Method for Cohesion/Adhesion of Sprayed Fire-resistive Materials Applied to Structural Members)

19. American Society for Testing and Materials (ASTM A325M-93 Standard Specification for High-Strength Bolts for Structural Steel Joints)

20. American Society for Testing and Materials (ASTM A490-97 Standard Specification for Heat-Treated Steel Structural Bolts for Structural Steel Joints).

21. American Society for Testing and Materials (ASTM D420 Standard Guide to Site Characterization for Engineering, Design, and Construction Purposes).

22. American Society for Testing and Materials (ASTM D1140 Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75 μm) Sieve).

23. American Society for Testing and Materials (ASTM D4318-05 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils)

24. American Society for Testing and Materials (ASTM D698 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (600kN-m/m^3)

25. American Society for Testing and Materials (ASTM D2922-01 Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth).

26. American Society for Testing and Materials (ASTM, (D1194-94 Standard Test Method for Bearing Capacity of Soil for Static Load and Spread Footings).

27. American Society for Testing and Materials (ASTM D2573-01Standard Test Method for Field Vane Shear Test in Cohesive Soil).

28. American Society for Testing and Materials (ASTM D3080-04 Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions).

29. American Society for Testing and Materials (ASTM D3441-05 Standard Test Method for Mechanical Cone Penetration Tests of Soil).

30. American Welding Society (AWS), "Structural Welding Code—Steel", D1.1—00, Miami, FL 33126.

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Gratitude

In appreciation and gratitude
to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence
& Aviation and Inspector General

For their continuous support and gracious consideration,
the Saudi Building Code National Committee (SBCNC)
is honored to present the first issue of
the Saudi Building Code (SBC).

Saudi Building Code Requirements

201	Architectural	
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PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11th June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Structural Requirements for Soil and Foundations (SBC 303) were developed based on ICC code in addition to American Concrete Institute (ACI) materials. ACI grants permission to the SBCNC to include ACI materials in the SBC, and ACI is not responsible for any modifications or changes that SBCNC has made to accommodate local conditions.

Throughout the development of the document, several key aspects were considered; among them are the current local practice of geotechnical engineering and the causes related to soil and foundations problems.

The development process of SBC 303 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made in the IBC and the most important ones were that some sections have been extended to become entire new chapters in the SBC 303, as for the case of retaining walls, design for expansive soil, and design for vibratory loads. Particularly, design for expansive soil has been thoroughly enhanced with the additions of foundation systems that are common in local construction practice, and by emphasizing pre- and post-construction detailing which are usually overlooked and lead to many sequential problems. Sabkha and collapsible soils were not covered in the IBC document, yet these two soil formations are abundantly found on vast areas throughout the Kingdom and historically have created problems for structures. Thus, besides provisions relevant to identification and testing of these soil formations, which have been added to Chapter 2 “Site Investigations”, an entire chapter has been devoted for each soil type, covering all aspects relevant to design and construction of foundations systems on such problematic soil formations.

Although the provisions presume the existence of certain standard conditions, more often than not, every project has a unique combination of variables, and for that reason, all attempts have been made to make these requirements flexible.

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CHAPTER 1
GENERAL

SECTION 1.1
SCOPE

1.1.0 The Saudi Building Code for Soils and foundations referred to as SBC 303, provides minimum requirements for footing and foundation systems in those areas not subject to scour or water pressure by wind and wave action. Buildings and foundations subject to such scour or water pressure loads shall be designed in accordance with SBC 301. This requirement shall govern in all matters pertaining to design, construction, and material properties wherever this requirement is in conflict with requirements contained in other standards referenced in this requirement.

SECTION 1.2
DESIGN

1.2.0 Allowable bearing pressures, allowable stresses and design formulas provided in this code shall be used with the allowable stress design load combinations specified in Section 2.4 SBC 301. The quality and design of materials used structurally in excavations, footings and foundations shall conform to the requirements specified in SBC 301, SBC 304, SBC 305, and SBC 306 of the Saudi Building Code. Excavations and fills shall also comply with SBC 201.

1.2.1 **Foundation design for seismic overturning.** Where the foundation is proportioned using the strength design load combinations of Section 2.3.2 SBC 301, the seismic overturning moment need not exceed 75 percent of the value computed from Section 10.9.6 SBC 301 for the equivalent lateral force method, or Sections 10.10 and 10.14 SBC 301 for the modal analysis method.

SECTION 1.3
DEFINITIONS

1.3.0 The following words and terms shall, for the purposes of this code, have the meanings shown herein.

Acceptance Level. Acceptance level is the vibration level (displacement, velocity, or acceleration) at which a machine can run indefinitely without inducing vibration related maintenance.

Active Zone. Active zone is the upper zone of the soil deposit which is affected by the seasonal moisture content variations.

Alarm Level. Alarm level is the vibration level at which a machine is considered to have developed a defect that will result in related downtime. This level is usually higher than the acceptance level to allow for conservatism and machinery variance and is recommended as 1.5 times the acceptance level but may be varied, depending on specific experience or operational requirements.

Allowable Foundation Pressure. Allowable foundation pressure is a vertical pressure exerted by a foundation on a supporting formation which can be safely tolerated without causing detrimental settlement or shear failure.

Allowable Lateral Pressure. Allowable lateral pressure is a lateral pressure exerted due to a foundation or earth pressure, which can be safely tolerated without causing neither shear failure nor detrimental lateral movement.

Augered Uncased Piles. Augered uncased piles are piles constructed by depositing concrete into an uncased augered hole, either during or after the withdrawal of the auger.

Backfill. Backfill is earth filling a trench or an excavation under or around a building.

Belled Piers. Belled piers are cast-in-place concrete piers constructed with a base that is larger than the diameter of the remainder of the pier. The belled base is designed to increase the load-bearing area of the pier in end bearing and to resist upward heave in expansive soils.

Building Official. Building official means the officer or other designated authority charged with the administration and enforcement of this code, or his duly authorized representatives.

Borehole. Borehole is a hole made by boring into the ground to study stratification, to obtain natural resources, or to release underground pressures.

Caisson Piles. Caisson piles are cast-in-place concrete piles extending into bedrock. The upper portion of a caisson pile consists of a cased pile that extends to the bedrock. The lower portion of the caisson pile consists of an uncased socket drilled into the bedrock.

Cantilever Reinforced Concrete Wall. A cantilever T-type reinforced concrete wall consists of a concrete stem and base slab which form an inverted T.

Cantilever or Strap Footing. Cantilever or strap footing is a setup of a concrete beam placed on two adjacent footings which supports concentrated loads exerted at or close to the edge of the beam. The strap footing is used to connect an eccentrically loaded column footing to an interior column such that the transmitted moment caused from eccentricity to the interior column footing so that a uniform soil pressure is computed beneath both footings.

Cavity. Cavity is an underground opening with widely varying sizes caused mainly by solution of rock materials by water.

Collapse Index. Collapse index is the percentage of vertical relative magnitude of soil collapse determined at 200 kPa as per ASTM D 5333.

Collapse Potential. Collapse potential is the percentage of vertical relative magnitude of soil collapse determined at any stress level as per ASTM D 5333.

Collapsible Soils. Collapsible soils are deposits that are characterized by sudden and large volume decrease at constant stress when inundated with water. These deposits are comprised primarily of silt or fine sand-sized particles with small amounts of clay, and may contain gravel. Collapsible soils have low density, but are relatively stiff and strong in their dry state.

Column. Column is a member with a ratio of height-to-least-lateral dimension exceeding three, used primarily to support axial compressive load.

Combined Footing. Combined footing is a structural unit or assembly of units supporting more than one column load.

Compaction. Compaction is increasing the dry density of soils by means such as impact or by rolling the surface layers.

Concrete-Filled Steel Pipe and Tube Piles. Concrete-filled steel pipe and tube piles are constructed by driving a steel pipe or tube section into the soil and filling the pipe or tube section with concrete. The steel pipe or tube section is left in place during and after the deposition of the concrete.

Contact Pressure. Contact pressure or soil pressure is the pressure acting at and perpendicular to the contact area between footing and soil, produced by the weight of the footing and all forces acting on it.

Continuous or Strip Footing. Continuous or strip footing is a combined footing of prismatic or truncated shape, supporting two or more columns in a row. Continuous or strip footings may be of fixed thickness or upper face can be stepped or inclined with inclination or steepness not exceeding 1 unit vertical in 2 units horizontal.

Distortion Resistance. Distortion resistance corresponds to moment resistance to bending of beams, columns, footings and joints between them.

Driven Uncased Piles. Driven uncased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole that is later filled with concrete. The steel casing is lifted out of the hole during the deposition of the concrete.

Effective Depth of Section. Effective depth of section is the distance measured from the extreme compression fiber to centeroid of tension reinforcement.

Enlarged Based Piles. Enlarged base piles are cast-in-place concrete piles constructed with a base that is larger than the diameter of the remainder of the pile. The enlarged base is designed to increase the load-bearing area of the pile in end bearing.

Erosion. Erosion is the wearing away of the ground surface as a result of the movement of wind and water.

Excavation. Excavation is the mechanical or manual removal of earth material.

Expansion Index. Expansion index is the percent volume change determined in accordance with ASTM-D4829 multiplied by fraction passing No. 4 sieve of the soil multiplied by 100.

Expansion Joints. Expansion joints are intentional plane of weakness between parts of a concrete structure designed to prevent the crushing and distortion, including displacement, buckling, warping of abutting concrete structural units that might otherwise be developed by expansion, applied loads, or differential movements arising from the configuration of the structure or its settlement.

Expansive Soil. Expansive soil is a soil or rock material that has a potential for shrinking or swelling under changing moisture conditions. These soils are known to exist in many locations in the Kingdom such as Al-Ghatt, Tabuk, Tyma, Al-Madinah Al-Munuwarah, Al-Hafouf, and Sharora.

Factor of Safety. Factor of safety is the ratio of ultimate bearing capacity to the allowable load-bearing.

Fill. Fill is a deposit of earth material placed by artificial means.

Flexural Length. Flexural length is the length of the pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam.

Footing. Footing is that portion of the foundation of a structure which spreads and transmits loads directly to the soil.

Foundation. Foundation is the portion of a structure which transmits the building load to the ground.

Geotechnical Engineer. Geotechnical engineer is an engineer knowledgeable and experienced in soil and rock engineering.

Geotechnical Engineering. Geotechnical engineering is the application of the principles of soils and rock mechanics in the investigation, evaluation and design of civil works involving the use of earth materials and the inspection and/or testing of the construction thereof.

Grade. Grade is the vertical location of the ground surface.

Grade Beam. Grade beam is a continuous beam subject to flexure longitudinally, loaded by the line of columns it supports.

Gravity Concrete Wall. A gravity wall consists of mass concrete, generally without reinforcement. It is proportioned so that the resultant of the forces acting on any internal plane through the wall falls within, or close to, the kern of the section.

Grid Foundation. Grid foundation is a combined footing, formed by intersecting continuous footings, loaded at the intersection points and covering much of the total area within the outer limits of assembly.

Group R Occupancy. See **SBC 201**.

Group U Occupancy. See **SBC 201**.

Heavy Machinery. Heavy machinery is any machinery having rotating or reciprocating masses as the major moving parts (such as compressors, pumps, electric motors, diesel engines and turbines).

High-Tuned System. High-tuned system is a machine support/foundation system in which the operating frequency (range) of the machinery (train) is below all natural frequencies of the system.

Influence Zone. Influence zone is the zone under the foundation lying inside the vertical stress contours of value 0.1 of applied pressure.

Karst Formation. Karst formation is a type of topography that is formed on limestone, dolomite, marble, gypsum, anhydrite, halite or other soluble rocks. Its formation is the result of chemical solution of these rocks by percolating waters that commonly follow the pre-existing joint patterns and enlarge them to caverns. Sinkholes and solution cavities at or near the ground surface are characteristic features of karst, and pose a hazard in the Eastern and Central regions of Saudi Arabia. Collapse features are widespread in these regions and are commonly associated with carbonate and evaporite formations that have been subjected to karst development during Quaternary pluvial epochs.

Lateral Sliding Resistance. Lateral sliding resistance is the resistance of structural walls or foundations to lateral sliding, and it is controlled by interface friction and vertical loads.

Low-Tuned System. Low-tuned system is a machine support/foundation system in which the operating frequency (range) of the machinery (train) is above all natural frequencies of the system.

Machine Support/Foundation System. Machine support/foundation system is a system consisting of the machinery (train) including base plate and the foundation, support structure plus all piers, equipment and process piping supported on the foundation or machinery. The supporting soil, piling or structure shall be considered part of the machine foundation system.

Mat Area. Mat area is the contact area between mat foundation and supporting soil.

Mat Foundation. Mat foundation is a continuous footing supporting an array of columns in several rows in each direction, having a slab like shape with or without depressions or openings, covering an area of at least 75 % of the total area within the outer limits of the assembly.

Mixed System. A mixed system is a machine support/foundation system having one or more of its natural frequencies below and the rest above the operating frequency (range) of the machinery (train).

Modulus of Elasticity. Modulus of elasticity is the ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material.

Modulus of Subgrade Reaction. Modulus of subgrade reaction is the ratio between the vertical pressure against the footing or mat and the deflection at a point of the surface of contact.

Mortar. Mortar is a mixture of cementitious material and aggregate to which sufficient water and approved additives, if any, have been added to achieve a workable, plastic consistency.

Natural Frequency. Natural frequency is the frequency with which an elastic system vibrates under the action of forces inherent in the system and in the absence of any externally applied force.

Net Pressure. Net pressure is the pressure that can be applied to the soil in addition to the overburden due to the lowest adjacent grade.

Overburden. Overburden is the weight of soil or backfill from base of foundation to ground surface.

Overturning. Overturning is the horizontal resultant of any combination of forces acting on the structure tending to rotate as a whole about a horizontal axis.

Pier Foundations. Pier foundations consist of isolated cast-in-place concrete structural elements extending into firm materials. Piers are relatively short in comparison to their width, with lengths less than or equal to 12 times the least horizontal dimension of the pier. Piers derive their load-carrying capacity through skin friction, through end bearing, or a combination of both.

Pile Foundations. Pile foundations consist of concrete or steel structural elements either driven into the ground or cast in place. Piles are relatively slender in comparison to their length, with lengths exceeding 12 times the least horizontal dimension. Piles derive their load-carrying capacity through skin friction, through end bearing, or a combination of both.

Pressed Edge. Pressed edge is the edge of footing or mat along which the greatest soil pressure occurs under the condition of overturning.

Rectangular Combined Footing. Rectangular combined footing is a combined footing used if the column which is eccentric with respect to a spread footing carries a smaller load than the interior columns.

Registered Design Professional. Registered design professional is an individual who is registered or licensed to practice the respective design profession as defined by the statutory requirements of the professional registration laws of the state or jurisdiction in which the project is to be constructed.

Reinforced Concrete. Reinforced concrete is structural concrete reinforced with no less than the minimum amounts of nonprestressed reinforcement specified in SBC 304.

Reinforcement. Reinforcement is material that conforms to SBC 304 Section 3.5, excluding prestressing steel unless specifically included.

Retaining Walls. Retaining walls are structures that laterally support and provide stability for soils or other materials, where existing conditions do not provide stability with neither natural nor artificial slope.

Rocks. Rocks are natural aggregate of minerals or mineraloids that are connected together by strong bondings or attractive forces and have some degree of chemical and mineralogical constancy.

Rock Quality Designation. Rock quality designation, RQD, is an index or measure of the quality of a rock mass, and is calculated as summation of length of intact pieces of core greater than100 mm in length divided by the whole length of core advance.

Sabkhas. Sabkhas are salt bearing arid climate sediments covering vast areas of the coasts of Saudi Arabia. These soils either border partially land-locked seas or cover a number of continental depressions. The development of this material is due to low wave energy allowing the settlement of silt and clay particles to take place and then be loosely cemented by soluble material. Varying quantities of calcium carbonate, magnesium carbonate, calcium sulphate and calcium, magnesium, and sodium chlorides are found. The sabkha sediments are highly variable in lateral and vertical extent; various soil types, primarily composed of clays, silts, fine sands, and organic matter are inter-layered at random. In general, sabkha sediments are characterized by high void ratios and low dry densities. Accordingly, upon wetting sabkha soil is renowned for being highly compressible material with low bearing resistance, and hence considered among the poorest of foundation materials. Sabkha terrains are known to exist in many locations in the Kingdom such as Jubail, Rastanura, Abqaiq, Dammam, and Shaibah along the Arabian Gulf coast. They are prevailed in Jeddah, Jizan, Qunfudah, Al-Lith, Rabigh, and Yanbu along the Western coast as well as in Wadi As-Sirhan, around Qasim, and around Riyadh.

Settlement. Settlement is the gradual downward movement of an engineering structure, due to compression of the soil below the foundation.

Shallow Foundations. Shallow foundations are foundations with their depths less or equal to their widths.

Shoring. Shoring is the process of strengthening the side of excavation during construction stage.

Slope. Slope is the inclined surface of any part of the earth’s surface.

Soils. Soils are uncemented or weakly cemented accumulation of solid particles that have resulted from the disintegration of rocks.

Soil mechanics. Soil mechanics is the branch of geotechnical engineering that deals with the physical properties of soil and the behavior of soil masses subjected to various types of forces. It applies the basic principles of mechanics including kinematics, dynamics, fluid mechanics, and the mechanics of materials to soils.

Spiral reinforcement. Spiral reinforcement is **c**ontinuously wound reinforcement in the form of a cylindrical helix.

Spread Footing. Spread footing is a concrete pad supporting column load. It can take a rectangular, square or a circular shape and having a uniform or tapered thickness not less than 250 mm.

Spring Constant. Spring constant is the soil resistance in load per unit deflection obtained as the product of the contributory area and coefficient of vertical subgrade reaction.

Steady-State Dynamic Force. Steady-state dynamic force is any dynamic force which is periodic in nature and generated during normal operating conditions, such as centrifugal forces due to unbalances in rotating machinery or piston forces in reciprocating machinery.

Steel-Cased Piles. Steel-cased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole. The steel casing is left permanently in place and filled with concrete.

Support/Foundation. Support/foundation is the part of the machine support not supplied by the equipment manufacturer as part of the machinery (train). This may include but is not limited to piers, concrete mat or block, pilings, steel structures, anchor bolts and embedded foundation plates.

Surcharge. Surcharge is the load applied to ground surface above a foundation, retaining wall, or slope.

Swell Pressure. Swell pressure is the maximum applied stress required to maintain constant volume of an inundated sample in the oedometer.

Table Top. Table top is a reinforced concrete structure supporting elevated machinery.

Total Core Recovery. Total core recovery, TCR, is the total length of rock pieces recovered divided by the total length of core advance.

Transient Dynamic Force. Transient dynamic force is any dynamic force, which is short term in nature such as starting torques or short circuit moments in electrical machinery, hydraulic forces, resonance forces of low-tuned or mixed systems during start-up or shutdown.

Trapezoidal-Shaped Combined Footing. Trapezoidal-shaped combined footing is a combined footing used when the column which has too limited space for a spread footing carries the larger load.

Underpinning. Underpinning is the process of strengthening and stabilizing the foundation of an existing building or other structure. Underpinning may be necessary for a variety of reasons including, but not limited to, the original foundation is simply not be strong enough or stable enough, the use of the structure has changed, the properties of the soil supporting the foundation may have changed or was mischaracterized during planning, the construction of nearby structures necessitates the excavation of soil supporting existing foundation. Underpinning is accomplished by extending the foundation in depth or in breadth so it either rests on a stronger soil stratum or distributes its load across a greater area.

Wall Footing. Wall footing is strip footing supporting wall such that the centerlines of the footing and the wall coincide.

Water Table. Water table is the planar surface between the zone of saturation and the zone of aeration. Also known as free-water elevation; free water surface; groundwater level; groundwater surface, groundwater table; level of saturation; phreatic surface; plane of saturation; saturated surface; water level; and waterline.

Weep Holes. Weep holes are openings used in retaining walls to permit passage of water from the backfill to the front.

CHAPTER 2
SITE INVESTIGATIONS

SECTION 2.1
GENERAL

2.1.0 Site investigations shall be conducted in conformance with Sections 2.2 through 2.6. Where required by the building official, the classification and investigation of the soil shall be made by a registered design professional.

2.1.1 **Objectives**. Site investigation shall be planned and executed to determine the following:

1. Lateral distribution and thickness of the soil and rock strata within the zone of influence of the proposed construction.
2. Suitability of the site for the proposed work.
3. Proposal of best method for construction on the site.
4. Physical and engineering properties of the soil and rock formations.
5. Groundwater conditions with consideration of seasonal changes and the effects of extraction due to construction.
6. Hazardous conditions including unstable slopes, active or potentially active faults, regional seismicity, floodplains, ground subsidence, collapse, and heave potential.
7. Changes that may arise in the environment and the effects of these changes on the proposed and adjacent buildings.
8. Advice on the suitability of alternative location for the proposed building, if exists.
9. Thorough understanding of all subsurface conditions that may affect the proposed building.

SECTION 2.2
SCOPE

2.2.0 The owner or applicant shall submit a site investigation to the building official where required in Sections 2.2.1 through 2.2.6.

Exception:
The building official need not require a site investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in Sections 2.2.1 through 2.2.5.

No site investigation report is needed if the building meets the following combined criteria:

1. The net applied load on the foundation is less than 50 kPa.
2. There are no dynamic or vibratory loads on the building.
3. Questionable or problematic soil is not suspected underneath the building.
4. Cavities are not suspected underneath the footing of the building.

2.2.1 **Questionable soil.** Where the safe-sustaining power of the soil is in doubt, or where a load-bearing value superior to that specified in this code is claimed, the building official shall require that the necessary investigation be made. Such investigation shall comply with the provisions of Sections 2.4 through 2.6.

2.2.2 **Problematic soils.** In areas likely to have expansive, collapsible, or sabkha soils, the building official shall require site investigation to determine where such soils do exist.

2.2.3 **Ground-water table.** A subsurface soil investigation shall be performed to determine whether the existing ground-water table is within the influence zone underneath the footing of the building.

2.2.4 **Rock strata.** Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than 3 m below the level of the foundations to provide assurance of the soundness of the foundation bed and its load-bearing capacity.

2.2.4.1 **Rock cavities.** In areas of karst formations, the building official shall require site investigation to determine the potential sizes and locations of cavities underneath the building. If cavities are encountered, such investigation shall recommend remedies and construction procedures.

2.2.5 **Seismic Design Category C.** Where a structure is determined to be in Seismic Design Category C in accordance with Chapters 9 through 16 of SBC 301, an investigation shall be conducted, and shall include an evaluation of the following potential hazards resulting from earthquake motions: slope instability, liquefaction and surface rupture due to faulting or lateral spreading.

2.2.6 **Seismic Design Category D.** Where the structure is determined to be in Seismic Design Category D, in accordance with Chapters 9 through 16 of SBC 301, the soils investigation requirements for Seismic Design Category C, given in Section 2.2.5, shall be met, in addition to the following. The investigation shall include:

1. A determination of lateral pressures on basement and retaining walls due to earthquake motions.
2. An assessment of potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall address mitigation measures. Such measures shall be given consideration in the design of the structure and can include, but are not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements or any combination of these measures. The potential for liquefaction and soil strength loss shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions.

SECTION 2.3
SOIL CLASSIFICATION

2.3.0 Where required, soils shall be classified in accordance with Sections 2.3.1, 2.3.2, 2.3.3, or 2.3.4.

2.3.1 **General.** For the purposes of this section, the definition and classification of soil materials for use in Table 4.1 shall be in accordance with ASTM D 2487.

2.3.2 **Expansive soils.** Soils meeting all four of the following provisions shall be considered expansive. Compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Plasticity index of 15 or greater, determined in accordance with ASTM D 4318.
2. More than 10 percent of the soil particles pass a No. 200 sieve (75 micrometers), determined in accordance with ASTM D 422.
3. More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.
4. Expansion index greater than 20, determined in accordance with ASTM D 4829.

2.3.3 **Collapsible soils.** Soils meeting all four of the following provisions shall be considered collapsible. Compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Desiccated Alluvial (Wadi) soils
2. Dry field density less than 17 kN/m^3 determined in accordance with ASTM D1556
3. Clay content 10 to 30 percent, determined in accordance with ASTM D422
4. Collapse index greater than 1 percent, determined in accordance with ASTM D5333.

2.3.4 **Sabkha soils.** Soils meeting the following shall be suspected as sabkha soils:

1. Very soft, with SPT values in the range of 0 to 8, determined in accordance with ASTM D1586.
2. Precipitated salts of different sizes, shape, and composition within the sediments.
3. High soluble salt content.
4. Soil exhibits significant variations in its chemical composition.
5. Soil exhibits high degree of variability of its sediments in both vertical and lateral extent within a considerably short distance.
6. Upon wetting soil becomes impassible.

SECTION 2.4
INVESTIGATION

2.4.0 Soil investigation shall be based on observation and any necessary tests of the materials disclosed by borings, test pits or other subsurface exploration made in appropriate locations. Additional studies shall be made as necessary to evaluate slope stability, soil strength, position and adequacy of load-bearing soils, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction, expansiveness, and collapsibility.

2.4.1 **Exploratory boring.** The scope of the site investigation including the number and types of borings or soundings, the equipment used to drill and sample, the in-situ testing equipment and the laboratory testing program shall be determined by a registered design professional. In areas likely to have problematic soils, field explorations shall include:

1. Investigations of soils between the ground surface and the bottom of the foundation, as well as materials beneath the proposed depth of foundation.
2. Evaluations and interpretations of the environmental conditions that would contribute to moisture changes and their probable effects on the behavior of such soils.

2.4.2 **Number of boreholes.** The minimum number of boreholes in a given site shall be taken in accordance with Table 2.1 and its provisions. The values included in Table 2.1 shall be considered as minimum guideline.

2.4.3 **Depth of boreholes.** The depth of boreholes shall cover all strata likely to be affected by the loads from the building and adjacent buildings. The minimum depth of boreholes shall be taken from Table 2.1.

SECTION 2.5
SOIL BORING AND SAMPLING

2.5.0 The soil boring and sampling procedure and apparatus shall be in accordance with generally accepted engineering practice. The registered design professional shall have a fully qualified representative on the site during all boring and sampling operations.

2.5.1 **Soil boring and sampling of expansive soils**. In areas likely to have expansive soils the following shall be taken into considerations:

1. Air drilling shall be used to maintain the natural moisture contents of the samples more effectively.
2. The use of lubricant that might react with the soil and change its properties shall be avoided.

3. The depth of sampling shall be at least as deep as the probable depth to which moisture changes will occur (active zone) but shall not be less than 1.5 times the minimum width of slab foundations to a maximum of 30 meters and a minimum of three base diameters beneath the base of shaft foundations.
4. Undisturbed samples shall be obtained at intervals of not greater than 1500 mm of depth. Sampling interval may be increased with depth.
5. A coating of wax shall be brushed on the sample before wrapping.
6. The outer perimeter of the sample shall be trimmed during the preparation of specimens for laboratory tests, leaving the more undisturbed inner core.
7. The sample shall be taken as soon as possible, after advancing the hole to the proper depth and cleaning out the hole, and personnel shall be well trained to expedite proper sampling, sealing, and storage in sample containers.

2.5.2 **Soil boring and sampling of collapsible soils**. In areas likely to have collapsible soils the following shall be taken into considerations:

1. Air drilling shall be used to maintain the natural moisture contents of the samples.
2. The depth of sampling shall be at least as deep as the probable depth to which moisture changes will occur but shall not be less than 2 times the minimum width of footing to a maximum of 30 meters and a minimum of three base diameters beneath the base of shaft foundations.
3. Undisturbed samples shall be obtained at intervals of not greater than 1500 mm of depth.
4. In the event undisturbed samples cannot be obtained from a borehole, test pits shall be excavated to sufficient depth and dry density of the soil shall be measured at various horizons in the pit.
5. Where possible, hand carved undisturbed samples taken in a vertical direction shall be obtained for odometer testing. Alternately, plate load test in unsoaked and soaked conditions shall be performed to determine the most critical collapse potential below foundation level.

2.5.3 **Soil boring and sampling of sabkha soils**. In areas likely to have sabkha soils the following shall be taken into considerations:

1. A full chemical analyses on soil and ground water to determine the average and range of the aggressive compounds and the variation in content with depth.
2. Grading of sabkha shall be determined by using wet sieving with non-polar solvent (sabkha brine, methylene chloride)
3. Basic properties including moisture content and specific gravity shall be determined by using oven drying at 60°C in accordance with ASTM D854 and ASTM D2216.

SECTION 2.6
REPORTS

2.6.0 The soil classification and design load-bearing capacity shall be shown on the construction document. Where required by the building official, a written report of the investigation shall be submitted that includes, but need not be limited to, the following information:

1. Introduction with location map depicting adjacent buildings, existing roads, and utility lines.
2. Climatic conditions such as rain rate, storm water discharge, etc. if relevant effect is suspected on the soil or rock formations.
3. Description of site topography and relevant geological information.
4. A plot showing the location of test borings and/or excavation pits.
5. A complete record of the soil samples.
6. A complete record of the borehole log with the standard penetration test, SPT, values at the corresponding depths for soil samples and RQD and TCR values for rock samples.
7. A record of the soil profile.
8. Elevation of the water table, if encountered and recommended procedures for dewatering, if necessary.
9. Brief description of conducted laboratory and field tests (or its SASO or ASTM standards, or equivalent standard number) and a summary of the results.
10. Recommendations for foundation type and design criteria, including but not limited to: bearing capacity of natural or compacted soil; provisions to mitigate the effects of problematic soils (expansive, collapsible, sabkha, etc.); mitigation of the effects of liquefaction, differential settlement and varying soil strength; and the effects of adjacent loads. The recommendations for foundation design must be based on the facts stated in the report, i.e. on the borehole records and test data. They must not be based on conjecture.
11. Expected total and differential settlements.
12. Pile and pier foundation information in accordance with Section 14.2.
13. Combined footings and mats information in accordance with Section 8.1.
14. Special design and construction provisions for footings or foundations founded on problematic soils in accordance with Chapters 9, 10, and 11, as necessary.
15. Compacted fill material properties and testing in accordance with Section 3.10.
16. Recommended sites for waste material disposal.
17. Suitability of excavated material for reuse as fill material in site.

TABLE 2.1
MINIMUM NUMBER AND MINIMUM DEPTHS OF BOREHOLES FOR BUILDINGS[a,b,c,d,e]

NO. OF STORIES	BUILT AREA (m^2)	NO. OF BOREHOLES	MINIMUM DEPTH[f] OF TWO THIRDS OF THE BOREHOLES (m)	MINIMUM DEPTH[f] OF ONE THIRD OF THE BOREHOLES (m)
2 or less	< 600	3	4	6
	600 – 5000	3 – 10[g]	5	8
	> 5000	Special investigation		
3 - 4	< 600	3	6 - 8	9 - 12
	600 – 5000	3 – 10[g]		
	> 5000	Special investigation		
5 or higher	Special investigation			

a. If possible, standard penetration tests, SPT, shall be performed in all sites.

b. If questionable soils do exist underneath the building, a minimum of one borehole shall penetrate all layers containing this soil.

c. Seasonal changes in groundwater table and the degree of saturation shall be considered.

d. If sufficient data is available, a registered design professional may use number and depth of boreholes that are different from the tabular values.

e. For foundation of pole and towers, a minimum of one boring with sufficient depth shall be located in the center of the foundation.

f. Depth is measured from level of foundation bottom.

g. Number of boreholes shall be selected by a registered design professional based on variations in site conditions, and contractor shall advice if additional or special tests are required.

CHAPTER 3
EXCAVATION, GRADING AND FILL

SECTION 3.1
GENERAL

3.1.0 Proper safety precautions shall be considered at all stages of excavation. Special care, measures, and techniques shall be followed for excavation below groundwater table.

The investigation and report provisions of Chapter 2 shall be expanded to include, but need not be limited to, the following:

1. Property limits and accurate contours of existing ground and details of terrain and area drainage.

2. Limiting dimensions, elevations or finish contours to be achieved by the grading, and proposed drainage channels and related construction.

3. Detail plans of all surface and subsurface drainage devices, walls, cribbing, and other protective devices to be constructed with, or as a part of, the proposed work.

4. Location of any buildings or structures on the property where the work is to be performed and the location of any buildings or structures on adjacent land which are within 5 m of the property or which may be affected by the proposed grading operations.

5. Conclusions and recommendations regarding the effect of geologic conditions on the proposed construction, and the adequacy of sites to be developed by the proposed grading.

SECTION 3.2
COMMENCEMENT

3.2.0 Excavation, grading and fill shall not be commenced without first having obtained a permit from the building official.

Exception: Permit shall not be required for the following:

1. Grading in an isolated, self-contained area if there is no apparent danger to private or public property.

2. Exploratory excavations under the direction of geotechnical engineers.

3. An excavation which (a) is less than 600 mm in depth, or (b) which does not create a cut slope greater than 1500 mm in height and steeper than three units horizontal to two units vertical.

4. A fill less than 300 mm in depth and placed on natural terrain with a slope flatter than five units horizontal to one unit vertical, or less than 1 m in depth, not intended to support structures, does not exceed 40 cubic meters on any one lot and does not obstruct a drainage course.

SECTION 3.3
EXCAVATIONS NEAR FOOTINGS OR FOUNDATIONS

3.3.0 Excavations for buildings shall be carried out as not to endanger life or property. Excavations for any purposes shall not remove lateral support from any footing or foundation without first underpinning or protecting the footing or foundation against settlement or lateral translation. Proper underpinning, sequence of construction, and method of shoring shall be approved by a registered design professional and carried out immediately after start of excavation. Underpinning system shall be periodically checked for safety assurance.

SECTION 3.4
SLOPE LIMITS

3.4.0 Slopes for permanent fill shall not be steeper than one unit vertical in two units horizontal (50-percent slope). Cut slopes for permanent excavations shall not be steeper than one unit vertical in two units horizontal (50-percent slope). Deviation from the foregoing limitations for cut slopes shall be permitted only upon the presentation of a soil investigation report acceptable to the building official and shows that a steeper slope will be stable and not create a hazard to public or private property.

SECTION 3.5
SURCHARGE

3.5.0 No fill or other surcharge loads shall be placed adjacent to any building or structure unless such building or structure is capable of withstanding the additional loads caused by the fill or surcharge. Existing footings or foundations, which can be affected by any excavation shall be underpinned adequately or otherwise protected against settlement and shall be protected against later movement.

SECTION 3.6
PLACEMENT OF BACKFILL

3.6.0 The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders or a controlled low-strength material (CLSM). The ground surface shall be prepared to receive fill by removing vegetation, noncomplying fill, topsoil and other unsuitable materials. The backfill shall be placed in lifts and compacted, in a manner that does not damage the foundation or the waterproofing or damp proofing material. Special inspections of compacted fill shall be in accordance with Section 2.7 SBC 302.

Exception: Controlled low-strength material need not be compacted.

SECTION 3.7
SITE GRADING

3.7.0 The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5 percent slope) for a minimum distance of 3 m measured perpendicular to the face of the wall or an approved alternate method of diverting water away from the foundation shall be used.

Exception: Where climate or soil conditions warrant, the slope of the ground away from the building foundation is permitted to be reduced to not less than one unit vertical in 50 units horizontal (2 percent slope).

The procedure used to establish the final ground level adjacent to the foundation shall account for additional settlement of the backfill.

SECTION 3.8
GRADING DESIGNATION

3.8.0 The faces of cut and fill slopes shall be prepared and maintained to control against erosion. All grading in excess of 3500 cubic meters shall be performed in accordance with the approved grading plan prepared by a registered design professional, and shall be designated as “engineering grading”. Grading involving less than 3500 cubic meters shall be designated as “regular grading” unless required by the building official to be considered as “engineering grading”.

For engineering grading, grading plan shall be prepared and approved by a registered design professional. For regular grading, the building official may require inspection and testing by an approved agency. Where the building official has cause to believe that geologic factors may be involved, the grading operation shall conform to “engineering grading” requirements.

SECTION 3.9
GRADING AND FILL IN FLOODWAYS

3.9.0 In floodways shown on the flood hazard map established in SBC 301 Section 5.3, grading and/or fill shall not be approved unless it has been demonstrated through hydrologic and hydraulic analyses performed by a registered design professional in accordance with standard engineering practice that the proposed grading or fill, or both, will not result in any increase levels during the occurrence of the design flood.

SECTION 3.10
COMPACTED FILL MATERIAL

3.10.0 Where footings will bear on compacted fill material, the compacted fill shall comply with the provisions of an approved report, which shall contain, but need not be limited to, the following:

1. Specifications for the preparation of the site prior to placement of compacted fill material.
2. Specifications for material to be used as compacted fill.
3. Test method to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
4. Maximum allowable thickness of each lift of compacted fill material.
5. Field test method for determining the in-place dry density of the compacted fill.
6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.

7. Number and frequency of field tests required to determine compliance with Item 6.

Exception: Compacted fill material less than 300 mm in depth need not comply with an approved report, provided it has been compacted to a minimum of 95 percent Modified Proctor in accordance with ASTM D 1557. The compaction shall be verified by a qualified inspector approved by the building official.

3.10.1 **Oversized materials**. No rock or similar irreducible material with a maximum dimension greater than 300 mm shall be buried or placed in fills within 1.5 m, measured vertically, from the bottom of the footing or lowest finished floor elevation, whichever is lower, within the building pad. Oversized fill material shall be placed so as to assure the filling of all voids with well-graded soil. Specific placement and inspection criteria shall be stated and continuous special inspections shall be carried out during the placement of any oversized fill material.

SECTION 3.11
CONTROLLED LOW-STRENGTH MATERIAL (CLSM)

3.11.0 Where footings will bear on controlled low-strength material (CLSM), the CLSM shall comply with the provisions of an approved report, which shall contain, but need not be limited to, the following:

1. Specifications for the preparation of the site prior to placement of the CLSM.
2. Specifications for the CLSM.
3. Laboratory or field test method(s) to be used to determine the compressive strength or bearing capacity of the CLSM.
4. Test methods for determining the acceptance of the CLSM in the field.
5. Number and frequency of field tests required to determine compliance with Item 4.

CHAPTER 4
ALLOWABLE LOAD-BEARING VALUES OF SOILS

SECTION 4.1
DESIGN

4.1.0 The presumptive load-bearing values provided in Table 4.1 shall be used with the allowable stress design load combinations specified in Section 2.4 of SBC 301.

SECTION 4.2
PRESUMPTIVE LOAD-BEARING VALUES

4.2.0 The maximum allowable foundation pressure, lateral pressure or lateral sliding resistance values for supporting soils at or near the surface shall not exceed the values specified in Table 4.1 unless data to substantiate the use of a higher value are submitted and approved by the building official. In case of thin soft layers existing between layers of high bearing values, the foundation shall be designed according to the bearing capacity of the thin soft layers.

Presumptive load-bearing values shall apply to materials with similar physical characteristics and depositional conditions.

Mud, organic silt, organic clays, peat or unprepared fill shall not be assumed to have a presumptive load-bearing capacity unless data to substantiate the use of such a value are submitted.

Exception: A presumptive load-bearing capacity is permitted to be used where the building official deems the load-bearing capacity of mud, organic silt or unprepared fill is adequate for the support of lightweight and temporary structures.

TABLE 4.1
ALLOWABLE FOUNDATION AND LATERAL PRESSURE

CLASS OF MATERIALS	ALLOWABLE FOUNDATION PRESSURE (kPa)[a]	LATERAL BEARING (kPa/m below natural grade)[a]	LATERAL SLIDING	
			Coefficient of friction[b]	Resistance (kPa)[c]
1. Crystalline bedrock	600	200	0.70	–
2. Sedimentary and foliated rock	200	60	0.35	–
3. Sandy gravel and/or gravel (GW and GP)	150	30	0.35	–
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	100	25	0.25	–
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	75[d]	15	–	6

a. An increase of one-third is permitted when using the alternate load combinations in SBC 301 Section 2.4 that include wind or earthquake loads.

b. Coefficient to be multiplied by the dead load.

c. Lateral sliding resistance value to be multiplied by the contact area, as limited by Section 4.3.

d. Where the building official determines that in-place soils with an allowable bearing capacity of less than 70 kPa are likely to be present at the site, the allowable bearing capacity shall be determined by a site investigation in accordance with Chapter 2.

SECTION 4.3
LATERAL SLIDING RESISTANCE

4.3.0 The resistance of structural walls to lateral sliding shall be calculated by combining the values derived from the lateral bearing and the lateral sliding resistance shown in Table 4.1 unless data to substantiate the use of higher values are submitted for approval.

For clay, sandy clay, silty clay and clayey silt, in no case shall the lateral sliding resistance exceed one-half the dead load.

4.3.1 **Increases in allowable lateral sliding resistance.** The resistance values derived from Table 4.1 are permitted to be increased by the tabular value for each additional 300 mm of depth to a maximum of 15 times the tabular value.

Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a 13 mm motion at the ground surface due to short-term lateral loads are permitted to be designed using lateral-bearing values equal to two times the tabular values.

SECTION 4.4
COMPUTED LOAD-BEARING VALUES

4.4.0 It shall be permitted to obtain the ultimate bearing capacity from appropriate laboratory and/or field tests including, but need not be limited to, standard penetration test conforming to ASTM D1586 and plate load test conforming to ASTM D1194. Where the soil to a deep depth is homogeneous soil, the plate load test shall be conducted at the level of footing bottom. In case the soil medium is made of several layers, the test shall be conducted at each layer to a depth equal to not less than twice the width of footing measured from the bottom of footing. In case there is a large difference between the footing width and plate size, plates of different sizes shall be used to establish the relationship between width and load-bearing.

It shall be permitted to use formulae in the computations of ultimate bearing capacity that are of common use in geotechnical engineering practice or based on a sound engineering judgment and subject to approval to the building official.

4.4.1 **Effect of water table.** The submerged unit weight shall be used as appropriate to determine the actual influence of the groundwater on the bearing capacity of the soil. The foundation design shall consider the buoyant forces when groundwater is above or expected to rise above the foundation level.

CHAPTER 5
SPREAD FOOTINGS

SECTION 5.1
GENERAL

5.1.0 Spread footings shall be designed and constructed in accordance with Sections 5.1 through 5.6. Footings shall be built on undisturbed soil, compacted fill material or CLSM. Compacted fill material shall be placed in accordance with Section 3.10. CLSM shall be placed in accordance with Section 3.11.

The bottom surface of footings is permitted to have a slope not exceeding one unit vertical in 10 units horizontal (10 percent slope). Footings shall be stepped where it is necessary to change the elevation of the top surface of the footing or where the surface of the ground slopes more than one unit vertical in 10 units horizontal (10 percent slope).

SECTION 5.2
DEPTH OF FOOTINGS

5.2.0 The minimum depth of footing below the natural ground level shall not be less than 1.2 m for cohesionless soils, 1.5 m for silty and clay soils and 600 mm to 1200 mm for rocks depending on strength and integrity of the rock formations. Where applicable, the depth of footings shall also conform to Sections 5.2.1 through 5.2.3.

5.2.1 **Adjacent footings.** Footings on granular soil shall be so located that the line drawn between the lower edges of adjoining footings shall not have a slope steeper than 30 degrees (0.52 rad) with the horizontal, unless the material supporting the higher footing is braced or retained or otherwise laterally supported in an approved manner or a greater slope has been properly established by engineering analysis that is accepted by the building official.

5.2.2 **Shifting or moving soils.** Where it is known that the shallow subsoils are of a shifting or moving character, footings shall be carried to a sufficient depth to ensure stability.

5.2.3 **Stepped footings**. Footings for all buildings where the surface of the ground slopes more than one unit vertical in ten units horizontal (10 percent slope) shall be level or shall be stepped so that both top and bottom of such footing are level.

SECTION 5.3
FOOTINGS ON OR ADJACENT TO SLOPES

5.3.0 The placement of buildings and structures on or adjacent to slopes steeper than one unit vertical in three units horizontal shall conform to Sections 5.3.1 through 5.3.5.

5.3.1 **Building clearance from ascending slopes.** In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided for in Section 5.3.5 and Figure 5.1, the following criteria will be assumed to provide this protection. Where the existing slope is steeper than one unit vertical in one unit horizontal (100 percent slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees (0.79 rad) to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.

5.3.2 **Footing setback from descending slope surface.** Footings on or adjacent to slope surfaces shall be founded in firm material with an embedment and set back from the slope surface sufficient to provide vertical and lateral support for the footing without detrimental settlement. Except as provided for in Section 5.3.5 and Figure 5.1, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than one unit vertical in one unit horizontal (100 percent slope), the required setback shall be measured from imaginary plane 45 degrees (0.79 rad) to the horizontal, projected upward from the toe of the slope.

5.3.3 **Pools.** The setback between pools regulated by this code and slopes shall be equal to one-half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 2100 mm from the top of the slope shall be capable of supporting the water in the pool without soil support.

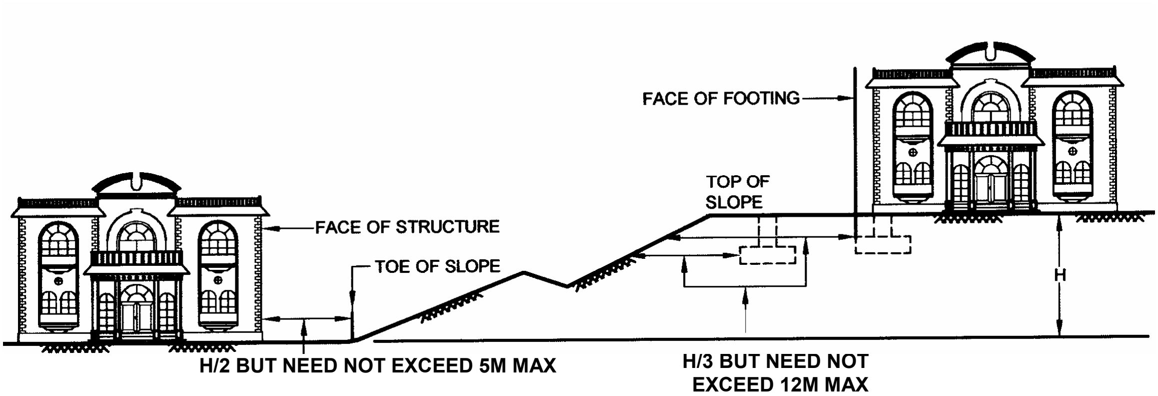


FIGURE 5.1
FOUNDATION CLEARANCES FROM SLOPES

5.3.4 **Foundation elevation.** On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 300 mm plus 2 percent. Alternate elevations are permitted subject to the approval of the building official, provided it can be demonstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.

5.3.5 **Alternate setback and clearance.** Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official is permitted to require an investigation and recommendation of a registered design professional to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

SECTION 5.4
DESIGN OF FOOTINGS

5.4.0 Spread footings shall be designed and constructed in accordance with Sections 5.4.1 through 5.4.4.

5.4.1 **General.** Footings shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that total and differential settlements are tolerable. The design of footings shall be under the direct supervision of a registered design professional who shall certify to the building official that the footing satisfies the design criteria. The minimum width of footings shall be 300 mm. Footings in areas with expansive soils shall be designed in accordance with the provisions of Chapter 9. Footings in areas with collapsible soils shall be designed in accordance with the provisions of Chapter 10. Footings in areas with sabkha soils shall be designed in accordance with the provisions of Chapter 11. Footings subject to vibratory loads shall be designed in accordance with the provisions of Chapter 12.

5.4.1.1 **Design loads.** Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in SBC 301 Section 2.4. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in SBC 301 Section 4.8, are permitted to be used in designing footings.

5.4.1.2 **Eccentric loads**. When the footings are subjected to moments or eccentric loads, the maximum stresses shall not exceed the allowable bearing capacity of the soil specified in Chapter 4. The centeroid of the loads exerted on the footings shall coincide with the centeroid of the footing area, and if not possible the eccentricity shall not exceed 1/6 times the dimension of the footing in both sides. For the purpose of estimating the ultimate load-bearing, use shall be made of the effective width taken as the actual width minus twice the eccentricity.

5.4.1.3 **Inclined loads.** For design of footings subjected to inclined loads, it shall be permitted to use the following simplified formula or any method of analysis, subject to the approval of the building official.

$$\frac{V}{P_v} + \frac{H}{P_h} < 1.0 \qquad \textbf{(Equation 5-1)}$$

where:

V = Vertical component of inclined load.

H = Horizontal component of inclined load.

P_v = Allowable vertical load.

P_h = Allowable horizontal load.

Horizontal component shall not exceed soil passive resistance along the footing vertical edge and friction resistance at the footing soil interface taking a factor of safety of 2.

5.4.1.4 **Adjacent loads**. Where footings are placed at varying elevations the effect of adjacent loads shall be included in the footing design.

5.4.1.5 **Design settlements.** Settlements shall be estimated by a registered design professional based on methods of analysis approved by the building official. The least value found from Tables 5.1 and 5.2 shall be taken as the allowable differential settlement.

Exceptions: Structures designed to stand excessive total settlement in coastal areas or heavily loaded structures, like silos and storage tanks, shall be allowed to exceed these limits subject to a recommendation of a registered design professional and approval of a building official.

TABLE 5.1
MAXIMUM ALLOWABLE TOTAL SETTLEMENT

FOOTING TYPE	TOTAL SETTLEMENT (mm)	
	CLAY	SAND
Spread Footings	60	40
Mat Foundations	80	60

TABLE 5.2
MAXIMUM ALLOWABLE ANGULAR DISTORTION[a]

BUILDING TYPE	L/H	δ/l
Multistory reinforced concrete structures founded on mat foundation	---	0.0015
Steel frame structure with side sway	---	0.008
Reinforced concrete or steel structure with interior or exterior glass or panel cladding	---	0.002-0.003
Reinforced concrete or steel structure with interior or exterior glass or panel cladding	$\geq$ 5 $\leq$ 3	0.002 0.001
Slip and high structures as silos and water tanks founded on stiff mat foundations	---	0.002
Cylindrical steel tank with fixed cover and founded on flexible footing	---	0.008
Cylindrical steel tank with portable cover and founded on flexible footing	---	0.002-0.003
Rail for supporting hanged lift	---	0.003

a. L = Building length
l = Span between adjacent footings
H = Overall height of the structure
δ = Differential settlement

5.4.1.6 **Factor of safety.** Factor of safety shall not be less than 3 for permanent structures and 2 for temporary structures. Consideration shall be given to all possible circumstances including, but not limited to, flooding of foundation soil, removal of existing overburden by scour or excavation, and change in groundwater table level.

5.4.2 **Concrete footings.** The design, materials and construction of concrete footings shall comply with Sections 5.4.2.1 through 5.4.2.8 and the provisions of SBC 304 where applicable.

Exception: Where a specific design is not provided, concrete footings supporting walls of light-frame construction are permitted to be designed in accordance with Table 5.3.

5.4.2.1 **Concrete strength.** Concrete in footings shall have a specified compressive strength ($f_c^{'}$) of not less than 20 MPa at 28 days.

5.4.2.2 **Footing seismic ties.** Where a structure is assigned to Seismic Design Category D in accordance with Chapters 9 through 16, SBC 301, individual spread footings founded on soil defined in Section 9.4.2, SBC 301 as Site Class E or F shall be interconnected by ties. Ties shall be capable of carrying, in tension or compression, a force equal to the product of the larger footing load times the seismic coefficient S_{DS} divided by 10 unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.

TABLE 5.3
FOOTINGS SUPPORTING WALLS OF LIGHT-FRAME CONSTRUCTION [a, b, c, d, e]

NUMBER OF FLOORS SUPPORTED BY THE FOOTING[f]	WIDTH OF FOOTING (mm)	THICKNESS OF FOOTING (mm)
1	300	150
2	375	150
3	450	200

a. Depth of footings shall be in accordance with Section 5.2.
b. The ground under the floor is permitted to be excavated to the elevation of the top of the footing.
c. Interior-stud-bearing walls are permitted to be supported by isolated footings. The footing width and length shall be twice the width shown in this table, and footings shall be spaced not more than 1800 mm on center.
d. See SBC 304 Chapter 21 for additional requirements for footings of structures assigned to Seismic Design Category C or D.
e. For thickness of foundation walls, see Chapter 6.
f. Footings are permitted to support a roof in addition to the stipulated number of floors. Footings supporting roof only shall be as required for supporting one floor.

5.4.2.3 **Placement of concrete.** Concrete footings shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water.

5.4.2.4 **Protection of concrete.** Water shall not be allowed to flow through the deposited concrete.

5.4.2.5 **Forming of concrete.** Concrete footings are permitted to be cast against the earth where, in the opinion of the building official, soil conditions do not require forming. Where forming is required, it shall be in accordance with Chapter 6 of SBC 304.

5.4.2.6 **Minimum concrete cover to reinforcement**. When the concrete of footings is poured directly on the ground or against excavation walls the minimum concrete cover to reinforcement shall not be less than 75 mm. This cover shall also satisfy other requirements with regard to concrete exposure conditions presented in SBC 304.

5.4.2.7 **Dewatering.** Where footings are carried to depths below water level, the footings shall be constructed by a method that will provide the depositing or construction of sound concrete in the dry.

5.4.3 **Steel grillage footings.** Grillage footings of structural steel shapes shall be separated with approved steel spacers and be entirely encased in concrete with at least 150 mm on the bottom and at least 100 mm at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.

SECTION 5.5
DESIGNS EMPLOYING LATERAL BEARING

5.5.0 Designs to resist both axial and lateral loads employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth shall conform to the requirements of Sections 5.5.1 through 5.5.3.

5.5.1 **Limitations.** The design procedures outlined in this section are subject to the following limitations:

1. The frictional resistance for structural walls and slabs on silts and clays shall be limited to one-half of the normal force imposed on the soil by the weight of the footing or slab.
2. Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided that develops the limited deflection required.

5.5.2 **Design criteria.** The depth to resist lateral loads shall be determined by the design criteria established in Sections 5.5.2.1 through 5.5.2.3, or by other methods approved by the building official.

5.5.2.1 **Nonconstrained.** The following formula shall be used in determining the depth of embedment required to resist lateral loads where no constraint is provided at the ground surface, such as rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as a structural diaphragm.

$$d = 0.5A\{1+[1+(4.36h/A)]^{1/2}\} \qquad \textbf{(Equation 5-2)}$$

where:

d = Depth of embedment in earth in meter but not over 3600 mm for purpose of computing lateral pressure.

h = Distance in meter from ground surface to point of application of "P".

A $= 2.34P/S_1\,b$

P = Applied lateral force in kN.

S_1 = Allowable lateral soil-bearing pressure as set forth in Section 4.3 based on a depth of one-third the depth of embedment in kPa.

b = Diameter of round post or footing or diagonal dimension of square post or footing, meter.

5.5.2.2 **Constrained.** The following formula shall be used to determine the depth of embedment required to resist lateral loads where constraint is provided at the ground surface, such as a rigid floor or pavement.

$$d^2 = 4.25(Ph/S_3\,b) \qquad \textbf{(Equation 5-3)}$$

or alternatively

$$d^2 = 4.25(M_g/S_3\,b) \qquad \textbf{(Equation 5-4)}$$

where:

M_g = Moment in the post at grade, in kN-m.

S_3 = Allowable lateral soil-bearing pressure as set forth in Section 4.3 based on a depth equal to the depth of embedment in kPa.

5.5.2.3 **Vertical load.** The resistance to vertical loads shall be determined by the allowable soil-bearing pressure set forth in Table 4.1.

5.5.3 **Backfill.** The backfill in the space around columns not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with compressive strength of 15 MPa at 28 days. The hole shall not be less than 100 mm larger than the diameter of the column at its bottom or 100 mm larger than the diagonal dimension of a square or rectangular column.

2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 200 mm in depth.

3. Backfill shall be of controlled low-strength material (CLSM) placed in accordance with Section 3.11.

SECTION 5.6
SEISMIC REQUIREMENTS

5.6.0 For footings of structures assigned to Seismic Design Category C or D, provisions of SBC 301 and SBC 304 shall apply when not in conflict with the provisions of Chapter 5.

CHAPTER 6
FOUNDATION WALLS

SECTION 6.1
GENERAL

6.1.0 Concrete and masonry foundation walls shall be designed in accordance with SBC 304 or SBC 305. Foundation walls that are laterally supported at the top and bottom and within the parameters of Tables 6.1 through 6.3 are permitted to be designed and constructed in accordance with Sections 6.2 through 6.6.

SECTION 6.2
FOUNDATION WALL THICKNESS

6.2.0 The minimum thickness of concrete and masonry foundation walls shall comply with Sections 6.2.1 through 6.2.3.

6.2.1 **Thickness based on walls supported.** The thickness of foundation walls shall not be less than the thickness of the wall supported, except that foundation walls of at least 200 mm nominal width are permitted to support brick-veneered frame walls and 250 mm cavity walls provided the requirements of Section 6.2.2 are met. Corbelling of masonry shall be in accordance with Section 4.2, SBC 305. Where a 200 mm wall is corbelled, the top corbel shall be a full course of headers at least 150 mm in length, extending not higher than the bottom of the floor framing.

TABLE 6.1
200-mm CONCRETE AND MASONRY FOUNDATION WALLS WITH REINFORCING WHERE EFFECTIVE DEPTH $d \geq$ 125 mm[a,b,c]

WALL HEIGHT (mm)	HEIGHT OF UNBALANCED BACKFILL (mm)	VERTICAL REINFORCEMENT		
		Soil classes and lateral soil loads[a] (kPa per meter below natural grade)		
		GW, GP, SW and SP soils (5)	GM, GC, SM, SM-SC and ML soils (7)	SC, MH, ML-CL and Inorganic CL soils (9)
2100	1200 (or less)	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.
	1500	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1000 o.c.
	1800	Dia 12 at 1200 o.c.	Dia 16 at 1200 o.c.	Dia 16 at 1000 o.c.
	2100	Dia 12 at1000 o.c.	Dia 16 at 1000 o.c.	Dia 18 at 1200 o.c.
2400	1200 (or less)	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.
	1500	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1000 o.c.
	1800	Dia 12 at 1200 o.c.	Dia 16 at 1200 o.c.	Dia 16 at 1000 o.c.
	2100	Dia 16 at 1200 o.c.	Dia 18 at 1200 o.c.	Dia 18 at 1000 o.c.
	2400	Dia 16 at 1000 o.c.	Dia 18 at 1000 o.c.	Dia 22 at 1000 o.c.
2700	1200 (or less)	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.
	1500	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 16 at 1200 o.c.
	1800	Dia 12 at 1200 o.c.	Dia 16 at 1200 o.c.	Dia 18 at 1200 o.c.
	2100	Dia 16 at 1200 o.c.	Dia 18 at 1200 o.c.	Dia 22 at 1200 o.c.
	2400	Dia 16 at 1000 o.c.	Dia 22 at 1200 o.c.	Dia 25 at 1200 o.c.
	2700	Dia 18 at 1000 o.c.	Dia 25 at 1200 o.c.	Dia 25 at 800 o.c.

a. For design lateral soil loads, see SBC 301 Section 5.1. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.
b. Provisions for this table are based on construction requirements specified in Section 6.3.
c. For alternative reinforcement, see Section 6.4.

6.2.2 **Thickness based on soil loads, unbalanced backfill height and wall height.** The thickness of foundation walls shall comply with the requirements of Table 6.1, 6.2 or 6.3. When using the tables, masonry shall be laid in running bond and the mortar shall be Type M or S.

TABLE 6.2
250-mm CONCRETE AND MASONRY FOUNDATION WALLS WITH REINFORCING WHERE EFFECTIVE DEPTH $d \geq$ 175 mm[a,b,c]

WALL HEIGHT	HEIGHT OF UNBALANCED BACKFILL	VERTICAL REINFORCEMENT		
		Soil classes and lateral soil load[a] (kPa per meter below natural grade)		
		GW, GP, SW and SP soils (5)	GM, GC, SM, SM-SC and ML soils (7)	SC, MH, ML-CL and Inorganic CL soils (9)
2100	1200 (or less)	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.
	1500	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.
	1800	Dia 12 at 1400 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1000 o.c.
	2100	Dia 12 at 1400 o.c.	Dia 16 at 1400 o.c.	Dia 16 at 1000 o.c.
2400	1200 (or less)	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.
	1500	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1200 o.c.
	1800	Dia 12 at 1400 o.c.	Dia 12 at 1200 o.c.	Dia 16 at 1400 o.c.
	2100	Dia 12 at 1200 o.c.	Dia 12 at 800 o.c.	Dia 18 at 1400 o.c.
	2400	Dia 16 at 1400 o.c.	Dia 16 at 1000 o.c.	Dia 22 at 1400 o.c.
2700	1200 (or less)	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.
	1500	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1200 o.c.
	1800	Dia12 at 1400 o.c.	Dia 12 at 1000 o.c.	Dia 12 at 800 o.c.
	2100	Dia 12 at 1000 o.c.	Dia 16 at 1200 o.c.	Dia 18 at 1200 o.c.
	2400	Dia 12 at 800 o.c.	Dia 18 at 1200 o.c.	Dia 12 at 400 o.c.
	2700	Dia16 at 1000 o.c.	Dia 18 at 1000 o.c.	Dia 22 at 1000 o.c.

a. For design lateral soil loads, see SBC 301 Section 5.1. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.
b. Provisions for this table are based on construction requirements specified in Section 6.3.
c. For alternative reinforcement, see Section 6.4.

TABLE 6.3
300-mm CONCRETE AND MASONRY FOUNDATION WALLS WITH REINFORCING WHERE EFFECTIVE DEPTH $d \geq$ 225 mm[a,b,c]

WALL HEIGHT (mm)	HEIGHT OF UNBALANCED BACKFILL (mm)	VERTICAL REINFORCEMENT		
		Soil classes and lateral soil load[a] (kPa per meter below natural grade)		
		GW, GP, SW and SP soils (5)	GM, GC, SM, SM-SC and ML soils (7)	SC, MH, ML-CL and Inorganic CL soils (9)
2100	1200 (or less)	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.
	1500	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.
	1800	Dia 12 at 1800 o.c.	Dia 12 at 1600 o.c.	Dia 12 at 1200 o.c.
	2100	Dia 12 at 1800 o.c.	Dia 12 at 1200 o.c.	Dia 16 at 1400 o.c.
2400	1200 (or less)	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.
	1500	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.
	1800	Dia 12 at 1800 o.c.	Dia 12 at 1400 o.c.	Dia 16 at 1800 o.c.
	2100	Dia 12 at 1600 o.c.	Dia 16 at 1400 o.c.	Dia 12 at 800 o.c.
	2400	Dia 16 at 1200 o.c.	Dia 12 at 800 o.c.	Dia 16 at 1000 o.c.
2700	1200 (or less)	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.
	1500	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1600 o.c.
	1800	Dia 12 at 1800 o.c.	Dia 12 at 1400 o.c.	Dia 16 at 1600 o.c.
	2100	Dia 12 at 1400 o.c.	Dia 12 at 1000 o.c.	Dia 18 at 1600 o.c.
	2400	Dia 12 at 1600 o.c.	Dia 18 at 1600 o.c.	Dia 18 at 1200 o.c.
	2700	Dia 16 at 1400 o.c.	Dia 22 at 1800 o.c.	Dia 18 at 1000 o.c.

a. For design lateral soil loads, see SBC 301 Section 5.1. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.
b. Provisions for this table are based on construction requirements specified in Section 6.3.
c. For alternative reinforcement, see Section 6.4.

Unbalanced backfill height is the difference in height of the exterior and interior finish ground levels. Where an interior concrete slab is provided, the unbalanced backfill height shall be measured from the exterior finish ground level to the top of the interior concrete slab.

6.2.3 **Rubble stone.** Foundation walls of rough or random rubble stone shall not be less than 400 mm thick. Rubble stone shall not be used for foundations for structures in Seismic Design Category C or D.

SECTION 6.3
FOUNDATION WALL MATERIALS

6.3.0 Foundation walls constructed in accordance with Table 6.1, 6.2, and 6.3 shall comply with the following:

1. Vertical reinforcement shall have minimum yield strength of 420 MPa.
2. The specified location of the reinforcement shall equal or exceed the effective depth distance, *d*, noted in Tables 6.1, 6.2 and 6.3 and shall be measured from the face of the soil side of the wall to the center of vertical reinforcement. The reinforcement shall be placed within the tolerances specified in ACI 530.1/ ASCE 6/TMS 402, Article 3.4 B7 of the specified location.
3. Concrete shall have a specified compressive strength of not less than 20 MPa at 28 days.
4. Grout shall have a specified compressive strength of not less than 15 MPa at 28 days.
5. Hollow masonry units shall comply with ASTM C 90 and be installed with Type M or S mortar.

SECTION 6.4
ALTERNATIVE FOUNDATION WALL REINFORCEMENT

6.4.1 In lieu of the reinforcement provisions in Table 6.1, 6.2 or 6.3, alternative reinforcing bar sizes and spacings having an equivalent cross-sectional area of reinforcement per linear meter of wall are permitted to be used, provided the spacing of reinforcement does not exceed 1.8 m and reinforcing bar sizes do not exceed Dia 36 mm.

SECTION 6.5
HOLLOW MASONRY WALLS

6.5.1 At least 100 mm of solid masonry shall be provided at girder supports at the top of hollow masonry unit foundation walls.

SECTION 6.6
SEISMIC REQUIREMENTS

6.6.0 Tables 6.1 through 6.3 shall be subject to the following limitations in Sections 6.6.1 and 6.6.2 based on the seismic design category assigned to the structure as defined in Chapters 9 through 16 of SBC 301.

6.6.1 **Seismic requirements for concrete foundation walls.** Concrete foundation walls designed using Tables 6.1 through 6.3 shall be subject to the following limitations:

1. Seismic Design Categories A and B. No limitations, except provide not less than two Dia 16 mm bars around window and door openings. Such bars shall extend at least 600 mm beyond the corners of the openings.

6.6.2 **Seismic requirements for masonry foundation walls.** Masonry foundation walls designed using Tables 6.1 through 6.3 shall be subject to the following limitations:

1. Seismic Design Categories A and B. No additional seismic requirements.

2. Seismic Design Category C. A design using Tables 6.1 through 6.3 subject to the seismic requirements of Section 6.1.4 SBC 305.

3. Seismic Design Category D. A design using Tables 6.1 through 6.3 subject to the seismic requirements of Section 6.1.5 SBC 305.

SECTION 6.7
FOUNDATION WALL DRAINAGE

6.7.1 Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 13.4.2 and 13.4.3.

SECTION 6.8
PIER AND CURTAIN WALL FOUNDATIONS

6.8.1 Except in Seismic Design Category D, pier and curtain wall foundations are permitted to be used to support light-frame construction not more than two stories in height, provided the following requirements are met:

1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.

2. The minimum actual thickness of a load-bearing masonry wall shall not be less than 100 mm nominal or 90 mm actual thickness, and shall be bonded integrally with piers spaced 1800 mm on center (o.c.).

3. Piers shall be constructed in accordance with SBC 305 and the following:

a) The unsupported height of the masonry piers shall not exceed 10 times their least dimension.

b) Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.

Exception: Unfilled hollow piers are permitted where the unsupported height of the pier is not more than four times its least dimension.

c) Hollow piers shall be capped with 100 mm of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.

4. The unbalanced fill for 100 mm foundation walls shall not exceed 600 mm for solid masonry, and 300 mm for hollow masonry.

SECTION 6.9
SEISMIC REQUIREMENTS

6.9.1 For foundations of structures assigned to Seismic Design Category C or D, provisions of SBC 301 and SBC 304 shall apply when not in conflict with the provisions of Chapter 6.

CHAPTER 7
RETAINING WALLS

SECTION 7.1
GENERAL

7.1.1 **SCOPE.** This Chapter shall apply to all matters pertaining to design and construction of rigid gravity, semi gravity, cantilever, buttressed, and counterfort retaining walls. For special types of retaining walls, provisions of this code shall apply where applicable. General safeguards during construction shall comply with provisions of Chapter 3.

SECTION 7.2
LATERAL EARTH PRESSURES

7.2.0 Computations of lateral earth pressures shall comply with the provisions of Sections 7.2.1 through 7.2.6. Wall movements set forth in Table 7.1 shall be considered the magnitude required for active and passive conditions to exist. Soil permeability characteristics, boundary drainage and loading conditions, and time shall be considered in selection of strength parameters. In soils where partial drainage occurs during the time of construction, analysis shall be performed for short-term and long-term conditions, and the wall shall be designed for the worse conditions.

7.2.1 **Wall friction.** Wall friction and vertical movement, slope of the wall in the backside and sloping backfill shall be considered in determining the lateral pressures applied against the wall. Unless data to substantiate the use of other values are submitted and approved by a registered design professional, the values set forth in Tables 7-2 and 7-3 shall be used in computations that include effects of wall friction.

TABLE 7.1
MAGNITUDE OF ROTATION TO REACH FAILURE

SOIL TYPE AND CONDITION	ROTATION (δ/H)[a]	
	ACTIVE	PASSIVE
Dense cohesionless soil	0.0005	0.002
Loose cohesionless soil	0.002	0.006
Stiff cohesive soil	0.01	0.02
Soft cohesive soil	0.02	0.04

a. δ = Horizontal translation at the top of the wall.
H = Height of the wall

TABLE 7.2
ULTIMATE FRICTION FACTORS FOR DISSIMILAR MATERIALS

INTERFACE MATERIALS	FRICTION FACTOR, tan δ [a]
Clean sound rock	0.7
Clean gravel, gravel-sand mixtures, coarse sand	0.55 – 0.60
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 – 0.55
Clean fine sand, silty or clayey fine to medium sand	0.35 – 0.45
Fine sandy silt, nonplastic silt	0.30 – 0.35
Very stiff and hard residual or preconsolidated clay	0.40 – 0.50
Medium stiff and stiff clay and silty clay	0.30 – 0.35

a. Values for δ shall not exceed one-half the angle of internal friction of the backfill soils for steel and precast concrete and two-third the angle of internal friction of the backfill soils for cast-in place concrete.

7.2.2 **Wall movement.** The effect of wall movement on the earth pressure coefficients shall conform to the provisions of Sections 7.2.2.1 and 7.2.2.2.

7.2.2.1 **Rotation.** If the wall is free at the top and there are no other structures associated with, wall tilting shall not exceed 0.1 times the height of the wall. Where the actual estimated wall rotation is less than the value required to fully mobilize active or passive conditions set forth in Table 7-1, the earth pressure coefficient shall be adjusted in accordance with Figure 7.1.

TABLE 7.3
ULTIMATE ADHESION FOR DISSIMILAR MATERIALS

INTERFACE MATERIALS	COHESION (kPa)	ADHESION (kPa)
Very soft cohesive soil	0 - 10	0 - 10
Soft cohesive soil	10 - 25	10 - 25
Medium stiff cohesive soil	25 - 50	25 - 35
Stiff cohesive soil	50 - 100	35 - 45
Very stiff cohesive soil	100 - 200	45 - 60

7.2.2.2 **Translation.** It shall be permitted to consider uniform translation required to mobilize ultimate passive resistance or active pressure equivalent to movement of top of wall based on rotation given in Table 7.1.

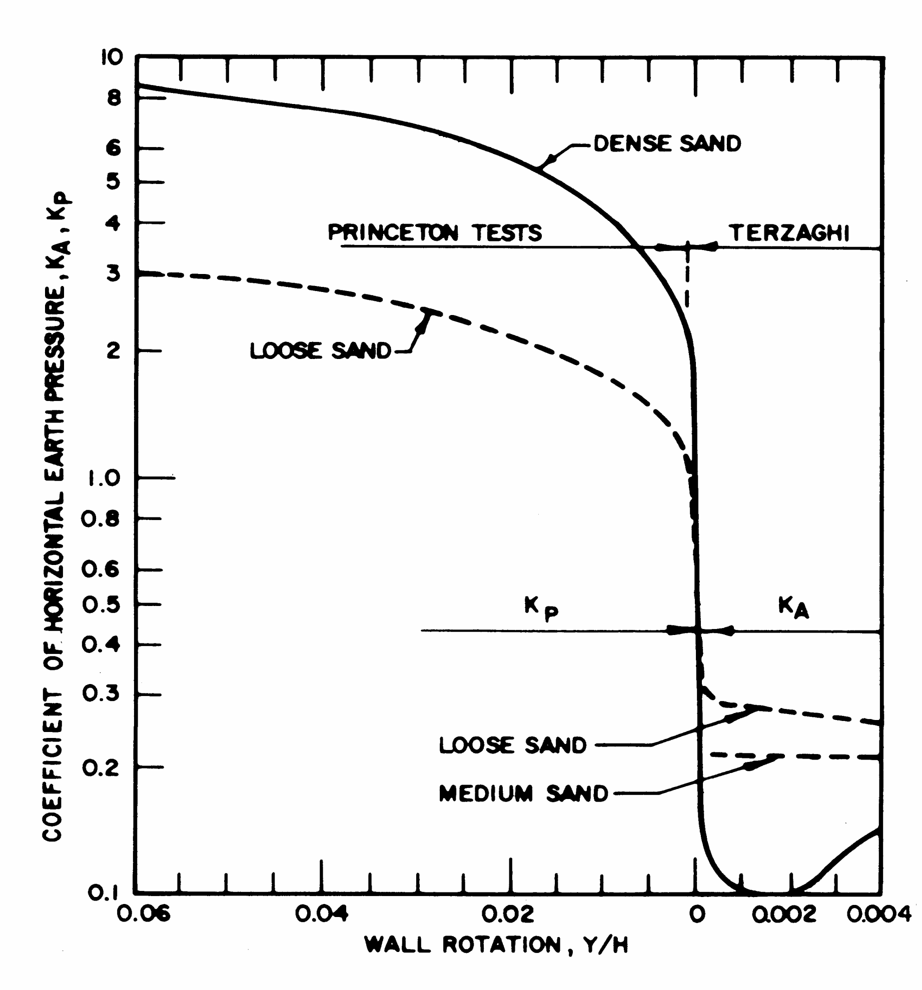


FIGURE 7.1
EFFECT OF WALL MOVEMENT ON WALL PRESSURES (NAFAC, 1986)

7.2.2.3 **Restrained wall**. Where wall is prevented from even slight movement, the earth pressure shall be considered to remain at rest conditions.

7.2.2.4 **Basement and other below grade walls**. Pressures on walls below grade shall be computed based on restrained conditions that prevail, type of backfill, and the amount of compaction. The provisions of Chapter 6 shall apply where applicable.

7.2.2.5 **Wall on rock**. Where the wall is founded on rock, sufficient rotation of the base and wall so that active pressure is developed, shall be accomplished by placing 150 to 300 mm thick earth pad beneath the base and by constructing the stem with sufficient flexibility to yield with the soil pressure.

7.2.3 **Groundwater conditions**. Pressure computations shall include uplift pressures and the effect of the greatest unbalanced water head anticipated to act across the wall. For cohesionless materials, increase in lateral force on wall due to rainfall shall be considered and walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 13.4.2 and 13.4.3.

7.2.4 **Surcharge.** Stability shall be checked with and without surcharge. Lateral pressure on wall due to point and line loads shall be computed based on the assumption of an unyielding rigid wall and the lateral pressures are set equal to double the values obtained by elastic equations. The applicability of the assumption of an unyielding rigid wall shall be evaluated for each specific wall.

For uniform surcharge loading it shall be permitted to compute lateral stress by treating the surcharge as if it were backfill and multiplying the vertical stress at any depth by the appropriate earth pressure coefficient. It shall be permitted for design purposes to considerer a distributed surface load surcharge on the order of 15 kPa to account for construction materials and equipment stored within 5 to 10 meters from the wall. Where construction equipment is anticipated within 2 meters of the wall, it must be accounted for separately.

7.2.5 **Compaction**. For backfill of granular soils compacted in a confined wedge behind the wall, the horizontal pressure beyond those represented by active or at rest values shall be computed in accordance with Figure 7.2.

Compaction-induced pressures shall not be considered in bearing, overturning and sliding analyses and need only be considered for structural design. Backfill shall be brought up equally on both sides until the lower side finished grade is reached and precautions shall be taken to prevent over-compaction which will cause excessive lateral forces to be applied to the wall.

Clays and other fine-grained soils, as well as granular soils, with amount of clay and silt greater than 15 percent shall not be used as a backfill behind retaining wall. Where they must be used, the lateral earth pressure shall be calculated based on at rest conditions, with due consideration to potential poor drainage conditions and swelling. Where loose hydraulic fill is used it shall be placed by procedures which permit runoff of wash water and prevent building up of large hydrostatic pressures.

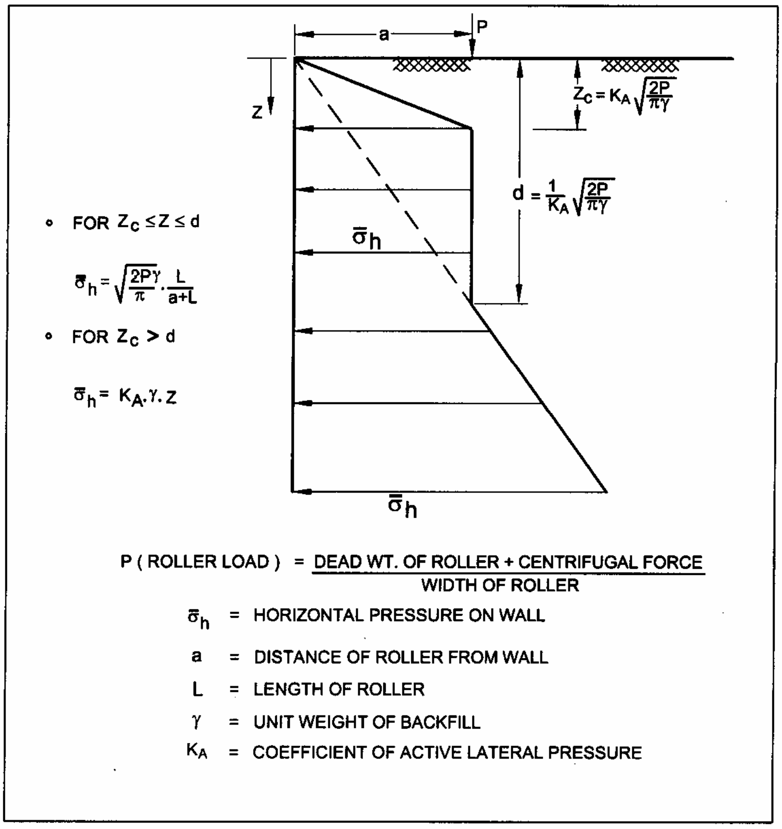


FIGURE 7.2
HORIZONTAL PRESSURE ON WALLS FROM COMPACTION EFFORT (NAFAC, 1986)

7.2.6 **Earthquake loading**. For retaining walls assigned to Seismic Design Category C or D, provisions of SBC 301 and SBC 304 shall apply when not in conflict with the provisions of Chapter 7.

The combined resultant active force due to initial static pressure and increase in pressure from ground motion shall be computed from the following formula

$$P_{AE} = \frac{1}{2}\gamma H^2 k_A(\beta^*,\theta^*)(1-k_v)\frac{\cos^2\theta^*}{\cos\psi\cos^2\theta} \qquad \textbf{(Equation 7-1)}$$

where:

P_{AE} = Combined resultant active force.

H = Wall height.

θ = Slope of wall back with respect to vertical

β = Inclination of soil surface (upward slopes away from the wall are positives).

θ* = $(\theta+\psi)$ = Modified slope of wall back.

β* = $(\beta+\psi)$ = Modified inclination of soil surface.

γ = Unit weight of soil.

ψ = Seismic inertia angle given as follows

$$\psi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right) \qquad \textbf{(Equation 7-2)}$$

where:

k_h = Horizontal ground acceleration in g's.

k_v = Vertical ground acceleration in g's.

For modified slope angle β^* and θ^*, the modified coefficient of earth pressures $k_A(\beta^*, \theta^*)$ shall be calculated from the Coulomb theory. Dynamic pressure increment shall be obtained by subtracting static active force (to be determined from Coulomb theory for given β and θ) from combined active force given by Equation 7-1. Location of resultant shall be obtained by considering the earth pressure to be composed of a static and dynamic component with the static component acts at the lowest third point, whereas the dynamic component acts above the base at 0.6 times the height of the wall. Under the combined effect of static and earthquake load the factor of safety shall not be less than 1.2.

Where soil is below water, the hydrodynamic pressure computed from the following formula shall be added

$$p_{wz} = \frac{3}{2} k_h \gamma_w (h_w z)^{1/2} \qquad \textbf{(Equation 7-3)}$$

where:

p_{wz} = Hydrodynamic pressure at depth z below water surface

h_w = Height of water

z = Depth below the water surface.

γ_w = Unit weight of water.

SECTION 7.3 BEARING CAPACITY

7.3.1 The allowable soil pressure shall be determined in accordance with the provisions of Chapter 4. The determination of the allowable bearing pressure shall be made according to the bearing capacity of a foundation subjected to eccentric loads. The bearing capacity shall be checked for the same loading conditions as determined by the overturning analysis for each case analyzed. Where the wall is founded on sloped ground, methods for determination of ultimate bearing capacity that deal with this situation shall be used. The factor of safety with respect to bearing capacity shall not be less than 3.

For walls founded on rocks, high toe pressure that may cause breaking the toe from the remainder of the base shall be avoided by proportioning the footing so that the resultant falls near its center.

SECTION 7.4 STABILITY

7.4.0 Retaining walls shall be designed to ensure stability against overturning, sliding, and stability of supporting ground. Stability analyses shall conform to the provisions of Sections 7.4.1 through 7.4.4.

7.4.1 **Sliding stability**. The base shall be at least 1000 mm below ground surface in front of the wall. Sliding stability shall be adequate without including passive pressure at the toe. Where insufficient sliding resistance is available, one provision shall be taken including, but need not be limited to, increasing the width of the wall base, founding the wall on piles or lowering the base of the wall. If the wall is supported by rock or very stiff clay, it shall be permitted to install a key below the foundation to provide additional resistance to sliding. The key shall conform to the provisions of Section 7.4.4. The factor of safety against sliding shall not be less than 1.5 for cohesionless backfill and 2.0 for cohesive backfill.

7.4.2 **Overturning stability**. For walls on relatively incompressible foundations, overturning check is ignored if the resultant is within the middle third of the base for walls founded on soils and if the resultant is within the middle half for walls founded on rocks. Where foundation is compressible settlement shall be computed based on any method approved by the building official. Tilt of rigid wall shall be obtained from the estimated settlement. Differential settlement shall be limited to the amount of tilting that shall not exceed 5 percent of wall height. If the consequent tilt exceeds acceptable limits, the wall shall be proportioned to keep the resultant force at the middle third of base. The retaining wall shall be proportioned so that the factor of safety against overturning is not less than 1.5. The value of angular distortion (settlement/length of structure) of retaining walls shall not exceed 0.002 radians.

7.4.3 **Deep-seated sliding**. Where retaining walls are underlain by weak soils, the overall stability of the soil mass containing the retaining wall shall be checked with respect to the most critical surface of sliding. The stability analysis shall be made for after construction and for long-term conditions. The factor of safety for the overall stability of the soil mass containing the wall shall not be less than 2.

7.4.4 **Wall with key**. Prior to performing an overturning analysis, the depth of the key and width of the base shall be determined from the sliding stability analysis. For a wall with a horizontal base and a key, it shall be permitted to assume the shearing resistance of the base to be zero and the horizontal resisting force acting on the key is that required for equilibrium. For a wall with a sloping base and a key, the horizontal force required for equilibrium shall be assumed to act on the base and the key In both cases the resisting soil force down to the bottom of the toe shall be computed using at-rest earth pressure if the material on the resisting side will not lose its resistance characteristics with any probable change in water content or environmental conditions and will not be eroded or excavated during the life of the wall.

SECTION 7.5
WALL DIMENSIONS

7.5.1 Thickness of the upper part of the wall shall not be less than 300 mm, where as thickness of the lower part of the wall shall be enough to resist shear without reinforcement. Depth of wall foundation shall be located below line of seasonal changes and shall be deep enough to provide adequate bearing capacity and soil sliding resistance. The wall foundation shall be proportioned such that the wall does not slide or overturn, the allowable bearing capacity of the soil is not exceeded, and that total and differential settlements are tolerable. The base and other dimensions shall be such that the resultant falls within the middle third of the base. Where additional front clearance is needed, it shall be permitted to construct counterfort retaining walls without a toe provided that the sliding and overturning stability requirements stated in Sections 7.4.1 and 7.4.2 are met.

SECTION 7.6
WALL CONSTRUCTION

7.6.0 Concrete shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water and that will provide the depositing or construction of sound concrete in the dry.

7.6.1 **Minimum concrete cover to reinforcement**. When the concrete of retaining walls is poured directly on the ground or against excavation walls the minimum concrete cover to reinforcement shall not be less than 75 mm and not less than 40 mm when concrete is poured against lean concrete or vertical forms. This cover shall also satisfy other requirements with regard to concrete exposure conditions presented in SBC 304.

7.6.2 **Joints**. Construction and expansion joints shall be provided where needed. Construction joints shall be constructed into a retaining wall between successive pours of concrete both horizontally and vertically. Horizontal construction joints shall be kept to a minimum and the top surface of each lift shall be cleaned and roughened before placing the next lift. Long walls shall have expansion joints at intervals of 10 meters. Where vertical-expansion joints are considered, they shall be placed along the wall at spacing of 20 to 30 meters. Reinforcing steel and other fixed metal embedded or bonded to the surface of the concrete shall not extend through the expansion joint. For cantilever concrete walls, it shall be permitted to locate the vertical expansion joints only on the stem, and the footing is a continuous placement.

The thickness of joint filler necessary to provide stress relief at a joint shall be determined from the estimated initial contraction and subsequent expansion from maximum temperature variation.

7.6.3 **Drainage.** Regardless of the drainage system used, the wall must have an adequate factor of safety assuming the drainage system is inoperative. Where drainage measures are considered they shall be designed by a registered design professional and subject to the approval of the building official. As a minimum, there shall be weep holes with pockets of coarse-grained material at the back of the wall, and a gutter shall be provided for collecting runoff. All retaining walls shall have adequate surface drainage to dispose of surface water. A layer of impervious soil shall be placed on top of the soil backfill to reduce surface infiltration of rainfall. It shall be permitted to use inclined and horizontal drains in conjunction with back drain.

The weep holes shall be of sufficient size and be carefully surrounded with a granular filter or by the use of filter fabric on the backfill side and directly surrounding the entrance to the weep holes. The weep holes shall be spaced not more than 3 m apart vertically and horizontally. Where longitudinal drains along the back face are used, a layer of free-draining granular material shall be placed along the back of the wall and surrounding the drain pipes opening. The gradation of the filter shall satisfy the following piping or stability criterion.

$$\frac{d_{15\mathrm{F}}}{d_{85B}} \leq 5 \qquad \textbf{(Equation 7-4)}$$

where:

d_{15F} = size of filter material at 15 percent passing.

d_{85B} = size of protected soil at 85 percent passing.

and

$$\frac{d_{50\mathrm{F}}}{d_{50\mathrm{B}}} \leq 25 \qquad \textbf{(Equation 7-5)}$$

where:

d_{50F} = size of filter material at 50 percent passing.

d_{50B} = size of protected soil at 50 percent passing.

The filter material shall be more permeable than the material being drained and the following condition shall be met

$$4 < \frac{d_{15\mathrm{F}}}{d_{15\mathrm{B}}} < 20 \qquad \textbf{(Equation 7-6)}$$

where:

d_{15B} = size of filter protected soil at 15 percent passing.

Where a blanket of well-graded sand and gravel that is placed along the back of the wall it shall satisfied the requirements of Equations 7-4 through 7-6. Where longitudinal drains are used within drainage blanket, they shall be large enough to carry the discharge and have adequate slope to provide sufficient velocity to remove sediment from the drain. Segregation of sand and gravel during construction shall be avoided. Filter or drain materials contaminated by muddy water, dust, etc. shall be replaced and filter materials subject to cementation shall be rejected.

In lieu of a granular filter, it shall be permitted to use prefabricated geocomposite drains with adequate filter flow capacity and acceptable retention. The size of filter material at 50 percent passing, d_{50F}, shall not be less than the diameter of the hole for circular openings and shall be 1.2 times slot width for slotted openings. The drainage composite manufacture's recommendations for backfilling and compaction near the composite shall be followed.

CHAPTER 8
COMBINED FOOTINGS AND MATS

SECTION 8.1
GENERAL

8.1.0 Analysis and design of combined footings and mats shall conform to all requirements of ACI 336.2R Suggested Analysis and Design Procedures for Combined Footings and Mats except as modified by Chapter 8. All provisions of SBC 303 not specifically excluded, and not in conflict with the provisions of Chapter 8 shall apply to combined footings and mats, where applicable. Design of combined or mat foundations shall be based on the Strength Design Method of SBC 304.

Combined footings and mats shall be designed and constructed on the basis of a site investigation as defined in Chapter 2, unless the building official ascertains that sufficient data upon which to base the design and installation is available. The investigation and report provisions of Chapter 2 shall be expanded to include, but need not be limited to, the following:

1. Values for modulus of subgrade reaction.

2. Recommended combined footings shapes.

8.1.1 **Approval of special systems of design or construction**. Sponsors of any system of design or construction within the scope of Chapter 8, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by Chapter 8, shall have the right to present that data on which their design is based to the building official or to a board of examiners appointed by the building official. This board shall be composed of competent geotechnical and structural engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of Chapter 8. These rules when approved by the building official and promulgated shall be of the same force as the provisions of Chapter 8.

SECTION 8.2
LOADINGS

8.2.1 Combined footings and mats shall be designed for the most unfavorable effects due to the combinations of loads specified in SBC 301 Section 2.4. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in Section 4.8 SBC 301, are permitted to be used in designing footings. Strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

SECTION 8.3
CONCRETE

8.3.1 Material, construction, and placement of concrete shall be in accordance with the provisions of Section 5.4.2. For mats construction joints shall be carefully located at sections of low shear stress or at the center lines between columns. An elapse of at least 24 hours shall be left between pours of adjacent areas. If bar splicing is needed, sufficient overlapping shall be provided. The concrete shall be strong enough to transfer the shear stress across the joint. If necessary, the mat may be thickened to provide sufficient strength in the joints.

SECTION 8.4
CONTACT PRESSURE

8.4.0 Soil contact pressure acting on a combined footing or mat and the internal stresses produced by them shall be determined from one of the load combinations given in Section 2.4 SBC 301**,** whichever produces the maximum value for the element under investigation.

The combinations of unfactored loads which will produce the greatest contact pressure on a base area of given shape and size shall be selected. The allowable soil pressure shall be determined in accordance with the provisions of Chapter 4. Loads shall include the vertical effects of moments caused by horizontal components of these forces and by eccentrically applied vertical loads. Buoyancy of submerged parts where this reduces the factor of safety or increases the contact pressures, as in flood conditions shall be considered.

The maximum unfactored design contact pressures shall not exceed the allowable soil pressure as obtained from Chapter 4 or cause settlements that exceed the values set forth in Table 5.1 and 8.3. Where wind or earthquake forces form a part of the load combination, the allowable soil pressure may be increased as allowed by the Saudi Building Code or approved by the building official.

In determination of the contact pressures and associated subgrade response, the validity of simplifying assumptions and the accuracy of any resulting computations shall be approved by the building official and evaluated on the basis of the following variables:

1. The increased unit pressures developed along the edges of rigid footings on cohesive soils and at the center for rigid footings on cohesionless soils.

2. The effect of embedment of the footing on pressure variation.

3. Consideration in the analysis of the behavior of the foundations immediately after the structure is built as well as the effects of long-term consolidation of deep soil layers.

4. Consideration of size of the footing in determination of the modulus of subgrade reaction of soil.

5. The variation of contact pressures from eccentric loading conditions.

6. Consideration of the influence of the stiffness of the footing and the superstructure on deformations that can occur at the contact surface and the corresponding variation on contact pressure and redistribution of reactions occurring within the superstructure frame.

8.4.1 **Distribution of soil reactions**. Contact pressures at the base of combined footings and mats shall be determined in accordance with Sections 8.4.1.1 through 8.4.1.3.

8.4.1.1 **General.** Except for unusual conditions, the contact pressures at the base of a combined footings and mats may be assumed to follow either a distribution governed by elastic subgrade reaction or a straight-line distribution. At no place shall the calculated contact pressure exceed the allowable bearing capacity as determined from Chapter 4.

8.4.1.2 **Straight-line distribution of contact pressure.** It shall be permitted to assume a linear distribution for soil contact pressure if continuous footings meet the requirement of Section 8.7.1 and mats conform to the requirements of Section 8.9.3.2.

8.4.1.2.1 **Contact pressure over total base area**. If the resultant of all forces is such that all portions of the foundation contact area are in compression, the maximum and minimum soil pressure may then be calculated from the following formula, which applies only to the rectangular base areas and only when eccentricity is located along one of the principal axes of the footing

$$q_{\max,\min} = \frac{\sum P}{BL}\left(1 \pm \frac{6e}{L}\right) \qquad \textbf{(Equation 8-1)}$$

where:

$q_{\max}$ = Maximum soil contact pressure.

$q_{\min}$ = Minimum soil contact pressure.

P = Any force acting perpendicular to base area.

B = Foundation width or width of beam column element.

e = Eccentricity of resultant of all vertical forces with center of footing area ($e \le L/6$).

L = Foundation base length or length of beam column element.

For footings with eccentricity about both axes, soil pressure is obtained from

$$q = \frac{\sum P}{BL}\left(1 \pm \frac{6e_x}{L} \pm \frac{6e_y}{B}\right) \qquad \textbf{(Equation 8-2)}$$

where:

q = Soil contact pressure.

e_x = Eccentricity of resultant of all vertical forces with respect to the x-axis.

e_y = Eccentricity of resultant of all vertical forces with respect to the y-axis.

P = Any force acting perpendicular to base area.

8.4.1.2.2 **Contact pressure over part of area.** The soil pressure distribution shall be assumed to be triangular and the resultant has the same magnitude and colinear, but acts in the opposite direction of the resultant of the applied forces.

The maximum soil pressure at the footing edge under this condition shall be calculated from the following expression

$$q_{ult} = \frac{2P}{3B}\left(\frac{L}{2} - e\right) \qquad \textbf{(Equation 8-3)}$$

The minimum soil pressure at distance L_1 is set equal to zero, where L_1 is the footing effective length measured from the pressed edge to the position at which the contact pressure is zero and is given by

$$L_1 = 3\left(\frac{L}{2} - e\right) \qquad \textbf{(Equation 8-4)}$$

Equations 8-3 and 8-4 are applied based on the assumption that no tensile stresses exist between footing and soil and for cases where the resultant force falls out of the middle third of the base.

8.4.1.3 **Distribution of contact pressure governed by the modulus of subgrade reaction.** It shall be permitted to get the distribution of contact pressure based on modulus of subgrade reaction obtained from Section 8.4.1.3.2. The thickness shall be sized for shear without using reinforcement. The flexural steel is then obtained by assuming a linear soil pressure distribution and using simplified procedures in which the foundation satisfies static equilibrium. The flexural steel may also be obtained by assuming that the foundation is an elastic member interacting with an elastic soil.

8.4.1.3.1 **Beams on elastic foundations.** If the combined footing is assumed to be a flexible slab, it may be analyzed as a beam on elastic foundation. It shall be permitted to analyze a beam on elastic foundation using the discrete element method, the finite element method, or other methods approved by the building official.

8.4.1.3.2 **Estimating the modulus of subgrade reaction.** The value for modulus of subgrade reaction may be obtained from one of the methods in Sections 8.4.1.3.2.1 through 8.4.1.3.2.3. It shall be permitted to use a constant value for the modulus of subgrade reaction except where the rigidity of the footing and superstructure is considered small, the decrease in the value of modulus of subgrade reaction, k_s, with increasing applied load shall be taken into consideration.

8.4.1.3.2.1 **Presumptive modulus of subgrade reaction values**. It shall be permitted to use values for the modulus of subgrade reaction for supporting soils as set forth in Table 8.1 and 8.2 to determine about the correct order of magnitude of the subgrade modulus obtained from Sections 8.4.1.3.2.1 through 8.4.1.3.2.5. The tables' values shall be used only as a representative guide.

8.4.1.3.2.2 **Modulus of subgrade reaction from plate load test**. For mat foundations, this soil property shall not be estimated on the basis of field plate load tests and shall be obtained using subgrade reaction theory, but shall be modified to individually consider dead loading, live loading, size effects, and the associated subgrade response. Zones of different constant subgrade moduli shall be considered to provide a more accurate estimate of the subgrade response as compared to that predicted by a single modulus of subgrade reaction.

TABLE 8.1
PRESUMPTIVE MODULUS OF SUBGRADE REACTION VALUES FOR COHESIONLESS SOILS

RELATIVE DENSITY	UNCORRECTED SPT VALUES	MODULUS OF SUBGRADE REACTION (kN/m^3)	
		DRY AND MOIST SOILS	SUBMERGED SOILS
Loose	Less Than 10	15000	10000
Medium dense	10-30	45000	30000
Dense	> 30	175000	100000

TABLE 8.2
PRESUMPTIVE MODULUS OF SUBGRADE REACTION VALUES FOR COHESIVE SOILS

CONSISTENCY	SHEAR STRENGTH FROM UNCONFINED COMPRESSION TEST (kPa)	MODULUS OF SUBGRADE REACTION (kN/m^3)
Stiff	105-215	25000
Very stiff	215-430	50000
Rigid	> 430	100000

The value for the modulus of subgrade reaction for use in elastic foundation analysis may be estimates from a plate load test carried out in accordance with ASTM D1194. Since plate load tests are of necessity on small plates, great care must be exercised to insure that results are properly extrapolated. The modulus of subgrade reaction from plate load test shall be converted to that of mat using the following formula

$$k_s = k_p \left(\frac{B_p}{B_m} \right)^n \qquad \textbf{(Equation 8-5)}$$

where:

k_s = Coefficient (or modulus) of vertical subgrade reaction; generic term dependent on dimensions of loaded area.

k_p = Coefficient of subgrade reaction from a plate load test.

B_m = Mat width.

B_p = Plate width.

n = Factor that ranges from 0.5 to 0.7.

Allowance shall be made for the depth of compressible strata beneath the mat and if it is less than about four times the width of footing, lower values shall be used for n.

8.4.1.3.2.3 **Modulus of subgrade reaction from elastic parameters.** It shall be permitted to estimate the value for the modulus of subgrade reaction based on laboratory or in situ tests to determine the elastic parameters of the foundation material. This shall be done by numerically integrating the strain over the depth of influence to obtain a settlement ΔH and back computing k_s as

$$k_s = \frac{q}{\Delta H} \qquad \textbf{(Equation 8-6)}$$

where:

q = Applied pressure.

ΔH = Settlement.

Several values of strain shall be used in the influence depth of approximately four times the largest dimension of the base.

It shall be permitted to estimate the modulus of subgrade reaction based on laboratory measured modulus of elasticity such that

$$k_s = \frac{E_s}{B(1-\nu^2)}$$ **(Equation 8-7)**

where:

ν = Poisons ratio for soil.

E_s = Modulus of Elasticity for soil.

8.4.1.3.2.4 **Modulus of subgrade reaction from load bearing.** In the absence of a more rigorous data, it shall be permitted to consider a value for the modulus of subgrade reaction equal to 120 times the allowable load bearing. The value for k_s shall be verified from in situ tests in case of sensitive and important structures.

8.4.1.3.2.5 **Time-dependent subgrade response**. Consideration shall be given to the time-dependent subgrade response to the loading conditions. An iterative procedure may be necessary to compare the mat deflections with computed soil response. Since the soil response profile is based on contact stresses which are in turn based on mat loads, flexibility, and modulus of subgrade reaction, iterations shall be made until the computed mat deflection and soil response converge within acceptable tolerance.

SECTION 8.5
SETTLEMENT

8.5.0 Settlements of combined footings and mats shall conform to the provisions of Sections 8.5.1 through 8.5.3.

8.5.1 **General**. The combinations of unfactored loads which will produce the greatest settlement or deformation of the foundation, occurring either during and immediately after the load application or at a later date, shall be selected. Loadings at various stages of construction such as dead load or related internal moments and forces, stage dead load consisting of the unfactored dead load of the structure and foundation at a particular time or stage of construction, and stage service live load consisting of the sum of all unfactored live loads at a particular stage of construction, shall be evaluated to determine the initial settlement, long-term settlement due to consolidation, and differential settlement of the foundation.

8.5.2 **Total settlements**. Total settlement of combined footings and mats shall not exceed the value set forth in Table 5.1.

8.5.3 **Differential settlement.** Differential settlements for combined footings shall not exceed the values set forth in Table 5.2. For mats the differential settlement shall be taken as three-fourths the total settlement if this is not more than 50 mm or determined based on relative stiffness, k_r, as shown in Table 8.3.

SECTION 8.6
COMBINED FOOTINGS

8.6.0 Combined footings shall be designed and constructed in accordance with Sections 8.6.1 through 8.6.3.

8.6.1 **Rectangular-shaped footings**. The length and width of rectangular-shaped footings shall be established such that the maximum contact pressure at no place exceeds the allowable soil pressure as obtained from Chapter 4. All moments shall be calculated about the centeroid of the footing area and the bottom of the footing. All footing dimension shall be computed on the assumption that the footing acts as a rigid body.

TABLE 8.3
MAXIMUM ALLOWABLE DIFFERENTIAL SETTLEMENTS OF MATS

k_r	SHAPE	DIFFERENTIAL SETTLEMENT (mm)
0	Rectangular base	$0.5 \times \Delta H$[a]
	Square base	$0.35 \times \Delta H$
0.5		$0.1 \times \Delta H$
> 0.5	Rigid mat: no differential settlement	

a. ΔH = Total settlement estimated based on approved methods of analysis but shall not exceed values in Table 5.1.

When the resultant of the columns load, including consideration of the moments from lateral forces, coincides with centeroid of the footing area, it shall be permitted to assume that the contact pressure is uniform over the entire area of the footing. The resultant of the load of the two columns shall not fall outside the middle third of the footing. In case where this provision cannot be fulfilled the contact pressure may be assumed to follow a linear distribution such that it varies from a maximum at the pressed edge to a minimum either beneath the footing or at the opposite edge to zero at a distance that is equal three times the distance between the point of action of the resultant of loads and the pressed edge..

Consideration shall be given to horizontal forces that can generate vertical components to the foundation due, but need not be limited to, wind, earth pressure, and unbalanced hydrostatic pressure. A careful examination of the free body must be made with the geotechnical engineer to fully define the force systems acting on the foundation before the structural analyses are attempted.

8.6.2 **Trapezoidal or irregularly shaped footings.** For reducing eccentric loading conditions, it shall be permitted to design a trapezoidal or irregularly shaped footing with the footing considered to act as a rigid body and the contact pressure determined in accordance with Section 8.4.

8.6.3 **Strap footings.** The strap shall be rigid enough to avoid rotation of the exterior footing and the footings shall be proportioned for approximately equal soil pressure. A large difference in footing width shall be avoided to reduce differential settlement. It shall be permitted to consider the strap to be rigid if it has a moment of inertia that is not less than four times that of the footing to which it is attached. The width of the strap shall be equal to the smallest column width.

Shear reinforcement in the strap shall not be used to increase rigidity. If the depth of footing is restricted, the depth of the strap may be increased to obtain the necessary rigidity. The strap shall be out of contact with soil. The strap shall be securely attached to the column and footing by dowels so that the system acts as a unit. The footings shall be proportioned so that the least lateral dimensions are within 300 to 600 mm of each other and the soil pressures are approximately equal.

8.6.4 **Overturning calculations.** In analyzing overturning of the footing, the combination of unfactored loading that produces the greatest ratio of overturning moment to the corresponding vertical load shall be used. Where the eccentricity is inside the footing edge, the factor of safety against overturning shall be taken as the ratio of resisting moment to the maximum overturning moment. The maximum overturning moment and the resisting moment caused by the minimum dead weight of the structure; both shall be calculated about the pressed edge of the footing. The factor of safety shall not be less than 1.5.

If overturning is considered to occur by yielding of the subsoil inside and along the pressed edge of the footing, the factor of safety against overturning shall be calculated from

$$FS = \frac{R_{v\,\min}(c - v)}{M_0} \qquad \textbf{(Equation 8-8)}$$

where:

FS = Factor of safety.

c = Distance from resultant of vertical forces to overturning edge.

v = Distance from the pressed edge to $R_{v\,\min}$.

M_0 = Overturning moment.

$R_{v\,\min}$ = Least resultant of all forces acting perpendicular to base area under any condition of loading simultaneous with the overturning moment.

Both cases of rectangular and triangular distribution of the soil pressure along the pressed edge of the footing shall be considered and the value for FS shall not be less than 1.5.

SECTION 8.7
CONTINUOUS FOOTINGS

8.7.0 Continuous foundations shall be designed and constructed in accordance with Sections 8.7.1 through 8.7.3.

8.7.1 **Design for rigid structures.** Continuous strip footings supporting structures which, because of their stiffness, will not allow the individual columns to settle differentially may be designed using the rigid body assumption with a linear distribution of soil pressure determined based on statics.

8.7.1.1 **Rigidity check based on relative stiffness.** If the analysis of the relative stiffness of the footing yields a value greater than 0.5 the footing can be considered rigid and the variation of soil pressure shall be determined on the basis of simple statics. If the relative stiffness factor is found to be equal or less than 0.5, the footing shall be designed as a flexible member using the foundation modulus approach as described under Section 8.7.2. The relative stiffness shall be determined as

$$k_r = \frac{EI_B}{E_s B^3} \qquad \textbf{(Equation 8-9)}$$

where

k_r = Relative stiffness.

E = Modulus of elasticity of the material used in the superstructure.

E_s = Modulus of elasticity of soil.

B = Base width of foundation perpendicular to direction of interest.

I_B = Moment of inertia per one unit width of the superstructure.

An approximate value for the flexural rigidity of structure and footing, EI_B, for one unit width of the structure can be obtained by adding the flexural rigidity for footing, E_fI_f, flexural rigidity for each member in the superstructure, EI_b, and flexural rigidity for shear walls, $Eah^3/12$ as follows

$$EI_B = E_f I_f + \sum EI_{bi} + \sum \frac{Eah^3}{12} \qquad \textbf{(Equation 8-10)}$$

where

h = Wall height.

a = Wall thickness.

E_f = Modulus of elasticity for footing.

I_{bi} = Moment of inertia for any member making up the frame resistance perpendicular to B.

I_f = Moment of inertia per one unit width of the foundation.

8.7.1.2 **Rigidity check based on column spacing**. If the average of two adjacent spans in a continuous strip having adjacent loads and column spacings that vary by not more than 20 percent of the greater value, and is less than $1.75/\lambda$, the footing can be considered rigid and the variation of soil pressure shall be determined on the basis of simple statics. The characteristic coefficient λ is given by

$$\lambda = \sqrt[4]{\frac{k_s b}{4E_c I}} \qquad \textbf{(Equation 8-11)}$$

where:

b = Width of continuous footing or a strip of mat between centers of adjacent bays.

E_c = Modulus of elasticity of concrete.

I = Moment of inertia of the strip of width b.

k_s = Modulus of subgrade reaction of soil.

If the average length of two adjacent spans as limited above is greater than $1.75/\lambda$, the beam-on-elastic foundation method noted in Section 8.7.2 shall be used. For general cases falling outside the limitations given above, the critical spacing at which the subgrade modulus theory becomes effective shall be determined individually.

8.7.2 **Design for flexible footings.** A flexible continuous footing (either isolated or taken from a mat) shall be analyzed as a beam-on-elastic foundation. Thickness shall be established on the basis of allowable wide beam or punching shear without use of shear reinforcement.

The evaluation of moments and shears can be simplified from the procedure involved in the classical theory of a beam supported by subgrade reactions, if the footing meets the following basic requirements

1. The minimum number of bays is three.

2. The variation in adjacent column loads is not greater than 20 percent.

3. The variation in adjacent spans is not greater than 20 percent.
4. The average length of adjacent spans is between the limits $1.75/\lambda$ and $3.50/\lambda$.

If these limitations are met, the contact pressures can be assumed to vary linearly, with the maximum value under the columns and a minimum value at the center of each bay.

SECTION 8.8
GRID FOUNDATIONS

8.8.1 Grid foundations shall be designed and constructed in accordance with provisions of Sections 8.7. Grid foundations shall be analyzed as independent strips using column loads proportioned in direct ratio to the stiffness of the strips acting in each direction.

SECTION 8.9
MAT FOUNDATIONS

8.9.1 **General**. Mats shall be designed and constructed in accordance with Sections 8.9.1 through 8.9.3. Mats may be designed and analyzed as either rigid bodies or as flexible plates supported by an elastic foundation (the soil). In the analysis and design of mats a number of factors shall be considered that include, but need not be limited to, the following:

1. Reliability of proposed value for the modulus of subgrade reaction obtained in accordance with Section 8.4.1.3.2.
2. Finite soil-strata thickness and variations in soil properties both horizontally and vertically.
3. Mat shape.
4. Variety of superstructure loads and assumptions in their development.
5. Effect of superstructure stiffness on mat and vice versa.

The design and construction of mats shall be under the direct supervision of a registered design professional having sufficient knowledge and experience in foundation slab engineering, who shall certify to the building official that the mats as constructed satisfy the design criteria.

8.9.2 **Excavation heaves**. The influence of heave on subgrade response shall be determined by a geotechnical engineer. Recovery of the heave remaining after placing the mat shall be treated as either a recompression or as an elastic problem. If the problem is analyzed as a recompression problem, the subsurface response related to recompression shall be obtained by a geotechnical engineer. The subsurface response may be in the form of a recompression index or deflections computed by the geotechnical engineer based on elastic and consolidation subsurface behavior.

8.9.3 **Design.** A mat may be designed using the Strength Design Method of SBC 304. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. The mat plan shall be proportioned using unfactored loads and any overturning moments. The pressure diagram is considered linear and computed from Equation 8-2 and shall be less than allowable load bearing. Loads shall include the effect of any column moments and any overturning moment due to wind or other effects. Any moments applied to the mat from columns or overturning, etc., shall be included when computing the eccentricity.

The contact pressure shall not exceed the allowable load bearing determined from Chapter 4. The allowable soil pressure may be furnished as one or more values depending on long-term loading or including transient loads such as wind. The soil pressure furnished by the geotechnical engineer shall be factored to a pseudo "ultimate" value by multiplying the allowable pressure by the ratio of the sum of factored design loads to the sum of the unfactored design loads.

8.9.3.1 **Mat thickness** The minimum mat thickness based on punching shear at critical columns shall be computed based on column load and shear perimeter. The depth of the mat shall be found without using shear reinforcement and determined on the basis of diagonal-tension shear as noted in SBC 304 Chapter 15. Investigation of a two-sided (corner column) or three-sided diagonal tension shear perimeter shall be made for columns adjacent to mat edge. An investigation for wide-beam or diagonal tension shall be made for perimeter load-bearing walls.

8.9.3.2 **Rigid design.** It shall be permitted to design mats as rigid body with linear distribution for contact pressure if the mat, superstructure, or both is rigid enough not to allow differential settlement for columns. The reinforcing steel for bending is designed by treating the mat as a rigid body and considering strips both ways, if the following criteria are met:

1. Column spacing is less than $1.75/\lambda$ or the mat is very thick.

2. Relative stiffness k_r as noted in Equation 8-9 is greater than 0.5.

3. Variation in column loads and spacing is not over 20 percent

These strips are analyzed as combined footings with multiple columns loaded with the soil pressure on the strip, and column reactions equal to the factored (or unfactored) loads obtained from the superstructure analysis. Consideration shall be given to the shear transfer between strips to satisfy a vertical load summation.

8.9.3.3 **Flexible design**. For mats not meeting the criteria of Section 8.9.3.2, it shall be designed as a flexible plate in accordance with Sections 8.9.3.3.1 and 8.9.3.3.2.

8.9.3.3.1 **Uniform loads and spacings.** If variation in adjacent column loads and in adjacent spans is not greater than 20 percent it shall be permitted to analyze mats as continuous footings that can be analyzed according to the provisions of Section 8.7.2. The mat shall be divided into strips the width of each is equal to the distance between adjacent bays. Each strip shall be analyzed independently considering column loads in both directions. The contact pressure is equal to the average contact pressure evaluated for each strip in each direction.

8.9.3.3.2 **Nonuniform loads and spacings.** If columns have irregular spacings or loads, mats may be analyzed based on theory of modulus subgrade reaction, elastic, plate method, finite difference method, finite grid method, finite element method, or any other method approved by the building official.

8.9.4 **Circular mats or plates.** For tall structure, differential settlements shall be carefully controlled to avoid toppling when the line of action of gravity forces falls out of the base. The plate depth shall be designed for wide-beam or diagonal-tension shear as appropriate.

8.9.5 **Ring foundations**. For ring foundations used for water-tower structures, transmission towers, television antennas, and various other possible superstructures, analysis and designed shall be carried out using advanced method of analysis and carried out by a registered design profession knowledgeable in geotechnical and structural engineering.

SECTION 8.10
SEISMIC REQUIREMENTS

8.10.1 For combined footings and mats of structures assigned to Seismic Design Category C or D, provisions of SBC 301 and SBC 304 shall apply when not in conflict with the provisions of Chapter 5. Strips between adjacent columns shall be capable of carrying, in tension or compression, a force equal to the product of the larger column load times the seismic coefficient S_{DS} divided by 10 unless it is demonstrated that equivalent restraint is provided by the strips.

CHAPTER 9
DESIGN FOR EXPANSIVE SOILS

SECTION 9.1
GENERAL

9.1.0 **Scope.** Provisions of this chapter shall apply to building foundation systems in expansive soil areas. Foundation design and construction shall be based on a site investigation as defined in Chapter 2, unless the building official ascertains that sufficient data upon which to base the design and construction of the foundation system is available.

9.1.1 **Approval of special systems of design or construction.** Sponsors of any system of design or construction within the scope of Chapter 9, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by Chapter 9, shall have the right to present that data on which their design is based to the building official or to a board of examiners appointed by the building official. This board shall be composed of competent geotechnical and structural engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of Chapter 9. These rules when approved by the building official and promulgated shall be of the same force as the provisions of Chapter 9.

SECTION 9.2
LOADINGS

9.2.1 Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 2.4 SBC 301. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in SBC 301 Section 4.8, are permitted to be used in designing footings. Strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

SECTION 9.3
DESIGN

9.3.0 **SCOPE.** Design for expansive soils shall be in accordance with the provisions of Sections 9.3.1 through 9.3.5. Provisions of Chapters 5 and 8 not specifically excluded and not in conflict with the provisions of Chapter 9 shall apply, where applicable.

9.3.1 **General requirements.** Footings or foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Deflection and racking of the supported structure shall be limited to that which will not interfere with the usability and serviceability of the structure. Foundations placed below where volume change occurs or below expansive soil shall comply with the following provisions:

1. Foundations extending into or penetrating expansive soils shall be designed to prevent uplift of the supported structure.

2. Foundations penetrating expansive soils shall be designed to resist forces exerted on the foundation due to soil volume changes or shall be isolated from the expansive soil.

Soil investigation report shall indicate the value or range of heave that might take place for the subject structure. Potential soil movement shall be determined based on the estimated depth of the active zone in combination with either of the following:

1. ASTM-D 4546.

2. Any other method which can be documented and defended as a good engineering practice in accordance with the principles of unsaturated soil mechanics carried out by a Geotechnical Engineer and approved by the building official.

9.3.2 **Foundations.** Footings or foundations for buildings and structures founded on expansive soil areas shall be designed in accordance with Sections 9.3.2.1, 9.3.2.2, or 9.3.2.3. Alternate foundation designs shall be permitted subject to the provisions of Section 9.1.1. Footing or foundation design need not comply with Section 9.3.2.1, 9.3.2.2, or 9.3.2.3 where the soil is removed in accordance with Section 9.3.3, nor where the building official approves stabilization of the soil in accordance with Section 9.3.4, nor where the superstructure is design by a registered design professional to accommodate the potential heave.

9.3.2.1 **Shallow foundations.** Continuous or spread footings shall not be used on expansive soils unless the soil deposit has a low expansion potential, as determined in accordance with Table 9.1 or the superstructure is designed to account for the potential foundation movement. The uplift pressures on the sides of the footing shall be minimized to the extent possible.

For continuous footings, the swell pressure shall be counteracted without exceeding the bearing capacity of the soil deposit by narrowing the width of the strip footing and/or providing void spaces within the supporting beam or wall. The continuous foundation shall be stiffened by increasing the reinforcement around the perimeter and into the floor slab.

For spread footings, a void space shall be provided beneath the grade beams using the same technique as described for pier and grade beam construction in Section 9.3.2.3. The footings shall be designed using as high bearing pressure, as practicable.

TABLE 9.1
CLASSIFICATION OF EXPANSION POTENTIAL

Expansion Index (EI)[a]	Expansion Potential
0 – 20	Very low
21– 50	Low
51 – 90	Medium
91 –130	High
> 130	Very high

a. EI = {1000 × (final height of specimen – initial height of specimen)/initial height of specimen}, as per ASTM D 4829.

9.3.2.2 **Slab-on-grade foundations.** Slab-on-grade (Slab-on-ground) foundations on expansive soils shall be designed and constructed in accordance with WRI/CRSI *Design of Slab-on-Ground Foundations*.

A conventionally reinforced slab-on-grade foundation shall conform to applicable provisions of SBC 304, where applicable. All variables affecting finished-slab performance shall be considered when selecting a slab type and when specifying or executing a slab design. All slab-on-grade foundations, with the exception of conventionally reinforced slabs less than 50 m^2, shall be designed by a registered design professional having sufficient knowledge and experience in structural and foundation engineering. Design of slab shall be conducted for conditions of both center and edge heave. Construction joints shall be placed at intervals not exceeding 4.5 m.

Exception: Slab-on-grade systems that have performed adequately in soil conditions similar to those encountered at the building site are permitted subject to the approval of the building official.

9.3.2.3 **Beam-on-drilled pier.** The design provisions of Chapter 17 shall be expanded to include, but need not be limited to, the requirements of Sections 9.3.2.3.1 and 9.3.2.3.2.

9.3.2.3.1 **General requirements.**

1. A void space shall be maintained beneath the grade beam between the piers. The required void space shall be determined based on the predicted heave of the soil beneath the beam but shall not be less than 150 mm.
2. Care shall be taken in the design to provide for sealing the space between the soil and the pier, such that deep seated heave that may result from water gaining access to soils below active zone along the shaft of the pier, is prevented.
3. Sufficient field penetration resistance tests shall be performed not only to establish the proper friction value but also to ensure that soft soils are not the cause of tensile forces developed in the pier.
4. The upper 1.5 m of soil around the pier shall be excluded when calculating the pier load capacity.
5. Friction piers shall not be used at sites where groundwater table is either high or expected to become high in the future.
6. Uplift skin friction shall be permitted to be assumed constant throughout the active zone.
7. Where the upper soils are highly expansive or if there is a possibility of loss of skin friction along the lower anchorage portion of the shaft due to rise of groundwater table, the bottom of the shaft shall be belled or under-reamed. The vertical side shall be a minimum of 150 mm high and the sloping sides of the bell shall be formed at either 60° or 45°. For piers founded well below the active zone, the shaft may not be under-reamed.
8. Upward movement of the top of the pier and the tensile forces developed in the pier shall be considered in the design of drilled piers.

9. Mushrooming of the pier near the top shall be avoided. Cylindrical cardboard at/or extended above the top of the concrete shall be used to prevent formation of mushroomed piers.

9.3.2.3.2 **Reinforcement**. Reinforcing steel shall extend the entire length of the pier and shall be hooked into the belled bottom, if used, and into the grade beam at the top. The area of the steel shall be designed to resist all tensile loads to which the pier may be subjected but shall not be less than a minimum of 1 percent of the cross-sectional area of the pier.

9.3.3 **Removal of expansive soil**. Where expansive soil is removed in lieu of designing footings or foundations in accordance with Section 9.3.2.1, 9.3.2.2, or 9.3.2.3, the soil shall be removed to a depth sufficient to ensure constant moisture content in the remaining soil. Fill material shall not contain expansive soils and shall comply with Sections 3.6 and 3.10 or 3.11. If the expansive strata are not entirely removed, the fill material shall be impermeable enough not to provide access for water into expansive grades or foundation soils.

Exception: Expansive soil need not be removed to the depth of constant moisture, provided the confining pressure in the expansive soil created by the fill and supported structure exceeds the swell pressure.

9.3.4 **Stabilization.** Where the active zone of expansive soils is stabilized in lieu of designing footings or foundations in accordance with Section 9.3.2.1, 9.3.2.2, or 9.3.2.3, the soil shall be stabilized by chemical, installation of moisture barriers, pre-wetting or other techniques designed by a geotechnical engineer knowledgeable in unsaturated soil mechanics and approved by the building official. In pre-wetting technique, the effect of strength loss shall be evaluated to ensure that strength criteria are met. Limitations and implementation procedures of the contemplated stabilization technique shall receive careful consideration and thorough evaluation.

9.3.5 **Required preventive measures.** Applicable provisions of Chapter 13 shall be expanded to include, but need not be limited to, the following:

1. All water-supply pipes and wastewater pipes shall be watertight and have flexible connections and couplings.
2. All rainwater pipes shall be ducted well away from the foundations. It shall be made sure that all water from downspout is discharged away from the building into storm sewer or suitable ground surface location downhill.
3. The ground surface shall slope away from the structure. Bare or paved areas shall have a slope not less than 2 %, and if possible the ground surface within 3 meters of the structure shall be sloped at a 10 percent grade.
4. Storage tanks and septic tanks shall be reinforced to minimize cracking and have adequate flexible waterproofing as per section 13.5.
5. Plants and irrigation systems shall not be placed immediately adjacent to the structure and spray heads shall be directed away from the structure. Large trees and bushes shall be kept away from the foundations for a distance greater than half their mature height.
6. If horizontal moisture barriers are installed around the building to move edge effects away from the foundation and minimize seasonal fluctuations of water

content directly below the structure, care shall be taken to seal joints, seams, rips, or holes in the barrier. Horizontal moisture barriers may take different forms including, but not necessarily limited to, membranes, rigid paving (concrete aprons, etc.), or flexible paving (asphalt membranes, etc.).

7. If vertical moisture barriers are used around the perimeter of the building they shall be installed at least one meter from the foundation to a depth equal to or greater than the depth of seasonal moisture variation (active zone). Buried vertical barriers may consist of polyvinyl chloride, polyethylene, polymer-modified asphalt or any other approved methods or materials.

8. If the structure has a basement, the backfill shall consist of non-expansive soils and it shall comply with Sections 3.6 and 3.10 or 3.11.

SECTION 9.4
PRE-CONSTRUCTION INSPECTIONS

9.4.1 A pre-construction site inspection shall be performed to verify the following:

1. Vegetation and associated root systems have been removed from the construction site.

2. No beam trench cuttings or scarified material have been placed as fill material.

3. All fill has been placed in accordance with Sections 3.6 and 3.10 or 3.11 in any portions or sections of the foundation supporting grade.

4. Proper soil compaction of the foundation footprint and fill material has been performed to a minimum of 95 percent standard proctor density.

SECTION 9.5
INSPECTION PRIOR TO PLACEMENT OF CONCRETE

9.5.1 Prior to the placement of concrete, an inspection of the beam geometrics, penetrations, cable(s), cable(s) anchorage/steel placements and other details of the design shall be made to verify conformance with the design plans.

SECTION 9.6
CONCRETE

9.6.1 Material, construction, and placement of concrete shall be in accordance with the provisions of Section 5.4.2 and Section 8.3.

CHAPTER 10
DESIGN FOR COLLAPSIBLE SOILS

SECTION 10.1
GENERAL

10.1.0 **Scope**. Provisions of this chapter shall apply to building foundation systems on collapsible soil areas. Foundation design and construction shall be based on a site investigation as defined in Chapter 2, unless the building official ascertains that sufficient data upon which to base the design and construction of the foundation system is available.

10.1.1 **Approval of special systems of design or construction.** Sponsors of any system of design or construction within the scope of Chapter 10, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by Chapter 10, shall have the right to present that data on which their design is based to the building official or to a board of examiners appointed by the building official. This board shall be composed of competent geotechnical and structural engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of Chapter 10. These rules when approved by the building official and promulgated shall be of the same force as the provisions of Chapter 10.

SECTION 10.2
LOADINGS

10.2.1 Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 2.4 SBC 301. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in SBC 301 Section 4.8, are permitted to be used in designing footings. Strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

SECTION 10.3
DESIGN

10.3.0 Design for collapsible soils shall be in accordance with the provisions of Sections 10.3.1 through 10.3.3. Provisions of Chapters 5 and 8 not specifically excluded and not in conflict with the provisions of Chapter 10 shall apply, where applicable.

10.3.1 **Foundations.** Footings or foundations for buildings and structures founded on collapsible soil areas shall be designed in accordance with Sections 10.3.1.1 through 10.3.1.4 Alternate foundation designs shall be permitted subject to the provisions of Section 10.1.1. Footing or foundation design need not comply with Sections 10.3.1.1 and 10.3.1.2 where the soil is removed in accordance with Section 10.3.2, nor where the building official approves stabilization of the soil in accordance with Section 10.3.3, nor where the superstructure is designed by a registered design professional to accommodate the potential settlements.

10.3.1.1 **Classification of collapse potential.** Collapse potential shall be permitted to be classified in accordance with one of the methods prescribed in Sections 10.3.1.1.1, 10.3.1.1.2, or 10.3.1.1.3.

10.3.1.1.1 **Collapse index method.** The collapsibility of a particular soil under specified conditions could be determined in accordance with ASTM D5333. The specimen collapse shall be classified according to the collapse index, I_e, as set forth in Table 10.1.

10.3.1.1.2 **Standard plate load test method**. Where undisturbed soil specimens are irretrievable, collapse potential for specific field conditions could be estimated from standard plate load tests (SPLT), conducted in test pit under unsoaked and soaked conditions in accordance with ASTM D1194.

10.3.1.1.3 **BREA infiltration and plate load test method.** Collapse potential could be determined in accordance with BREA *Building Regulations in Eastern Arriyadh Sensitive Soils* procedures (BPLT). The procedures shall apply to tests performed in test pits or trenches. The infiltration field test shall be performed in accordance with the procedure set forth in Table 10.2 and the field plate load test shall be carried following the procedure outlined in Table 10.3. The stability of the reaction column and side-wall of the test pit shall be considered, particularly for test pits deeper than 4 meters.

10.3.1.1.3.1 **Design curve construction.** A design curve for the site shall be constructed in accordance with the steps outlined in Table 10.4. A data sheet in the form shown in Table 10.4 may be used for the raw data gathered during the test and for the reduced data.

10.3.1.2 **Design procedure**. Based on which method used in estimating the collapse potential of the soil deposit as provided in Sections 10.3.1.1.1, 10.3.1.1.2, or 10.3.1.1.3, design for collapsible soils shall be in accordance with Sections 10.3.1.2.1, 10.3.1.2.2, or 10.3.1.2.3, respectively.

10.3.1.2.1 **Design based on collapse index.** Potential settlement that may occur in a soil layer under the applied vertical stress is obtained as follows

$$\rho = \frac{H}{100} I_c \qquad \textbf{(Equation 10-1)}$$

where:

H = Thickness of the soil layer.

I_c = Collapse potential, determined using a predetermined applied vertical stress applied to a soil specimen taken from the soil layer as follows.

$$I_c = \frac{d_f - d_i}{h_o} \times 100 \qquad \textbf{(Equation 10-2)}$$

where:

d_i = Specimen height at the appropriate stress level before wetting.

d_f = Specimen height at the appropriate stress level after wetting.

h_o = Initial specimen height.

Based on settlement value determined by Equation 10-1, the foundation system shall be designed in accordance with the provisions of Chapters 5 and 8, where applicable.

Limitations. Amount of settlement depends on the extent of wetting front and availability of water, which can rarely be predicted prior to collapse. Prediction of settlement based on collapse potential shall be viewed and interpreted accordingly.

TABLE 10.1
CLASSIFICATION OF COLLAPSE POTENTIAL

Collapse index (I_e)[a] percent	Degree of Specimen Collapse
0	None
0.1-2.0	Slight
2.1-6.0	Moderate
6.1-10.0	Moderately severe
> 10	Severe

[a] $I_e = \frac{\Delta e}{1+e_o}100$, where Δe = change in void ratio resulting from wetting, and e_o = initial void ratio.

TABLE 10.2
BREA INFILTRATION FIELD TEST PROCEDURE

1. Excavate a trench or test pit to the desired depth of testing and provide a smooth flat surface for testing. Do not backfill to achieve smoothness.
2. At a distance no less than 3 plate diameters (3D) from the trench or test pit excavated in step 1, excavate a shallow infiltration pit to a depth of 60 to 100 mm and a diameter of 2D. This pit for the preliminary rate-of-infiltration test shall be separated by 3D from the supports of the reference beam. Measure the depth of the dry infiltration pit at the center.
3. Fill the infiltration pit with water and note the time at which wetting was commenced. Add water during infiltration as needed to keep the bottom of the pit covered.
4. After an infiltration time, t_p, of about 10 to 20 minutes, remove the excess water from the test pit, quickly excavate at the center of the pit to locate the depth of wetting, and measure down to the wetting front. The depth of wetting from the preliminary infiltration test (Z_{wp}) is equal to depth of wetting front minus the original depth of the dry pit.
5. The infiltration coefficient for the preliminary test (C_{ip}) is computed as

$C_{ip} = Z_{wp}/(t_p)^{1/2}$	Where: Z_{wp} = depth of infiltration for the preliminary infiltration test, mm. C_{ip} = infiltration coefficient for the preliminary infiltration test, mm/min$^{1/2}$. t_p = time duration of infiltration for the preliminary infiltration test, min.

TABLE 10.3
BREA PLATE LOAD TEST PROCEDURE (BPLT)

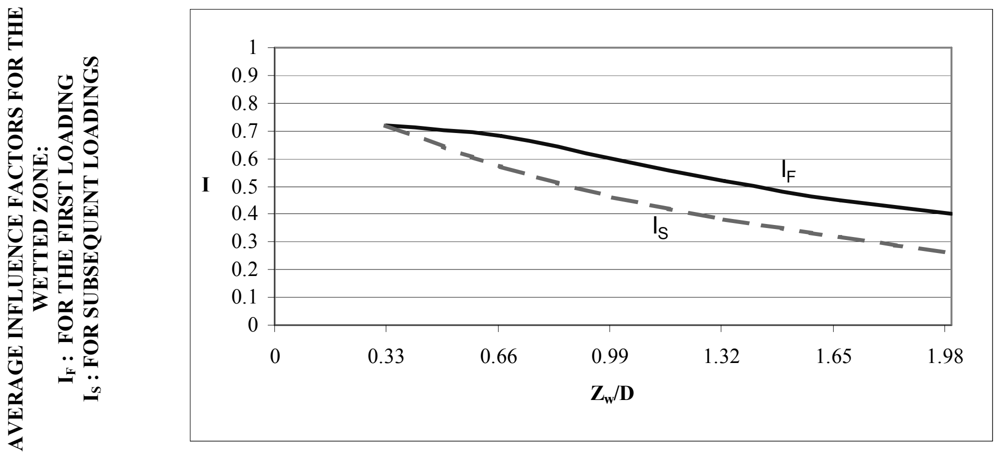
1. For the BPLT, choose the target depth of infiltration (Z_{tar}), equal 0.5D.
2. Compute target time of infiltration (t_{tar}) from: $t_{tar} = (Z_{tar}/C_{ip})^2$
3. Place the loading plate on a smooth flat surface and twist and tap lightly. The bottom of the loading plate may be coated with 5-10 mm of quick setting epoxy before placing it on the soil.
4. Construct a beam to hold water in preparation for ponding. The outside diameter of the ponded water shall be about 2D.
5. Install the reference beam to rest on firm supports located at least 3D from center of loading plate.
6. Attach displacement gauges so that they touch the loading plate on opposite sides and approximately equidistant from the center of the plate.
7. Install the loading jack and reaction column.
8. Apply a seating load of 3 to 8 kPa and zero the displacement gauges. It may be convenient to use the weight of the loading jack and plate as the seating load.
9. Increase load to 15 kPa. Wait one minute and take displacement readings.
10. Commence wetting and note starting time. Maintain water level 10 to 20 mm above the top of the plate.
11. Continue wetting until t_{tar} (computed from step 2) has elapsed. Read displacement gauges, note time and increase load to 40 kPa.
12. Wait Δ t minutes, read displacement gauges, note time, increase load to 100 kPa. The time increment Δ t may

be chosen as the larger of 2 minutes or $0.1t_{tar}$. For convenience, Δt may be rounded to the nearest minute.

13. Wait Δt minutes, read displacement gauges, note time, increase load to 200 kPa.
14. Wait Δt minutes, read displacement gauges, note time, increase load to 400 kPa.
15. Wait Δt minutes, read displacement gauges, note time, remove load from the plate, remove the plate and quickly excavate to determine the final depth (Z_{wfinal}) and note the corresponding time t_{final}.

TABLE 10.4
DATA REDUCTION FOR DESIGN CURVE CONSTRUCTION

1. In column-1, description of the test stage. This will give meaning to column-2 (time column).
2. In column-2, time shall be recorded.
3. In column-3, elapsed time (t_w) shall be computed. Elapsed time is set equal to zero when ponding is commenced.
4. In column-4, pressure reading on the jack shall be recorded. A load cell could be substituted for the pressure gauge on the jack.
5. In column-5, the added load on the plate shall be recorded.
6. In column-6, the total load on the plate shall be recorded. This is obtained by adding column-5 to the weight (in kN) of the jack and the loading plate.
7. In column-7, the left and right displacement gauges readings shall be recorded.
8. In column-8, the displacement, ΔH, for the left and right gauges shall be recorded. They are obtained by subtracting the initial gauge readings at seating load from each subsequent reading.
9. In column-9, the average displacement (ΔH_{ave}) is obtained by averaging the ΔH values from the left and right reading in column-8.
10. In column-10, the depth of wetting (Z_w) shall be computed by first determining C_{itest} from: $C_{itest} = (Z_{wfinal})/(t_{final})^{1/2}$ and then use it with t_w to get Z_w from: $Z_w = (C_{itest})(t_w)$ ½ .
11. In column-11, Z_w/D shall be computed (column-10 divided by plate diameter).
12. In column-12, the influence factors I_F and I_S obtained from the below figure at Z_w/D shall be recorded.
13. In column-13, the contact pressure (q_{con}) shall be calculated by dividing column-6 by the plate area.
14. In column-14, the average stress within the wetted zone (q_{ave}) shall be calculated by multiplying column-13 (q_{con}) by column-12 (I_F for first loading or I_S for subsequent loadings).
15. In column-15, the average strain (ε_{ave}) shall be computed by dividing column-9 (ΔH_{ave}) by column-10 (Z_w) and multiplying the result by 100.
16. Plot q_{ave} versus ε_{ave} for different tests on the same diagram. Obtain the average of all tests and sketch a DESIGN CURVE.



Data sheet and computations

Date: **Job No.:** **Test Location:** **Test Pit No.:**	**Depth of Test Pit:** **Ground Surface Elevation:** **Z_{wp}:** **t_p:**	**C_{ip}:** **Z_{tar}:** **t_{tar}:** **Δ t:**

(1)	(2)	(3)	(4)	(5)	(6)	(7)		(8)		(9)	(10)	(11)	(12)	(13)	(14)	(15)
Stage	Time	Elapsed time, t_w	Pressure reading on Jack	Added load	Total load on plate	Dial gauge reading (mm)		Displacement (mm)		ΔH_{ave}	Z_w	Z_w/D	I_F or I_S	q_{con}	q_{ave}	ε_{ave}
	(min.)	(min.)		(kN)	(kN)	Left	Right	Left	Right	(mm)	(mm)			(kPa)	(kPa)	(percent)
Seating load applied																
Dry loading																
Wetting commenced																
etc.																

10.3.1.2.2 **Design based on SPLT**. From the load-deformation curve obtained from standard plate load test under soaked condition in accordance with Section 10.3.1.1.2, the allowable load bearing is taken equal to the value corresponding to settlement of test plate determined from Equation 10-3 as follows

$$\delta_P = \frac{1}{4}\left(1+\frac{B_0}{B_p}\right)^2 \delta_D \qquad \textbf{(Equation 10-3)}$$

where:

δ_p = Settlement of test plate.

δ_D = Design settlement of prototype foundation taken to be equal to half the allowable settlement value given from Section 5.4.1.5.

B_o = Width of prototype footing.

B_p = Width of test plate.

Based on the obtained allowable load bearing, the foundation system shall be designed in accordance with the provisions of Chapters 5 and 8, where applicable.

Limitations. In determining the bearing pressure for the specified tolerable differential settlement, the validity and accuracy of any resulting computations shall be approved by the building official and evaluated on the basis of the following variables:

1. Dependence of the amount of settlement on the extent of the wetting front and availability of water, which can rarely be predicted prior to collapse.
2. The influence depth set to be four times the footing width is significantly different for the model versus prototype footing.
3. Increase soil stiffness due to increased confinement with depth.

10.3.1.2.3 **Design based on BPLT.** Design of spread and strip footings shall conform to the provisions of Section 10.3.1.2.3.1 and mats shall be designed in accordance with the provisions of Section 10.3.1.2.3.2.

10.3.1.2.3.1 **Spread and continuous footings.** Spread and continuous footings are permitted to be used without modifications in areas with low collapse potential, as determined in accordance with Table 10.5. In areas with higher collapse potential, strip footings are permitted, provided that the requirements for additional distortion resistance specified in Table 10.8 are met.

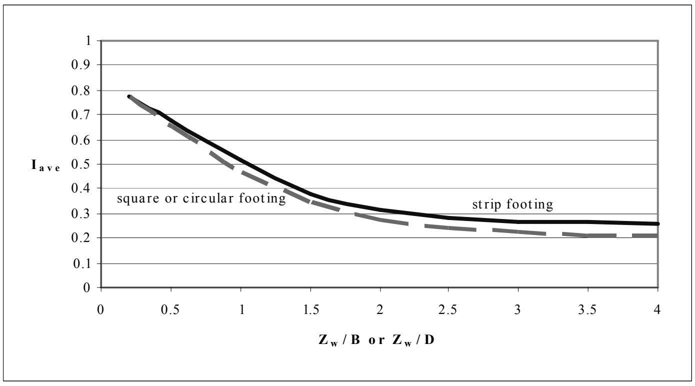
10.3.1.2.3.2 **Stiffened mat foundations.** The design procedure for mat foundations in collapsible soils is summarized in Table 10.9. The mat shall be designed and constructed in accordance with the provisions of Chapter 8, where applicable, and the requirements for additional distortion resistance specified in Table 10.8 shall be met.

10.3.2 **Removal of collapsible soil**. Where collapsible soil is removed in lieu of designing footings or foundations in accordance with Section 10.3.1, the soil shall be removed to a sufficient depth to ensure constant moisture content in the remaining soil. Fill material shall not contain collapsible soils and shall comply with the provisions of Sections 3.6 and 3.10 or 3.11.

TABLE 10.5
DESIGN OF SPREAD AND STRIP FOOTINGS ON COLLAPSIBLE SOILS

1. From Table 10.6, use strain under the plate corresponding to stress of 100 kPa (ε_{100}) to classify the site with respect to collapse potential.
2. Pre-wetting is required for very high collapse potential, and permitted but not required for high collapse potential.
3. When pre-wetting is chosen or required, the DESIGN CURVE constructed in Table 10.4 is replaced with a recompression design curve whose strain values are everywhere 15 percent of those on the original design curve, and the site is reclassified accordingly.
4. Use Tables 10.6 and 10.7 to obtain required design parameters D_f (foundation depth), D_{wdes} (depth of wetted bulb), foundation type, required distortion resistance, ΔH_{diff} /L and $\Delta H_{diff} / \Delta H_{tot}$.
5. From Table 10.7 use column spacing, L, to compute ΔH_{diff} , then compute ΔH_{tot} from the same table.
6. Compute $Z_w = D_{wdes} - D_f$.
7. Compute maximum allowable $\varepsilon_{ave} = \Delta H_{tot} / Z_w$.
8. Compute the overburden pressure (P_{odes}) at D_{wdes}.
9. From the DESIGN CURVE get allowable ε_{ave} corresponding to P_{odes}.
10. If $\varepsilon_{ave} \geq$ allowable ε_{ave} from step-7, change D_f, foundation type or stiffness level and recalculate.
11. If $\varepsilon_{ave} <$ allowable ε_{ave} , use allowable ε_{ave} in the DESIGN CURVE to get allowable P_{tot}, then find q_{ave} as follows: $q_{ave} =$ allowable $P_{tot} - P_{odes}$
12. Assume a first trial value of B (footing width). Compute Z_w/B, find I_{ave} from the Figure below and compute q_{all} as follows: q_{all} = allowable q_{ave}/I_{ave}
13. Use the trial value of B, footing shape and column load to compute q_{con}. If $q_{con} \approx q_{all}$, then B value is accepted, otherwise change B and iterate until convergence, then proceed with structural design.
14. In case of no convergence or if B or q_{all} are not acceptable, increase D_f, change footing type, change distortion stiffness or a combination of those.

INFLUENCE FACTOR FOR AVERAGE STRESS WITHIN WETTED ZONE VS. DEPTH OF WETTED ZONE



10.3.3 **Stabilization**. Where collapsible soils are stabilized in lieu of designing footings or foundations in accordance with Section 10.3.1, the soil shall be stabilized by compaction, pre-wetting, vibroflotation, chemical, or other techniques designed by a geotechnical engineer knowledgeable in unsaturated soil mechanics and approved by the building official. The provisions of Section 9.3.5 shall also be considered, where applicable. In pre-wetting technique, the effect of strength loss shall be evaluated to ensure that strength criteria are met. Great care must be exercised when using pre-wetting near existing structures that underlain by collapsible soils, particularly if the soil has strong stratification, as in the case of many alluvial soils, and injected water may flow horizontally more than it does vertically. Limitations and implementation procedures of the contemplated stabilization technique shall receive careful consideration and thorough evaluation.

TABLE 10.6
MINIMUM DESIGN PARAMETERS AS A FUNCTION OF COLLAPSE POTENTIAL

ε_{100} (percent)	Collapse potential	Minimum D_f (m)	Minimum design depth of wetting D_{wdes} (m)	Required distortion stiffness	Allowable types of foundation
0 - 0.5	Low	1.0	3.5	Level 0	Spread, Strip, Mat
0.5 - 1.5	Moderate	1.5	3.5	Level I	Strip, Mat
1.5 - 5.0	High	2.0	3.5	Level II	Strip, Mat
>5.0	Very high	2.5	3.5	Level II	Strip, Mat

TABLE 10.7
REQUIRED MINIMUM RATIOS OF DIFFERENTIAL TO TOTAL SETTLEMENT AS A FUNCTION OF FOUNDATION TYPE AND DISTORTION STIFFNESS

Type of foundation	Min. Design $\Delta H_{diff}/L$	$\Delta H_{diff} / \Delta H_{tot}$ as a function of Required Extra Distortion Resistance		
		Level 0	Level I	Level II
Spread	1/500	0.85	0.75	0.65
Strip	1/500	0.65	0.55	0.45
Mat	1/500	0.35	0.30	0.25

TABLE 10.8
REQUIREMENTS FOR EXTRA DISTORTION RESISTANCE

Distortion Stiffness	Type of Foundation	Extra Concrete in Grade Beam	Extra Concrete in Footing	Extra Steel in Footing	Extra Steel in Wall and Floor	Extra Steel in Foundation Column
Level 0	Refers to standard design and requires no extra distortion resistance					
Level I	Spread Footing	10percent higher	10percent thicker	2 bars	–	3 bars
	Strip Footing	10percent higher	10percent thicker	2 bars	–	3 bars
	Mat	–	–	–	15percent, see Figure 10.1	–
Level II	Spread Footing	20percent higher	20percent thicker	3 bars	–	6 bars
	Strip Footing	20percent higher	20percent thicker	3 bars	–	6 bars
	Mat	–	–	–	25percent, see Figure 10.1	–

SECTION 10.4
INSPECTIONS

10.4.1 A pre-construction site inspection shall be conducted to verify that the provisions of Section 9.4 have been met.

SECTION 10.5
CONCRETE

10.5.1 Material, construction, and placement of concrete shall be in accordance with the provisions of Section 5.4.2 and Section 8.3. Prior to the placement of concrete, an inspection of the beam geometrics, reinforcements and other details of the design shall be made to verify conformance with the design plans.

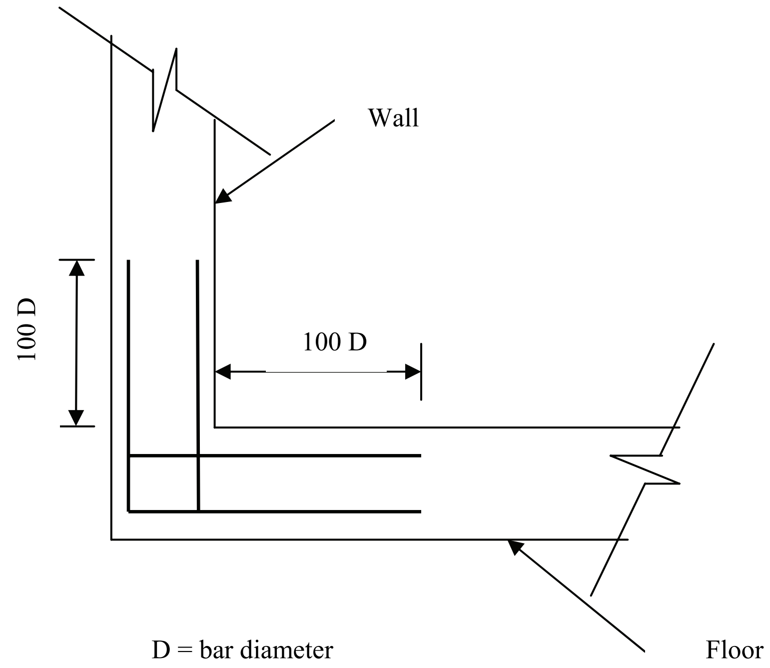


FIGURE 10.1
EXTRA STEEL IN WALL AND FLOOR FOR MAT FOUNDATION

TABLE 10.9
DESIGN OF MAT FOUNDATION ON COLLAPSIBLE SOILS

1. From Table 10.6, use strain under the plate corresponding to stress of 100 kPa (ε_{100}) to classify the site with respect to collapse potential.
2. Pre-wetting is required for very high collapse potential, and permitted but not required for high collapse potential.
3. When pre-wetting is chosen or required, the DESIGN CURVE is replaced with a recompression design curve whose strain values are everywhere 15percent of those on the original design curve, and the site is reclassified accordingly.
4. Use Tables 10.6 and 10.7 to obtain required design parameters D_f (foundation depth), D_{wdes} (depth of wetted bulb), foundation type, required distortion resistance, ΔH_{diff} /L and $\Delta H_{diff} / \Delta H_{tot}$.
5. Compute ΔH_{tot} .
6. Compute $Z_w = D_{wdes} - D_f$.
7. Compute maximum allowable $\varepsilon_{ave} = \Delta H_{tot} / Z_w$.
8. Compute the overburden pressure (P_{odes}) at 1/3 of the way from the base of the foundation to D_{wdes}. For mat foundations, only a fraction of the P_{odes} acts on the soil during wetting as seen from Table below.
9. The average contact stress under a mat, q_{con}, is governed by the weight of the structure including the mat and the footprint of the structure. Only D_f and distortion stiffness can be changed in pursuit of an acceptable design.
10. Compute q_{con}, Z_w/B and estimate I_{ave} from Table 10.5, using the curve for square footing or interpolate between the curves as a function of the shape of structure in plan (length/width $\geq$ 4 can be interpreted as strip).
11. Compute $q_{ave} = I_{ave} \times q_{con}$
12. Compute $P_{odes} = a \times P_o$, where the factor 'a' represent percentage of overburden stress acting on the wetted soil under mats and is obtained as

Z_w/B	factor a
0 - 0.1	0.1
0.1 - 0.3	0.3
0.3 - 0.6	0.5

13. Compute $P_{tot} = q_{ave} + P_{odes}$
14. Enter the DESIGN CURVE to get ε_{ave} and compute $\Delta H_{tot} = \varepsilon_{ave} \times Z_w$
15. If ΔH_{tot} < allowable ΔH_{tot} from above, the design is acceptable. Proceed with structural design.
16. If ΔH_{tot} > allowable ΔH_{tot} from above, increase D_f and/or increase distortion stiffness and recalculate ΔH_{tot} .

Note: $P_{odes} = aP_o$ = That portion of the overburden stress assumed to produce strain upon wetting.

CHAPTER 11
DESIGN FOR SABKHA SOILS

SECTION 11.1
GENERAL

11.1.0 **SCOPE**. Provisions of this chapter shall apply to building foundation systems in sabkha soil areas. Foundation design and construction shall be based on a site investigation as defined in Chapter 2, unless the building official ascertains that sufficient data upon which to base the design and construction of the foundation system is available.

11.1.1 **Approval of special systems of design or construction.** Sponsors of any system of design or construction within the scope of Chapter 11, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by Chapter 11, shall have the right to present that data on which their design is based to the building official or to a board of examiners appointed by the building official. This board shall be composed of competent geotechnical and structural engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of Chapter 11. These rules when approved by the building official and promulgated shall be of the same force as the provisions of Chapter 11.

SECTION 11.2
LOADINGS

11.2.1 Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in SBC 301 Section 2.4. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in SBC 301 Section 4.8, are permitted to be used in designing footings. Strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

SECTION 11.3
DESIGN

11.3.0 Design for sabkha soils shall conform to the provisions of Sections 11.3.1 through 11.3.2. Provisions of Chapters 5 and 8 not specifically excluded and not in conflict with the provisions of Chapter 11 shall apply, where applicable.

11.3.1 **General requirements.** Footings or foundations placed on or within sabkha soils shall be designed to prevent structural damage to the supported structure due to detrimental settlement. Deflection and racking of the supported structure shall be limited to that which will not interfere with the usability and serviceability of the structure. Design shall consider, but need not limited to, the following:

1. The decrease in strength of the surface crust of the sabkha as a result of moisture content increase. This crust shall not be used as a foundation layer.
2. The variation of compressibility characteristics of the site resulting from differences in layer thickness, degree of cementation, and relative density of different locations within the site.

3. Differential settlements and foundation instabilities due to volume changes that accompany hydration and dehydration of gypsum rich layers under the hot and humid conditions.

4. High concentrations of chlorides and sulphates in the sabkha sediments and brines, and the subsequent highly corrosive to both concrete and steel.

Soil investigation report shall indicate the value or range of settlement that might take place for the subject structure. Potential settlements shall be estimated by a method of analysis that can be documented and defended as a good engineering practice and approved by the building official. Allowable settlements shall conform to the requirements of Sections 5.4.1.5 and 8.5, where applicable.

11.3.2 **Foundations.** For heavy structures**,** mat or deep foundations shall be considered, and provisions of Chapter 8 and Chapters 14 through 17 shall govern, where applicable.

For lightly loaded buildings and structures founded on sabkha soil areas, and provided that water table is always kept beneath the foundation level, it shall be permitted to design and construct footings or foundations in accordance with Sections 11.3.2.1 through 11.3.2.3, subject to the approval of building official, and under a direct supervision of a geotechnical engineer knowledgeable in sabkha soils.

Alternate foundation designs shall be permitted subject to the provisions of Section 11.1.1. Footing or foundation design need not comply with Section 11.3.1 and 11.3.2 where the soil is removed in accordance with Section 11.6, nor where the building official approves stabilization of the soil in accordance with Section 11.7, nor where the superstructure is design by a registered design professional to accommodate the potential settlements.

11.3.2.1 **Water table below 5 meters depth**

Where groundwater is 5 meters below the ground surface level, external and internal walls have to be supported by a concrete strip foundation. Water infiltration shall be prevented under the floor slabs by installing heavy duty polythene sheeting, or other approved materials, as shown in Figure 11.1(a). Joints in the polythene sheeting shall be lapped and sealed in accordance with the manufacturer's installation instructions. Strip foundation shall be supported by lean mix concrete to prevent contamination of the wet concrete when poured.

11.3.2.2 **Water table between 2.5 and 5 meters depth**

Where groundwater is between 2.5 meters and 5 meters below the ground surface level, provisions of Section 11.3.2.1 shall be satisfied. Slab floors have to be supported by a strip foundation as illustrated in Figure 11.1(b). Coarse, durable gravels shall be placed beneath the floor slab and around the strip foundation.

11.3.2.3 **Water table between ground level and 2.5 meters depth**

Where groundwater is between ground level and 2.5 meters, the provisions of Section 11.3.2.2 shall be fulfilled. Further, the strip foundation and the floor slab shall also be underlain by a rolled coarse gravel capillary cut-off, not less than 150 mm thick, resting on a compacted fill blanket as illustrated in Figure 11.1(c).

SECTION 11.4
REQUIRED PREVENTIVE MEASURES

11.4.1 The applicable provisions of Section 9.3.5 and Chapter 13 shall be expanded to include, but need not be limited to, the following:

1. Domestic and irrigation water shall be strictly controlled, especially where low density sands cemented with sodium chloride. Protection by drainage around major structures shall be considered to reduce the risks associated with rainstorms or burst water mains.

2. There shall be external protection against corrosion for all pipelines, fittings and valves, whether steel, ductile iron, or asbestos-cement. Ductile iron pipe work shall be factory coated with a bituminous coating compatible with a specified pipe wrapping material. Steel pipe work shall be factory coated with either a thermosetting, fusion bonded, dry powder epoxy coating not less than 300 micrometers thick or a catalyst-cured epoxy coating applied in three coats, to a total cured dry film thickness of 240 micrometers. Ductile iron, steel and asbestos-cement pipe work shall then be wrapped with heavy duty self-adhesive, rubber bitumen compound with PVC carrier strip. The pipe work shall be sleeved with 0.2 mm thick polyethylene sleeving.

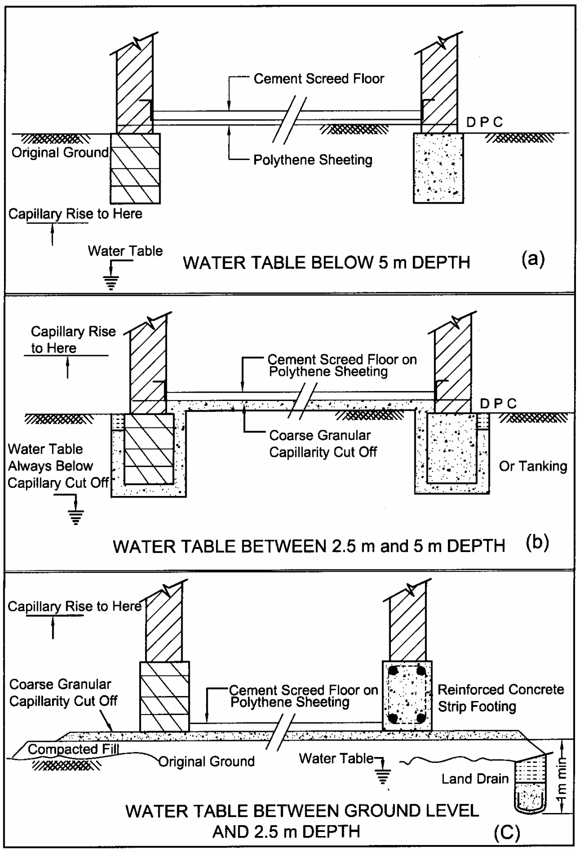


FIGURE 11.1
SHALLOW FOUNDATION DESIGN STRATEGIES
(a) WATER TABLE BELOW 5 m DEPTH;
(b) WATER TABLE BETWEEN 2.5 m AND 5 m DEPTH;
(c) WATER TABLE BETWEEN GROUND LEVEL AND 2.5 m DEPTH.

SECTION 11.5
CONCRETE

11.5.0 Material, construction, and placement of concrete shall be in accordance with the provisions of Section 5.4.2 and Section 8.3, where applicable.

11.5.1 **Concrete protection**. Concrete shall satisfy the durability criteria of SBC 304 Chapter 4. Protection against salt attack on foundation materials, buried pipes, and metal objects shall be provided by using sulphate resistance cement. Concrete used in the construction of foundation on sabkha formations shall be made from Type V Portland cement, with minimum cement content of 370 kg/m^3, and maximum water cement ratio of 0.4 for corrosion protection and 0.45 for sulfate protection. Reinforcement type shall be epoxy coated and a minimum cover to reinforcement of 75 mm shall be stringently enforced.

SECTION 11.6
REMOVAL OF SABKHA SOILS

Where sabkha soil is removed in lieu of designing footings or foundations in accordance with Section 11.3.2, the soil shall be removed to a depth sufficient to ensure adequate load-bearing and tolerable settlement for the remaining soil. Fill material shall not contain sabkha soils and shall comply with the provisions of Sections 3.10 or 3.11.

SECTION 11.7
STABILIZATION

Where the sabkha soil is stabilized in lieu of designing footings or foundations in accordance with Section 11.3.2, the soil shall be stabilized by stone columns, preloading, vibroflotation, or other techniques designed by a geotechnical engineer knowledgeable in sabkha soil and approved by the building official. Where attempts to densify the upper portion of sabkha material by conventional means, in order to improve its bearing capacity and reduce its settlement characteristics, the upper, loose portion of sabkha shall be densified, or treated without adversely affecting the underlying cemented layers. In pre-wetting technique, the effect of strength loss shall be evaluated to ensure that strength criteria are met. Limitations and implementation procedures of the contemplated stabilization technique shall receive careful consideration and thorough evaluation.

CHAPTER 12
DESIGN FOR VIBRATORY LOADS

SECTION 12.1
GENERAL

12.1.1 Where machinery operations or other vibrations are transmitted through the foundation, the foundations and support structures shall be designed according to Sections 12.2 through 12.4.7. Foundations and support structures designed for machinery vibrations must be capable of withstanding dynamic loading due to machinery vibrations and all other loadings to which they may be subjected with stresses not exceeding the allowable-load bearing values specified in Chapter 4.

SECTION 12.2
LOADS AND FORCES

12.2.0 All concrete sections shall be proportioned to resist the sum of the static loads and dynamic forces as described in Sections 12.2.1 through 12.2.3.

12.2.1 **Static loads**. Static loads shall consist of all dead and live loads on the foundation, etc., thermal and fluid forces from process piping, loads due to temperature differentials, wind loads and any other sustained loads.

12.2.2 **Transient dynamic forces.** If not specified by the equipment manufacturer, transient forces consisting of vertical, lateral, and longitudinal forces equal to 25 percent of the total weight of the machine train and acting through the center machine bearing axis shall be used in design. These forces need not be considered to act concurrently. For purposes of strength design, the forces shall be treated as quasi-static loads.

For low-tuned systems, dynamic load effects due to transient resonance during machine start-up or shutdown shall be considered. For transient response calculations, damping effects shall be included to avoid unrealistically high results as the frequency ratio passes through the 0.7 to 1.3 range. Unless foundations or structures or connecting piping are unusual, response due to transient dynamic forces need not be evaluated.

12.2.3 **Steady state dynamic forces.** Information on steady state dynamic forces shall be furnished by the equipment manufacturer(s). For reciprocating machinery the supplied information shall include weights of the machine and all auxiliary equipment with exact location of centers of gravity, number of revolutions per minute (Operating speed or range of operating speeds), diagrams showing all primary and secondary forces and moments, and curves of free forces and moments against crank angle degrees.

For rotating machinery the equipment manufacturer(s) shall supply the weights of the machine, rotor and auxiliary equipment with exact location of centers of gravity, range of operating speeds, possible unbalanced forces and points of application (for operating conditions based on alarm level).

Where there is no manufacturer information available, the steady state dynamic force for rotating machinery can be estimated as follows:

$$F_D = 0.001W(rpm)^{1.5} \quad \textbf{(Equation 12-1)}$$

where:

F_D = Steady state dynamic force in kN.

W = Total mass of the rotating part in kg.

rpm = Machine speed in revolutions per minute.

SECTION 12.3
SOIL BEARING PRESSURES, PILE CAPACITIES AND SETTLEMENTS

12.3.1 Foundation adequacy for static bearing capacity and settlement considerations shall be checked by a registered design professional. In addition, effect of dynamic loading on foundation soil shall be investigated. In-situ or laboratory testing to establish appropriate dynamic parameters of the foundation soils, whether in-situ treated or untreated, or compacted fill, shall be carried out by an approved agency. If a requirement for piles is established, appropriate dynamic parameters for the piles shall be determined by an approved agency.

The site investigation report shall give insight to the expected dynamic behavior of the soil or piles. As a minimum the report should give the density, Poisson's ratio, dynamic modulus of sub-grade reaction or dynamic pile spring constant and the shear modulus for soils, or the equivalent fixate level of piles.

Unless foundation settlement calculations for dynamic loads show otherwise, the allowable soil bearing pressures shall not exceed 50% of the allowable bearing pressure permitted for static loads, as determined from Chapter 4, for high-tuned foundations and 75% for low-tuned foundations. The allowable soil bearing pressure shall be reduced for heavy machinery foundations to provide a factor of safety against excessive settlement due to vibration.

SECTION 12.4
DESIGN REQUIREMENTS

12.4.0 Foundation and support structures designed for machinery vibrations shall meet the provisions of Sections 12.4.1 through 12.4.7

12.4.1 **General**. The provisions of Chapters 5 and 8 shall be expanded to include the following:

1. Support structures or foundations for centrifugal rotating machinery greater than 500 horsepower shall be designed for the expected dynamic forces using dynamic analysis procedures. For units less than 500 horsepower, in the absence of a detailed dynamic analysis, the foundation weight shall be at least three times the total machinery weight, unless specified otherwise by the Manufacturer.

2. For reciprocating machinery less than 200 horsepower, in the absence of a detailed dynamic analysis, the foundation weight shall be at least five times the total machinery weight, unless specified otherwise by the Manufacturer.

3. All coupled elements of the machinery train shall be mounted on a common foundation or support structure.

4. Foundations for heavy machinery shall be independent of adjacent foundations and buildings. Concrete slabs or paving adjacent to the foundation shall have a minimum 12 mm isolation joint around the foundation using an approved elastic joint filler with sealant on top. Joint filler material shall be an expansion joint material according to ACI 504R Guide for Sealing Joints in Concrete Structures. Preformed expansion joint filler shall be of the full thickness and depth of the joint with splicing only on the length.

5. The clear distance in any direction between adjacent foundations for heavy machinery shall be large enough to avoid transmission of detrimental vibration amplitudes through the surrounding soil or the foundations shall be protected in other ways. Transmissibility of amplitudes shall be limited to 20 percent between adjacent foundations, unless otherwise agreed by the Building official.

6. Where practical and economical, the machine foundation system shall be proportioned to be low-tuned.

7. High-tuned machine foundation systems shall be used only when a low-tuned system is not practical or economical.

8. For elevated machinery, the flexibility of the entire support structure shall be considered in the dynamic analysis.

9. The foundation design shall be capable of resisting all applied dynamic and static loads specified by the machinery manufacturer, loads from thermal movement, dead and live loads, wind or seismic forces as specified in SBC 301, any loads that may be associated with installation or maintenance of the equipment, and fatigue. For fatigue, the dynamic loads shall be increased by a factor of 1.5 and applied as quasi-static loads.

The applied loads shall be combined to produce the most unfavorable effect on the supporting foundations. The effect of both wind and seismic activity need not be considered to act simultaneously. Design load combinations shall be as specified in Section 2.4 SBC 301 except that strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

10. Design shall be such that buried cables, pipes etc., will not be incorporated in the foundation, and be protected from the influence of foundation stresses. If incorporation in the foundation cannot be avoided, cables and pipes shall be sleeved.

11. Where practical, operator platforms shall be independent from the main machinery carrying structure(s).

12. Quantifying whole-body vibration in relation to human health and comfort, the probability of vibration perception, and the incidence of motion sickness shall conform to International Organization for Standardization ISO 2631-1 Mechanical Vibration and Shock Evaluation of Human Exposure to Whole-Body Vibration – Part 1: General Requirements and Evaluation of Human Exposure to Whole-Body Vibration – Part 2: Continuous and Shock-induced Vibration in Buildings (1 to 80 Hz) ISO 2631-2.

12.4.2 **Reinforced concrete.** The structural design of all reinforced concrete shall be in accordance with SBC 304 when not in conflict with the provisions of Chapter 12. The following provisions shall be satisfied:

1. The minimum compressive strength of concrete at 28 days shall not be less than 28 MPa.
2. All faces of concrete shall be reinforced bi-axially. For deformed bars, the reinforcement in each direction shall not be less than 0.0018 times the gross area perpendicular to the direction of reinforcement.

   Exception: In the event that a foundation size greater than 1200 mm thick is required for stability, rigidity, or damping, the minimum reinforcing steel may be as recommended in ACI 207.2R Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concret*e* with a suggested minimum reinforcement of Dia 22 mm bars at 300 mm on center.
3. Main reinforcement in piers shall not be less than 1 percent nor more than 8 percent of the cross-sectional area of the piers. Main reinforcement in pedestals shall not be less than 1/2 percent.
4. Minimum tie size in piers shall be 12 mm.
5. Maximum tie spacing in piers shall be the smallest of 8-bar diameters, 24-tie diameters or 1/3 the least dimension of the pier.
6. Slabs with thickness of 500 mm or more shall be provided with shrinkage and temperature reinforcement in accordance with applicable provisions of SBC 304.
7. When foundation thickness is greater than 1200 mm thick, mix and placement of concrete shall conform to the provisions of ACI 207.2R and SBC 304.

12.4.3 **Anchor bolts.** Anchor bolts shall be in accordance with SAES-Q-005. When specified, the diameter, steel quality, projection and installation method shall be as required by the machine manufacturer. Requirements for anchor bolt coating shall be in compliance with Saudi Aramco Materials System Reports 12-SAMSS-007 Fabrication of Structural and Miscellaneous Steel and requirements for double nuts shall be in compliance with Saudi Aramco Engineering Standard SAES-Q-005 Concrete Foundations.

The foundation design engineer shall verify the capacity of any vendor furnished or detailed anchor bolts. Unless otherwise specified by the equipment manufacturer, equipment shall be installed on mounting plate(s), and the direct attachment of equipment feet to the foundation using the anchor bolts shall not be permitted. Mounting plates shall be of sufficient strength and rigidity to transfer the applied forces to the foundation. Grouting shall be in accordance with Saudi Aramco Engineering Standard SAES-Q-011 Epoxy Grout for machinery Support and machine manufacturer’s instructions.

The drawing shall clearly indicate the locations and types of the anchor bolts and sleeves, the anchor bolt diameter, the depth of embedment into the foundation of the anchor bolts, the length of the anchor bolts threads, and the length of the anchor bolt projections.

12.4.4 **Stiffness requirements.** The foundation must be of sufficient width to prevent rocking and adequate depth to permit properly embedded anchor bolts. The width of the foundation shall be at least 1.5 times the vertical distance from the base to the machine centerline, unless analysis carried out by a registered design professional demonstrates that a lesser value will perform adequately. For concrete foundations, the weight of the foundation for reciprocating equipment shall not be less than 5 times and, for rotary equipment, shall not be less than 3 times the weight of the machinery, including its base plate and the piping supported from the foundation, unless analysis carried out by a registered design professional demonstrates that a lesser value will perform adequately.

For foundations and piers constructed with normal weight concrete, the dynamic modulus of elasticity shall be taken as:

$$E_D = 6560(f_c^{'})^{0.5}$$ **(Equation 12-2)**

where:

E_D = Dynamic modulus of elasticity of concrete in MPa.

$f_c^{'}$ = Compressive strength of concrete at 28 days in MPa.

The minimum thickness of the concrete foundations shall not be less than (0.60+L/30) where L is the length of foundation in meters parallel to the machine bearing axis in meters. Piers shall not be used unless absolutely required by operation or maintenance or if required by machine vendor specification. Block foundations for reciprocating machines shall have a minimum of 50 % of the block thickness embedded in the soil, unless otherwise specified by the equipment manufacturer.

12.4.5 **Allowable eccentricities for concrete foundations with horizontal shaft machinery.** Secondary moments that could significantly influence the natural frequencies of the foundation shall be minimized. The horizontal eccentricity, perpendicular to the machine bearing axis, between the center of gravity of the machine foundation system and the centroid of the soil contact area (or in case of piled foundations, the elastic support point of the pile group) shall not exceed 0.05 times the width of foundation in meters.

The horizontal eccentricity, parallel to the bearing axis between the center of gravity of the machine foundation system and the centroid of the soil contact area (or in the case of piled foundations, the elastic support point of the pile group) shall not exceed 0.05 times length of foundation in meters. The machine bearing axis and the centroid of the support (soil contact area, or pile group) shall lie in a common vertical plane.

Piers and columns shall be proportioned in such a manner that the centroid of their vertical stiffness lies in the same vertical plane as the bearing axis and center of gravity of the machinery.

12.4.6 **Permissible frequency ratios.** The ratio between the operating frequency of the machinery, f, and each natural frequency of the machine foundation system, f_n shall not lie in the range of 0.7 to 1.3. Accordingly, for high-tuned systems, f/f_n, shall be less than 0.7 and for low-tuned systems f/f_n shall be greater than 1.3. A need for exceptions shall be approved by a registered design professional.

12.4.7 **Permissible vibration.** Where Manufacturer's vibration criteria are not available**,** the maximum velocity of movement during steady-state normal operation shall be limited to 3 mm per second for centrifugal machines and 4 mm per second for reciprocating machines. For rocking and torsional mode calculation the vibration velocities shall be computed with the dynamic forces of the machinery train components assumed in phase and 180 degrees out of phase.

CHAPTER 13
DAMPPROOFING AND WATERPROOFING

SECTION 13.1
SCOPE

13.1.0 Walls or portions thereof that retain earth and enclose interior spaces and floors below grade, and underground water-retention structures shall be waterproofed and dampproofed in accordance with provisions of this Chapter, with the exception of those spaces containing groups other than residential and institutional where such omission is not detrimental to the building or occupancy.
Ventilation for crawl spaces shall comply with Section 7.3.4 SBC 201.

13.1.1 **Story above grade.** Where a basement is considered a story above grade and the finished ground level adjacent to the basement wall is below the basement floor elevation for 25 percent or more of the perimeter, the floor and walls shall be damp proofed in accordance with Section 13.2 and a foundation drain shall be installed in accordance with Section 13.4.2. The foundation drain shall be installed around the portion of the perimeter where the basement floor is belowground level. The provisions of Sections 2.2.3, 13.3 and 13.4.1 shall not apply in this case.

13.1.2 **Under-floor space.** The finished ground level of an under-floor space such as a crawl space shall not be located below the bottom of the footings. Where there is evidence that the groundwater table rises to within 150 mm of the ground level at the outside building perimeter, or that the surface water does not readily drain from the building site, the ground level of the under-floor space shall be as high as the outside finished ground level, unless an approved drainage system is provided. The provisions of Sections 2.2.3, 13.2, 13.3 and 13.4.1 shall not apply in this case.

13.1.2.1 **Flood hazard areas.** For buildings and structures in flood hazard areas, as established in SBC 301 Section 5.3, the finished ground level of an under-floor space such as a crawl space shall be equal to or higher than the outside finished ground level.

Exception: Under-floor spaces of Group R-3 buildings that meet the requirements of FEMA/ FIA-TB-11.

13.1.3 **Groundwater control.** Where the ground-water table is lowered and maintained at an elevation not less than 150 mm below the bottom of the lowest floor, the floor and walls shall be dampproofed in accordance with Section 13.2. The design of the system to lower the groundwater table shall be based on accepted principles of engineering that shall consider, but not necessarily be limited to, permeability of the soil, rate at which water enters the drainage system, rated capacity of pumps, head against which pumps are to operate and the rated capacity of the disposal area of the system.

SECTION 13.2
DAMP PROOFING REQUIRED

13.2.0 Where hydrostatic pressure will not occur as determined by Section 2.2.3, floors and walls shall be dampproofed in accordance with this Section.

13.2.1 **Floors.** Dampproofing materials for floors shall be installed between the floor and the base course required by Section 13.4.1, except where a separate floor is provided above a concrete slab. Where installed beneath the slab, dampproofing shall consist of not less than 0.15 mm polyethylene with joints lapped not less than 150 mm, or other approved methods or materials. Where permitted to be installed on top of the slab, dampproofing shall consist of mopped-on bitumen, not less than 0.10 mm polyethylene, or other approved methods or materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

13.2.2 **Walls.** Dampproofing materials for walls shall be installed on the exterior surface of the wall, and shall extend from the top of the footing to above ground level.

Dampproofing shall consist of a bituminous material, 16 N/m^2 of acrylic modified cement, 3.0 mm coat of surface-bonding mortar complying with ASTM C 887, any of the materials permitted for waterproofing by Section 13.3.2 or other approved methods or materials.

13.2.2.1 **Surface preparation of walls.** Prior to application of dampproofing materials on concrete walls, holes and recesses resulting from the removal of form ties shall be sealed with a bituminous material or other approved methods or materials. Unit masonry walls shall be parged on the exterior surface belowground level with not less than 10 mm of Portland cement mortar. The parging shall be coved at the footing.

Exception: Parging of unit masonry walls is not required where a material is approved for direct application to the masonry.

SECTION 13.3
WATERPROOFING REQUIRED

13.3.0 Where the groundwater investigation required by Section 2.2.3 indicates that a hydrostatic pressure condition exists, and the design does not include a ground-water control system as described in Section 13.1.3, walls and floors shall be waterproofed in accordance with this section.

13.3.1 **Floors.** Floors required to be waterproofed shall be of concrete, designed and constructed to withstand the hydrostatic pressures to which the floors will be subjected.

Waterproofing shall be accomplished by placing a membrane of rubberized asphalt, butyl rubber, or not less than 0.15 mm polyvinyl chloride with joints lapped not less than 150 mm or other approved materials under the slab. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

13.3.2 **Walls.** Walls required to be waterproofed shall be of concrete or masonry and shall be designed and constructed to withstand the hydrostatic pressures and other lateral loads to which the walls will be subjected.

Waterproofing shall be applied from the bottom of the wall to not less than 300 mm above the maximum elevation of the groundwater table. The remainder of the wall shall be dampproofed in accordance with Section 13.2.2. Waterproofing shall consist of two-ply hot-mopped felts, not less than 0.15 mm polyvinyl chloride, 1.0

mm polymer-modified asphalt, 0.150 mm polyethylene or other approved methods or materials capable of bridging nonstructural cracks. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

13.3.2.1 **Surface preparation of walls.** Prior to the application of waterproofing materials on concrete or masonry walls, the walls shall be prepared in accordance with Section 13.2.2.1.

13.3.3 **Joints and penetrations.** Joints in walls and floors, joints between the wall and floor and penetrations of the wall and floor shall be made watertight utilizing approved methods and materials.

SECTION 13.4
SUBSOIL DRAINAGE SYSTEM

13.4.0 Where a hydrostatic pressure condition does not exist, damp proofing shall be provided and a base shall be installed under the floor and a drain installed around the foundation perimeter. A subsoil drainage system designed and constructed in accordance with Section 13.1.3 shall be deemed adequate for lowering the ground-water table.

13.4.1 **Floor base course.** Floors of basements, except as provided for in Section 13.1.1, shall be placed over a floor base course not less than 100 mm in thickness that consists of gravel or crushed stone containing not more than 10 percent of material that passes through a No. 4 (4.75 mm) sieve.

Exception: Where a site is located in well-drained gravel or sand/gravel mixture soils, a floor base course is not required.

13.4.2 **Foundation drain.** A drain shall be placed around the perimeter of a foundation. It shall satisfy the requirements of Equations 7-4 through 7-6 or in lieu it shall consist of gravel or crushed stone containing not more than 10-percent material that passes through a No. 4 (4.75 mm) sieve. The drain shall extend a minimum of 300 mm beyond the outside edge of the footing. The thickness shall be such that the bottom of the drain is not higher than the bottom of the base under the floor, and that the top of the drain is not less than 150 mm above the top of the footing. The top of the drain shall be covered with an approved filter membrane material. Where a drain tile or perforated pipe is used, the invert of the pipe or tile shall not be higher than the floor elevation. The top of joints or the top of perforations shall be protected with an approved filter membrane material. The pipe or tile shall be placed on not less than 50 mm of gravel or crushed stone complying with Section 13.4.1, and shall be covered with not less than 150 mm of the same material.

13.4.3 **Drainage discharge.** The floor base and foundation perimeter drain shall discharge by gravity or mechanical means into an approved drainage system that complies with the SBC 701.

Exception: Where a site is located in well-drained gravel or sand/gravel mixture soils, a dedicated drainage system is not required.

SECTION 13.5
UNDERGROUND WATER-RETENTION STRUCTURES

13.5.0 Underground water-retention structures shall meet the provisions of Sections 13.5.1 through 13.5.4.

13.5.1 General requirements. All underground water-retention structures shall meet the following requirements:

1. All internal faces (including the top face) of the water-retention structure shall be waterproofed.
2. Shall not be located under drainage or non-potable water piping.
3. Shall be provided with a waterproof cover to prevent water and foreign matter from entering the tank. The cover shall be large enough to allow access for maintenance.
4. Underground tanks in flood hazard areas shall be anchored to prevent flotation, collapse or lateral movement resulting from hydrostatic loads, including the effects of buoyancy, during conditions of the design flood.

13.5.2 **Design and construction.** For design and constructions of underground water-retention structures, provisions of SBC 304 and ACI 350 Environmental Engineering Concrete Structures shall govern, where applicable.

13.5.3 **Waterproofing**. All internal faces of an underground water-retention structure shall be waterproofed (using approved material such as epoxy films, concrete admixtures, etc.). All such waterproofing materials in contact with water shall neither be toxic nor hazardous to human health. All construction joints shall have proper water-stops. All construction holes, recesses, plumbing sleeves etc. shall be sealed properly.

In cases where the floor slab of the water-retention structure is less than one meter above the anticipated groundwater level, it is required to provide a base layer of compacted granular fill, followed by damp proofing layer as described in Section 13.2.1.

In cases where floor slab is below or close to groundwater level, the floor slab and all exterior faces of the structure shall be water proofed in accordance with Section 13.3. In cases where cover slab of the structure is below or close to groundwater level, all parts of the structure (including the opening of the water-retention structure) shall be waterproofed in accordance with Section 13.3.

13.5.4 **Testing.** Following complete application of waterproofing of the structure, and before backfilling is permitted; underground water-retention structures shall be tested against leakage full of water for a minimum of 48 hours.

CHAPTER 14
GENERAL REQUIREMENTS FOR PIER AND PILE FOUNDATIONS

SECTION 14.1
DESIGN

14.1.1 Piles are permitted to be designed in accordance with provisions for piers in Chapter 14 and Sections 17.3 through 17.10 where either of the following conditions exists, subject to the approval of the building official:

1. Group R-3 and U occupancies not exceeding two stories of light-frame construction, or
2. Where the surrounding foundation materials furnish adequate lateral support for the pile.

SECTION 14.2
GENERAL

14.2.1 Pier and pile foundations shall be designed and installed on the basis of a site investigation as defined in Chapter 2, unless the building official ascertains that sufficient data upon which to base the design and installation is available.

The investigation and report provisions of Chapter 2 shall be expanded to include, but not be limited to, the following:

1. Recommended pier or pile types and installed capacities.
2. Recommended center-to-center spacing of piers or piles.
3. Driving criteria.
4. Installation procedures.
5. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).
6. Pier or pile load test requirements.
7. Durability of pier or pile materials.
8. Designation of bearing stratum or strata.
9. Reductions for group action, where necessary.

SECTION 14.3
SPECIAL TYPES OF PILES

14.3.1 The use of types of piles not specifically mentioned herein is permitted, subject to the approval of the building official, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such piles. The allowable stresses shall not in any case exceed the limitations specified herein.

SECTION 14.4
PILE CAPS

14.4.1 Pile caps shall be of reinforced concrete, and shall include all elements to which piles are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of piles shall be embedded not less than 75 mm into pile caps and the caps shall extend at least 100 mm beyond the edges of piles. The tops of piles shall be cut back to sound material before capping.

SECTION 14.5
STABILITY

14.5.1 Piers or piles shall be braced to provide lateral stability in all directions. Three or more piles connected by a rigid cap shall be considered braced, provided that the piles are located in radial directions from the centeroid of the group not less than 60 degrees (1 radian) apart. A two-pile group in a rigid cap shall be considered to be braced along the axis connecting the two piles. Methods used to brace piers or piles shall be subject to the approval of the building official.

Piles supporting walls shall be driven alternatively in lines spaced at least 300 mm apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the wall piles are adequately braced to provide for lateral stability. A single row of piles without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories or 10 m in height, provided the centers of the piles are located within the width of the foundation wall.

SECTION 14.6
STRUCTURAL INTEGRITY

14.6.1 Piers or piles shall be installed in such a manner and sequence as to prevent distortion or damage to piles being installed or already in place to extent that such distortion or damage affects the structural integrity of the piles.

SECTION 14.7
SPLICES

14.7.1 Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the pier or pile during installation and subsequent thereto and shall be of adequate strength to transmit the vertical and lateral loads and moments occurring at the location of the splice during driving and under service loading. Splices shall develop not less than 50% of the least capacity of the pier or pile in bending. In addition, splices occurring in the upper 3 m of the embedded portion of the pier or pile shall be capable of resisting at allowable working stresses the moment and shear that would result from an assumed eccentricity of the pier or pile load of 75 mm, or the pier or pile shall be braced in accordance with Section 14.5 to other piers or piles that do not have splices in the upper 3 m of embedment.

SECTION 14.8
ALLOWABLE PIER OR PILE LOADS

14.8.1 **Determination of allowable loads.** The allowable axial and lateral loads on piers or piles shall be determined by an approved formula, load tests or method of analysis.

14.8.2 **Driving criteria.** The allowable compressive load on any pile where determined by the application of an approved driving formula shall not exceed 360 kN. For allowable loads above 360 kN, the wave equation method of analysis shall be used to estimate pile drivability of both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section 14.8.3. The formula or wave equation load shall be determined for gravity-drop or power-actuated hammers and the hammer energy used shall be the maximum consistent with the size, strength and weight of the driven piles. The use of a follower is permitted only with the approval of the building official. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.

14.8.3 **Load tests.** Where design compressive loads per pier or pile are greater than those permitted by Section 14.10, or where the design load for any pier or pile foundation is in doubt, control test piers or piles shall be tested in accordance with ASTM D 1143 or ASTM D 4945. At least one pier or pile shall be test loaded in each area of uniform subsoil conditions. Where required by the building official, additional piers or piles shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate load capacity of the test piers or pile as assessed by one of the published methods listed in Section 14.8.3.1 with consideration for the test type, duration and subsoil. The ultimate load capacity shall be determined by a registered design professional, but shall be no greater than two times the test load that produces a settlement of 7 mm. In subsequent installation of the balance of foundation piles, all piles shall be deemed to have a supporting capacity equal to the control pile where such piles are to the same type, size and relative length as the test pile; are installed using the same or comparable methods and equipment as the test pile; are installed in similar subsoil conditions as the test pile; and, for driven piles, where the rate of penetration (e.g., net displacement per blow) of such piles is equal to or less than that of the test pile through a comparable driving distance.

14.8.3.1 **Load test evaluation.** It shall be permitted to evaluate pile load tests with any of the following methods:

1. Davison Offset Limit.
2. Brinch-Hansen 90% Criterion.
3. Chin-Konder Extrapolation.
4. Other methods approved by the building official.

14.8.4 **Allowable frictional resistance.** The assumed frictional resistance developed by any pier or uncased cast-in-place pile shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 4.1, up to a maximum of 25 kPa, unless a greater value is allowed by the building official after a soil investigation as specified in Chapter 2 is submitted. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended by a soil investigation as specified in Chapter 2.

14.8.5 **Uplift capacity.** Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 14.8.3 divided by a factor of safety of two. For pile groups subjected to uplift, the allowable working uplift load for the group shall be the lesser of:

1. The proposed individual pile uplift working load times the number of piles in the group.
2. Two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the pile.

14.8.6 **Load-bearing capacity.** Piers, individual piles and groups of piles shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two.

14.8.7 **Bent piers or piles.** The load-bearing capacity of piers or piles discovered to have a sharp or sweeping bend shall be determined by an approved method of analysis or by load testing a representative pier or pile.

14.8.8 **Overloads on piers or piles.** The maximum compressive load on any pier or pile due to mislocation shall not exceed 110 % of the allowable design load.

SECTION 14.9
LATERAL SUPPORT

14.9.1 **General.** Any soil other than fluid soil shall be deemed to afford sufficient lateral support to the pier or pile to prevent buckling and to permit the design of the pier or pile in accordance with accepted engineering practice and the applicable provisions of this code.

14.9.2 **Unbraced piles.** Piles standing unbraced in air, water or in fluid soils shall be designed as columns in accordance with the provisions of this code. Such piles driven into firm ground can be considered fixed and laterally supported at 1.5 m below the ground surface and in soft material at 3 m below the ground surface unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

14.9.3 **Allowable lateral load.** Where required by the design, the lateral load capacity of a pier, a single pile or a pile group shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 25 mm at the ground surface.

SECTION 14.10
USE OF HIGHER ALLOWABLE PIER OR PILE STRESSES

14.10.1 Allowable stresses greater than those specified for piers or for each pile type in Chapters 14 and 15 are permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A soil investigation in accordance with Chapter 2.
2. Pier or pile load tests in accordance with Section 14.8.3, regardless of the load supported by the pier or pile.

The design and installation of the pier or pile foundation shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pier or pile foundations who shall certify to the building official that the piers or piles as installed satisfy the design criteria.

SECTION 14.11
PILES IN SUBSIDING AND CALCAREOUS AREAS

14.11.1 **Piles in subsiding areas**. Where piles are driven through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the piles by the subsiding upper strata.

Where the influence of subsiding fills is considered as imposing loads on the pile, the allowable stresses specified in this chapter are permitted to be increased where satisfactory substantiating data are submitted.

14.11.2 **Piles in calcareous soils.** Where piles are driven through calcareous soils and derive support from frictional forces developed between the pile and the surrounding soil, consideration shall be given to loss of frictional forces due to driving. For bored cast in-situ piles within calcareous soils where support is derived from both friction and tip resistance, consideration shall be given to the possibility of presence of voids/cavities below the tip of the bored pile.

SECTION 14.12
SETTLEMENT ANALYSIS

14.12.1 The settlement of piers, individual piles or groups of piles shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any stresses to exceed allowable values.

SECTION 14.13
PREEXCAVATION

14.13.1 The use of jetting, augering or other methods of preexcavation shall be subject to the approval of the building official. Where permitted, preexcavation shall be carried out in the same manner as used for piers or piles subject to load tests and in such a manner that will not impair the carrying capacity of the piers or piles already in place or damage adjacent structures. Pile tips shall be driven below the preexcavated depth until the required resistance or penetration is obtained.

SECTION 14.14
INSTALLATION SEQUENCE

14.14.1 Piles shall be installed in such sequence as to avoid compacting the surrounding soil to the extent that other piles cannot be installed properly, and to prevent ground movements that are capable of damaging adjacent structures.

SECTION 14.15
USE OF VIBRATORY DRIVERS

14.15.1 Vibratory drivers shall only be used to install piles where the pile load capacity is verified by load tests in accordance with Section 14.8.3. The installation of production piles shall be controlled according to power consumption, rate of penetration or other approved means that ensure pile capacities equal or exceed those of the test piles.

SECTION 14.16
PILE DRIVABILITY

14.16.1 Pile cross sections shall be of sufficient size and strength to withstand driving stresses without damage to the pile, and to provide sufficient stiffness to transmit the required driving forces.

SECTION 14.17
PROTECTION OF PILE MATERIALS

14.17.1 Where boring records or site conditions indicate possible deleterious action on pier or pile materials because of soil constituents, changing water levels or other factors, the pier or pile materials shall be adequately protected by materials, methods or processes approved by the building official. Protective materials shall be applied to the piles so as not to be rendered ineffective by driving. The effectiveness of such protective measures for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence.

SECTION 14.18
USE OF EXISTING PIERS OR PILES

14.18.1 Piers or piles left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the building official, which indicates that the piers or piles are sound and meet the requirements of this code. Such piers or piles shall be load tested or redriven to verify their capacities. The design load applied to such piers or piles shall be the lowest allowable load as determined by tests or redriving data.

SECTION 14.19
HEAVED PILES

14.19.1 Piles that have heaved during the driving of adjacent piles shall be redriven as necessary to develop the required capacity and penetration, or the capacity of the pile shall be verified by load tests in accordance with Section 14.8.3.

SECTION 14.20
IDENTIFICATION

14.20.1 Pier or pile materials shall be identified for conformity to the specified grade with this identity maintained continuously from the point of manufacture to the point of installation or shall be tested by an approved agency to determine conformity to the specified grade. The approved agency shall furnish an affidavit of compliance to the building official.

SECTION 14.21
PIER OR PILE LOCATION PLAN

14.21.1 A plan showing the location and designation of piers or piles by an identification system shall be filed with the building official prior to installation of such piers or piles. Detailed records for piers or individual piles shall bear an identification corresponding to that shown on the plan.

SECTION 14.22
SPECIAL INSPECTION

14.22.1 Special inspections in accordance with Sections 2.7.6 and 2.7.7, SBC 302 shall be provided for piles and piers, respectively.

SECTION 14.23
SEISMIC DESIGN OF PIERS OR PILES

14.23.1 **Seismic design category C.** Where a structure is assigned to Seismic Design Category C in accordance with SBC 301 Chapters 9 through 16, the following shall apply. Individual pile caps, piers or piles shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, a force equal to the product of the larger pile cap or column load times the seismic coefficient, S_{DS}, divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils.

Exception: Piers supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, lightly loaded exterior decks and patios, of Group R-3 and U occupancies not exceeding two stories of light-frame construction, are not subject to interconnection if it can be shown the soils are of adequate stiffness, subject to the approval of the building official.

14.23.1.1 **Connection to pile cap.** Concrete piles and concrete-filled steel pipe piles shall be connected to the pile cap by embedding the pile reinforcement or field-placed dowels anchored in the concrete pile in the pile cap for a distance equal to the development length. For deformed bars, the development length is the full development length for compression or tension, in the case of uplift, without reduction in length for excess area. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile will be permitted provided the design is such that any hinging occurs in the confined region.

Ends of hoops, spirals and ties shall be terminated with seismic hooks, as defined in Section 21.1 of SBC 304, turned into the confined concrete core. The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns.

For resistance to uplift forces, anchorage of steel pipe (round HSS sections), concrete-filled steel pipe or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section.

Exception: Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile.

Splices of pile segments shall develop the full strength of the pile, but the splice need not develop the nominal strength of the pile in tension, shear and bending when it has been designed to resist axial and shear forces and moments from the load combinations of Section 2.5 SBC 301.

14.23.1.2 **Design details.** Piers or pile moments, shears and lateral deflections used for design shall be established considering the nonlinear interaction of the shaft and soil, as recommended by a registered design professional. Where the ratio of the depth of embedment of the pile-to-pile diameter or width is less than or equal to six, the pile may be assumed to be rigid.

Pile group effects from soil on lateral pile nominal strength shall be included where pile center-to-center spacing in the direction of lateral force is less than eight pile diameters. Pile group effects on vertical nominal strength shall be included where pile center-to-center spacing is less than three pile diameters. The pile uplift soil nominal strength shall be taken as the pile uplift strength as limited by the frictional force developed between the soil and the pile.

Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pier or pile, provisions shall be made so that those specified lengths or extents are maintained after pier or pile cutoff.

14.23.2 **Seismic design category D**. Where a structure is assigned to Seismic Design Category D in accordance with SBC 301 Chapters 9 through 16, the requirements for Seismic Design Category C given in Section 14.23.1 shall be met, in addition to the following. Provisions of SBC 304 shall apply when not in conflict with the provisions of Chapter 14 through 17.

14.23.2.1 **Design details for piers, piles and grade beams.** Piers or piles shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces. Concrete piers or piles on Site Class E or F sites shall be designed and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of SBC 304 within seven pile diameters of the pile cap and the interfaces of soft to medium stiff clay or liquefiable strata. For precast prestressed concrete piles, detailing provisions as given in Sections 15.2.3.2.1 and 15.2.3.2.2 shall apply.

Grade beams shall be designed as beams in accordance with SBC 304, Chapter 21. When grade beams have the capacity to resist the forces from the load combinations in Section 2.5 SBC 301, they need not conform to Chapter 21 SBC 304.

14.23.2.2 **Connection to pile cap.** For piles required to resist uplift forces or provide rotational restraint, design of anchorage of piles into the pile cap shall be provided considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25% of the strength of the pile in tension. Anchorage into the pile cap shall be capable of developing the following:

1. In the case of uplift, the lesser of the nominal tensile strength of the longitudinal reinforcement in a concrete pile, or the nominal tensile strength of a steel pile, or the pile uplift soil nominal strength factored by 1.3 or the axial tension force resulting from the load combinations of Section 2.5 SBC 301.

2. In the case of rotational restraint, the lesser of the axial and shear forces, and moments resulting from the load combinations of Section 2.5 SBC 301 or development of the full axial, bending and shear nominal strength of the pile.

14.23.2.3 **Flexural strength.** Where the vertical lateral-force-resisting elements are columns, the grade beam or pile cap flexural strengths shall exceed the column flexural strength.

The connection between batter pile and grade beams or pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be capable of resisting forces and moments from the load combinations of Section 2.5 SBC 301.

CHAPTER 15
DRIVEN PILE FOUNDATIONS

SECTION 15.1
PRECAST CONCRETE PILES

15.1.1 **General.** The materials, reinforcement and installation of precast concrete piles shall conform to Sections 15.1.1.1 through 15.1.1.4.

15.1.1.1 **Design and manufacture**. Piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.

15.1.1.2 **Minimum dimension**. The minimum lateral dimension shall be 200 mm. Corners of square piles shall be chamfered.

15.1.1.3 **Reinforcement**. Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced not more than 100 mm apart, center to center, for a distance of 600 mm from the ends of the pile; and not more than 150 mm elsewhere except that at the ends of each pile, the first five ties or spirals shall be spaced 25 mm center to center. The gage of ties and spirals shall be as follows:

a. For piles having a diameter of 400 mm or less, wire shall not be smaller than 6 mm (No. 5 gage).

b. For piles having a diameter of 400 mm and less than 500 mm, wire shall not be smaller than 6 mm (No. 4 gage).

c. For piles having a diameter of 500 mm and larger, wire shall not be smaller than 7 mm (No. 3 gage).

15.1.1.4 **Installation**. Piles shall be handled and driven so as not to cause injury or overstressing, which affects durability or strength.

15.1.2 **Precast nonprestressed piles.** Precast nonprestressed concrete piles shall conform to Sections 15.1.2.1 through 15.1.2.5.

15.1.2.1 **Materials.** Concrete shall have a 28-day specified compressive strength ($f_c^{'}$) of not less than 20 MPa.

15.1.2.2 **Minimum reinforcement**. The minimum amount of longitudinal reinforcement shall be 0.8 percent of the concrete section and shall consist of at least four bars.

15.1.2.2.1 **Seismic reinforcement in seismic design category C**. Where a structure is assigned to Seismic Design Category C in accordance with SBC 301 Chapters 9 through 16, the following shall apply. Longitudinal reinforcement with a minimum steel ratio of 0.01 shall be provided throughout the length of precast concrete piles. Within three pile diameters of the bottom of the pile cap, the longitudinal reinforcement shall be confined with closed ties or spirals of a minimum 10 mm diameter. Ties or spirals shall be provided at a maximum spacing of eight times the diameter of the smallest longitudinal bar, not to exceed 150 mm. Throughout the remainder of the pile, the closed ties or spirals shall have a maximum spacing of 16 times the smallest longitudinal-bar diameter, not to exceed 200 mm.

15.1.2.2.2 **Seismic reinforcement in seismic design category D**. Where a structure is assigned to Seismic Design Category D in accordance with Chapters 9 through 16

SBC 301, the requirements for Seismic Design Category C in Section 15.1.2.2.1 shall apply except as modified by this section. Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of SBC 304 within three pile diameters of the bottom of the pile cap. For other than liquefiable sites and where spirals are used as the transverse reinforcement, it shall be permitted to use a volumetric ratio of spiral reinforcement of not less than one-half that required by Section 21.4.4.1(a) of SBC 304.

15.1.2.3 **Allowable stresses**. The allowable compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength ($f_c^{'}$) applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel (f_y) or a maximum of 210 MPa. The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel (f_y) or a maximum of 165 MPa.

15.1.2.4 **Installation**. A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength ($f_c^{'}$), but not less than the strength sufficient to withstand handling and driving forces.

15.1.2.5 **Concrete cover**. Reinforcement for piles that are not manufactured under plant conditions shall have a concrete cover of not less than 50 mm.

Reinforcement for piles manufactured under plant control conditions shall have a concrete cover of not less than 30 mm for Dia 16 mm bars and smaller, and not less than 40 mm for Dia 18 mm through Dia 36 mm bars except that longitudinal bars spaced less than 40 mm clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars.

Reinforcement for piles exposed to seawater shall have a concrete cover of not less than 75 mm.

15.1.3 **Precast prestressed piles**. Precast prestressed concrete piles shall conform to the requirements of Sections 15.1.3.1 through 15.1.3.5.

15.1.3.1 **Materials**. Prestressing steel shall conform to ASTM A416. Concrete shall have a 28-day specified compressive strength ($f_c^{'}$) of not less than 35 MPa.

15.1.3.2 **Design**. Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 3 MPa for piles up to 9 meters in length, 4 MPa for piles up to 15 meters in length and 5 MPa for piles greater than 15 m in length.

Effective prestress shall be based on an assumed loss of 210 MPa in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in SBC 304.

15.1.3.2.1 **Design in seismic design Category C.** Where a structure is assigned to Seismic Design Category C in accordance with SBC 301 Chapters 9 through 16, the following shall apply. The minimum volumetric ratio of spiral reinforcement shall not be less than 0.007 or the amount required by the following formula for the upper 6000 mm of the pile.

$$\rho_s = 0.12 f_c^{'} \big/ f_{yh}$$ **(Equation 15-1)**

where:

$f_c^{'}$ = Specified compressive strength of concrete, MPa.

f_{yh} = Yield strength of spiral reinforcement $\leq$ 586 MPa.

ρ_s = Spiral reinforcement index (vol. Spiral/vol. core).

At least one-half the volumetric ratio required by Equation 15-1 shall be provided below the upper 6000 mm of the pile.

The pile cap connection by means of dowels as indicated in Section 14.23.1 is permitted. Pile cap connection by means of developing pile reinforcing strand is permitted provided that the pile reinforcing strand results in a ductile connection.

15.1.3.2.2 **Design in seismic design category D**. Where a structure is assigned to Seismic Design Category D, in accordance with Chapters 9 through 16, SBC 301, the requirements for Seismic Design Category C in Section 15.1.3.2.1 shall be met, in addition to the following:

1. Requirements in Chapter 21, SBC 304, need not apply, unless specifically referenced.

2. Where the total pile length in the soil is 10 meters or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 10 m, the ductile pile region shall be taken as the greater of 10 m or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.

3. In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longitudinal strand, or 200 mm, whichever is smaller.

4. Circular spiral reinforcement shall be spliced by lapping one full turn and bending the end of the spiral to a 90-degree hook or by use of a mechanical or welded splice complying with Section 12.14.3 of SBC 304.

5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

$$\rho_s = 0.25(f_c^{'}/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/(f_c^{'}A_g)]$$ **(Equation 15-2)**

but not less than:

$$\rho_s = 0.12(f_c^{'}/f_{yh})[0.5 + 1.4P/(f_c^{'}A_g)]$$ **(Equation 15-3)**

and need not exceed:

$$\rho_s = 0.021$$ **(Equation 15-4)**

where:

A_g = Pile cross-sectional area, mm^2.
A_{ch} = Core area defined by spiral outside diameter, mm^2.
$f_c^{'}$ = Specified compressive strength of concrete, MPa.
f_{yh} = Yield strength of spiral reinforcement $\leq$ 580 MPa.
P = Axial load pile, kN, as determined from Section 2.3.2 SBC 301 Combinations (5, 6, and 7).
ρ_s = Volumetric ratio (vol. spiral/vol. core).

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.

6. When transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacing, and perpendicular to dimension, h_c, shall conform to:

$$A_{sh} = 0.3sh_c\,(f_c^{'}/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/(f_c^{'}A_g)] \qquad \textbf{(Equation 15-5)}$$

but not less than:

$$A_{sh} = 0.12sh_c\,(f_c^{'}/f_{yh})[0.5 + 1.4P/(f_c^{'}A_g)] \qquad \textbf{(Equation 15-6)}$$

where:

f_{yh} = $\leq$ 480 MPa.
h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, mm.
s = Spacing of transverse reinforcement measured along length of pile, mm.
A_{sh} = Cross-sectional area of transverse reinforcement, mm^2.
$f_c^{'}$ = Specified compressive strength of concrete~~,~~ MPa.

The hoops and cross ties shall be deformed bars not less than Dia 10 mm in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.
Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided.

15.1.3.3 **Allowable stresses**. The maximum allowable design compressive stress, f_c, in concrete shall be determined as follows:

$$f_c = 0.33 f_c^{'} - 0.27 f_{pc} \qquad \textbf{(Equation 15-7)}$$

where:

$f_c^{'}$ = The 28-day specified compressive strength of the concrete.
f_{pc} = The effective prestress stress on the gross section.

15.1.3.4 **Installation**. A prestressed pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength ($f_c^{'}$), but not less than the strength sufficient to withstand handling and driving forces.

15.1.3.5 **Concrete cover**. Prestressing steel and pile reinforcement shall have a concrete cover of not less than 30mm for square piles of 300 mm or smaller size and 40 mm for larger piles, except that for piles exposed to seawater, the minimum protective concrete cover shall not be less than 70 mm.

SECTION 15.2
STRUCTURAL STEEL PILES

15.2.1 **General**. Structural steel piles shall conform to the requirements of Sections 15.2.2 through 15.2.5.

15.2.2 **Materials**. Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A36, ASTM A252, ASTM A283, ASTM A572, ASTM A588 or ASTM A913.

15.2.3 **Allowable stresses**. The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength (f_y).

Exception: Where justified in accordance with Section 14.10, the allowable axial stress is permitted to be increased above 0.35 f_y, but shall not exceed 0.5 f_y.

15.2.4 **Dimensions of H-piles**. Sections of H-piles shall comply with the following:

1. The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.

2. The nominal depth in the direction of the web shall not be less than 200 mm.

3. Flanges and web shall have a minimum nominal thickness of 10 mm.

15.2.5 **Dimensions of steel pipe piles**. Steel pipe piles driven open ended shall have a nominal outside diameter of not less than 200 mm. The pipe shall have a minimum of 220 mm^2 of steel in cross section to resist each 1360 N-m of pile hammer energy or the equivalent strength for steels having a yield strength greater than 240 MPa. Where pipe wall thickness less than 5 mm is driven open ended, a suitable cutting shoe shall be provided.

CHAPTER 16
CAST-IN-PLACE CONCRETE PILE FOUNDATIONS

SECTION 16.1
GENERAL

16.1.0 The materials, reinforcement and installation of cast-in-place concrete piles shall conform to Sections 16.1.1 through 16.1.3.

16.1.1 **Materials.** Concrete shall have a 28-day specified compressive strength ($f_c^{'}$) of not less than 20 MPa. Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 100 mm and not more than 150 mm. Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

16.1.2 **Reinforcement.** Except for steel dowels embedded 1.5 m or less in the pile and as provided in Section 16.3.4, reinforcement where required shall be assembled and tied together and shall be placed in the pile as a unit before the reinforced portion of the pile is filled with concrete except in augered uncased cast-in-place piles. Tied reinforcement in augered uncased cast-in-place piles shall be placed after piles are concreted, while the concrete is still in a semifluid state.

16.1.2.1 **Reinforcement in seismic design Category C.** Where a structure is assigned to Seismic Design Category C in accordance with Chapters 9 through 16, SBC 301, the following shall apply. A minimum longitudinal reinforcement ratio of 0.0025 shall be provided for uncased cast-in-place concrete drilled or augered piles, piers or caissons in the top one-third of the pile length, a minimum length of 3 m below the ground or that required by analysis, whichever length is greatest. The minimum reinforcement ratio, but no less than that ratio required by rational analysis, shall be continued throughout the flexural length of the pile. There shall be a minimum of four longitudinal bars with closed ties (or equivalent spirals) of a minimum 10 mm diameter provided at 16-longitudinal-bar diameter maximum spacing. Transverse confinement reinforcing with a maximum spacing of 150 mm or 8-longitudinal-bar diameters, whichever is less, shall be provided within a distance equal to three times the least pile dimension of the bottom of the pile cap.

16.1.2.2 **Reinforcement in seismic design category D**. Where a structure is assigned to Seismic Design Category D in accordance with SBC 301 Chapters 9 through 16, the requirements for Seismic Design Category C given above shall be met, in addition to the following. A minimum longitudinal reinforcement ratio of 0.005 shall be provided for uncased cast-in-place drilled or augered concrete piles, piers or caissons in the top one-half of the pile length, a minimum length of 3 m below ground or throughout the flexural length of the pile, whichever length is greater. The flexural length shall be taken as the length of the pile to a point where the concrete section cracking moment strength multiplied by 0.4 exceeds the required moment strength at that point. There shall be a minimum of four longitudinal bars with transverse confinement reinforcing provided in the pile in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of SBC 304 three times the least pile dimension of the bottom of the pile cap. It shall be permitted to use a transverse spiral reinforcing ratio of not less than one-half of that required in Section 21.4.4.1

(a) of SBC 304 for other than liquefiable sites. Tie spacing throughout the remainder of the concrete section shall not exceed 12-longitudinal-bar diameters, one-half the least dimension of the section, nor 300 mm. Ties shall be a minimum of Dia 10 mm bars for piles with a least dimension up to 500 mm, and Dia 12 mm for larger piles.

16.1.3 **Concrete placement.** Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pile, the concrete shall not be chutted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pile.

SECTION 16.2
ENLARGED BASE PILES

16.2.0 Enlarged base piles shall conform to the requirements of Sections 16.2.1 through 16.2.5.

16.2.1 **Materials.** The maximum size for coarse aggregate for concrete shall be 20 mm. Concrete to be compacted shall have a zero slump.

16.2.2 **Allowable stresses.** The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25% of the 28-day specified compressive strength ($f_c^{'}$). Where the concrete is placed in a permanent steel casing, the maximum allowable concrete stress shall be 33% of the 28-day specified compressive strength ($f_c^{'}$).

16.2.3 **Installation.** Enlarged bases formed either by compacting concrete or driving a precast base shall be formed in or driven into granular soils. Piles shall be constructed in the same manner as successful prototype test piles driven for the project. Pile shafts extending through peat or other organic soil shall be encased in a permanent steel casing. Where a cased shaft is used, the shaft shall be adequately reinforced to resist column action or the annular space around the pile shaft shall be filled sufficiently to reestablish lateral support by the soil. Where pile heave occurs, the pile shall be replaced unless it is demonstrated that the pile is undamaged and capable of carrying twice its design load.

16.2.4 **Load-bearing capacity.** Pile load-bearing capacity shall be verified by load tests in accordance with Section 14.8.3.

16.2.5 **Concrete cover.** The minimum concrete cover shall be 70 mm for uncased shafts and 25 mm for cased shafts.

SECTION 16.3
DRILLED OR AUGERED UNCASED PILES

16.3.0 Drilled or augered uncased piles shall conform to Sections 16.3.1 through 16.3.5.

16.3.1 **Allowable stresses.** The allowable design stress in the concrete of drilled uncased piles shall not exceed 33 percent of the 28-day specified compressive strength

($f_c^{'}$). The allowable design stress in the concrete of augered cast-in-place piles shall not exceed 25 percent of the 28-day specified compressive strength ($f_c^{'}$). The allowable compressive stress of reinforcement shall not exceed 34 percent of the yield strength of the steel or (175 MPa).

16.3.2 **Dimensions.** The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 300 mm.

Exception: The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation are under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved construction documents.

16.3.3 **Installation.** Where pile shafts are formed through unstable soils and concrete is placed in an open-drilled hole, a steel liner shall be inserted in the hole prior to placing the concrete. Where the steel liner is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the liner at a sufficient height to offset any hydrostatic or lateral soil pressure.

Where concrete is placed by pumping through a hollow-stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. The auger shall be withdrawn in a continuous manner in increments of about 300 mm each. Concreting pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete volumes shall be measured to ensure that the volume of concrete placed in each pile is equal to or greater than the theoretical volume of the hole created by the auger. Where the installation process of any pile is interrupted or a loss of concreting pressure occurs, the pile shall be redrilled to 1500 mm below the elevation of the tip of the auger when the installation was interrupted or concrete pressure was lost and reformed. Augered cast-in-place piles shall not be installed within six pile diameters center to center of a pile filled with concrete less than 12 hours old, unless approved by the building official. If the concrete level in any completed pile drops during installation of an adjacent pile, the pile shall be replaced.

16.3.4 **Reinforcement.** For piles installed with a hollow-stem auger, where full-length longitudinal steel reinforcement is placed without lateral ties, the reinforcement shall be placed through ducts in the auger prior to filling the pile with concrete. All pile reinforcement shall have a concrete cover of not less than 70 mm.

Exception: Where physical constraints do not allow the placement of the longitudinal reinforcement prior to filling the pile with concrete or where partial-length longitudinal reinforcement is placed without lateral ties, the reinforcement is allowed to be placed after the piles are completely concreted but while concrete is still in a semifluid state.

16.3.5 **Reinforcement in seismic design category C and D.** Where a structure is assigned to Seismic Design Category C and D in accordance with Chapters 9 through 16, SBC 301, the corresponding requirements of Sections 16.1.2.1 and 16.1.2.2 shall be met.

SECTION 16.4
DRIVEN UNCASED PILES

16.4.0 Driven uncased piles shall conform to Sections 16.4.1 through 16.4.4.

16.4.1 **Allowable stresses.** The allowable design stress in the concrete shall not exceed 25 percent of the 28-day specified compressive strength (f_c') applied to a cross-sectional area not greater than the inside area of the drive casing or mandrel.

16.4.2 **Dimensions.** The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 300 mm.

Exception: The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation is under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved design.

16.4.3 **Installation.** Piles shall not be driven within six pile diameters center to center in granular soils or within one-half the pile length in cohesive soils of a pile filled with concrete less than 48 hours old unless approved by the building official. If the concrete surface in any completed pile rises or drops, the pile shall be replaced. Piles shall not be installed in soils that could cause pile heave.

16.4.4 **Concrete cover.** Pile reinforcement shall have a concrete cover of not less than 70 mm, measured from the inside face of the drive casing or mandrel.

SECTION 16.5
STEEL-CASED PILES

16.5.0 Steel-cased piles shall comply with the requirements of Sections 16.5.1 through 16.5.4.

16.5.1 **Materials.** Pile shells or casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water tight to exclude any foreign materials during the placing of concrete. Steel shells shall have a sealed tip with a diameter of not less than 200 mm.

16.5.2 **Allowable stresses.** The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f_c'). The allowable concrete compressive stress shall be 0.40 (f_c') for that portion of the pile meeting the conditions specified in Sections 16.5.2.1 through 16.5.2.4.

16.5.2.1 **Shell thickness.** The thickness of the steel shell shall not be less than manufacturer's standard gage No. 14 gage 1.75 mm minimum.

16.5.2.2 **Shell type.** The shell shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.

16.5.2.3 **Strength.** The ratio of steel yield strength (f_y) to 28-day specified compressive strength ($f_c^{'}$) shall not be less than six.

16.5.2.4 **Diameter.** The nominal pile diameter shall not be greater than 400 mm.

16.5.3 **Installation.** Steel shells shall be mandrel driven their full length in contact with the surrounding soil.

The steel shells shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. A pile shall not be driven within four and one-half average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the building official. Concrete shall not be placed in steel shells within heave range of driving.

16.5.4 **Reinforcement.** Reinforcement shall not be placed within 25 mm of the steel shell. Reinforcing shall be required for unsupported pile lengths or where the pile is designed to resist uplift or unbalanced lateral loads.

16.5.4.1 **Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C and D in accordance with Chapters 9 through 16, SBC 301, the reinforcement requirements for drilled or augered uncased piles in Section 16.3.5 shall be met.

Exception: A spiral-welded metal casing of a thickness not less than manufacturer's standard gage No. 14 gage 1.75 mm is permitted to provide concrete confinement in lieu of the closed ties or equivalent spirals required in an uncased concrete pile. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

SECTION 16.6
CONCRETE-FILLED STEEL PIPE AND TUBE PILES

16.6.0 Concrete-filled steel pipe and tube piles shall conform to the requirements of Sections 16.6.1 through 16.6.5.

16.6.1 **Materials.** Steel pipe and tube sections used for piles shall conform to ASTM A 252 or ASTM A 283. Concrete shall conform to Section 16.1.1. The maximum coarse aggregate size shall be 20 mm.

16.6.2 **Allowable stresses.** The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength ($f_c^{'}$). The allowable design compressive stress in the steel shall not exceed 35 percent of the minimum specified yield strength of the steel (f_y), provided f_y shall not be assumed greater than 250 MPa for computational purposes.

Exception: Where justified in accordance with Section 16.2, the allowable stresses are permitted to be increased to 0.50 f_y.

16.6.3 **Minimum dimensions.** Piles shall have a nominal outside diameter of not less than 200 mm and a minimum wall thickness in accordance with Section 15.2.5. For mandrel-driven pipe piles, the minimum wall thickness shall be 2.5 mm.

16.6.4 **Reinforcement.** Reinforcement steel shall conform to Section 16.1.2. Reinforcement shall not be placed within 25 mm of the steel casing.

16.6.4.1 **Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C and D in accordance with SBC 301 Chapters 9 through 16, the following shall apply. Minimum reinforcement no less than 0.01 times the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment anchorage into the pile cap, but not less than the tension development length of the reinforcement. The wall thickness of the steel pipe shall not be less than 5 mm.

16.6.5 **Placing concrete.** The placement of concrete shall conform to Section 16.1.3.

SECTION 16.7
CAISSON PILES

16.7.0 Caisson piles shall conform to the requirements of Sections 16.7.1 through 16.7.6.

16.7.1 **Construction.** Caisson piles shall consist of a shaft section of concrete-filled pipe extending to bedrock with an uncased socket drilled into the bedrock and filled with concrete. The caisson pile shall have a full-length structural steel core or a stub core installed in the rock socket and extending into the pipe portion a distance equal to the socket depth.

16.7.2 **Materials.** Pipe and steel cores shall conform to the material requirements in Section 15.2. Pipes shall have a minimum wall thickness of 10 mm and shall be fitted with a suitable steel-driving shoe welded to the bottom of the pipe. Concrete shall have a 28-day specified compressive strength ($f_c^{'}$) of not less than 28 MPa. The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 100 mm to 150 mm.

16.7.3 **Design.** The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the caisson pile with a minimum safety factor of two, but the depth shall not be less than the outside diameter of the pipe. The design of the rock socket is permitted to be predicted on the sum of the allowable load-bearing pressure on the bottom of the socket plus bond along the sides of the socket. The minimum outside diameter of the caisson pile shall be 450 mm, and the diameter of the rock socket shall be approximately equal to the inside diameter of the pile.

16.7.4 **Structural core.** The gross cross-sectional area of the structural steel core shall not exceed 25 percent of the gross area of the caisson. The minimum clearance between the structural core and the pipe shall be 50 mm. Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

16.7.5 **Allowable stresses.** The allowable design compressive stresses shall not exceed the following: concrete, 0.33 $f_c^{'}$; steel pipe, 0.35 f_y and structural steel core, 0.50 f_y.

16.7.6 **Installation.** The rock socket and pile shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket. Concrete shall not be placed through water except where a tremie or other approved method is used.

SECTION 16.8
COMPOSITE PILES

16.8.1 Composite piles shall conform to the requirements of Sections 16.8.2 through 16.8.5.

16.8.2 **Design.** Composite piles consisting of two or more approved pile types shall be designed to meet the conditions of installation.

16.8.3 **Limitation of load.** The maximum allowable load shall be limited by the capacity of the weakest section incorporated in the pile.

16.8.4 **Splices.** Splices between concrete and steel sections shall be designed to prevent separation both before and after the concrete portion has set, and to ensure the alignment and transmission of the total pile load. Splices shall be designed to resist uplift caused by upheaval during driving of adjacent piles, and shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section.

16.8.5 **Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C and D in accordance with SBC 301 Chapters 9 through 16, the following shall apply. Where concrete and steel are used as part of the pile assembly, the concrete reinforcement shall comply with that given in Sections 16.1.2.1 and 16.1.2.2 or the steel section shall comply with Section 15.2.5 or 16.6.4.1.

CHAPTER 17
PIER FOUNDATIONS

SECTION 17.1
GENERAL

17.1.1 Isolated and multiple piers used as foundations shall conform to the requirements of Sections 17.2 through 17.10, as well as the applicable provisions of Chapter 14.

SECTION 17.2
LATERAL DIMENSIONS AND HEIGHT

17.2.1 The minimum dimension of isolated piers used as foundations shall be 600 mm, and the height shall not exceed 12 times the least horizontal dimension.

SECTION 17.3
MATERIALS

17.3.1 Concrete shall have a 28-day specified compressive strength ($f_c^{'}$) of not less than 20 MPa. Where concrete is placed through a funnel hopper at the top of the pier, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 100 mm and not more than 150 mm. Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

SECTION 17.4
REINFORCEMENT

17.4.1 Except for steel dowels embedded 1500 mm or less in the pier, reinforcement where required shall be assembled and tied together and shall be placed in the pier hole as a unit before the reinforced portion of the pier is filled with concrete.

Exception: Reinforcement is permitted to be wet set and the 70 mm concrete cover requirement be reduced to 50 mm for Group R-3 and U occupancies not exceeding two stories of light-frame construction, provided the construction method can be demonstrated to the satisfaction of the building official.

Reinforcement shall conform to the requirements of Section 16.1.2.1 and 16.1.2.2.

Exceptions:

1. Isolated piers supporting posts of Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than a minimum of one Dia 12 mm bar, without ties or spirals, when detailed so the pier is not subject to lateral loads and the soil is determined to be of adequate stiffness.
2. Isolated piers supporting posts and bracing from decks and patios appurtenant to Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than one Dia 12 mm bar, without ties or spirals, when the lateral load, *E*, to the top of the pier does not exceed 900 N and the soil is determined to be of adequate stiffness.

3. Piers supporting the concrete foundation wall of Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than two Dia 12 mm bars, without ties or spirals, when it can be shown the concrete pier will not rupture when designed for the maximum seismic load, E_m, and the soil is determined to be of adequate stiffness.
4. Closed ties or spirals where required by Section 16.1.2.2 are permitted to be limited to the top 1 m of the piers 3 m or less in depth supporting Group R-3 and U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

SECTION 17.5
CONCRETE PLACEMENT

17.5.1 Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pier, the concrete shall not be chutted directly into the pier but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pier.

SECTION 17.6
BELLED BOTTOMS

17.6.1 Where pier foundations are belled at the bottom, the edge thickness of the bell shall not be less than that required for the edge of footings. Where the sides of the bell slope at an angle less than 60 degrees (1 rad) from the horizontal, the effects of vertical shear shall be considered.

SECTION 17.7
MASONRY

17.7.1 Where the unsupported height of foundation piers exceeds six times the least dimension, the allowable working stress on piers of unit masonry shall be reduced in accordance with SBC 305.

SECTION 17.8
CONCRETE

17.8.1 Where adequate lateral support is not provided, piers shall be constructed of reinforced concrete, and shall conform to the requirements for columns in SBC 304. Where adequate lateral support is furnished by the surrounding materials as defined in Section 14.9, piers are permitted to be constructed of reinforced concrete, and the requirements of SBC 304 for bearing on concrete shall apply.

SECTION 17.9
STEEL SHELL

17.9.1 Where concrete piers are entirely encased with a circular steel shell, and the area of the shell steel is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 14.17. Horizontal joints in the shell shall be spliced to comply with Section 14.7.

SECTION 17.10
DEWATERING

17.10.1 Where piers are carried to depths below water level, the piers shall be constructed by a method that will provide accurate preparation and inspection of the bottom, and the depositing or construction of sound concrete or other masonry in the dry.

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Gratitude

In appreciation and gratitude
to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence
& Aviation and Inspector General

For their continuous support and gracious consideration,
the Saudi Building Code National Committee (SBCNC)
is honored to present the first issue of
the Saudi Building Code (SBC).

Saudi Building Code Requirements

201	Architectural	
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PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11^{th} June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Requirements for Concrete Structures (SBC 304) were developed based on ICC code in addition to American Concrete Institute (ACI) materials. ACI grants permission to the SBCNC to include ACI materials in the SBC, and ACI is not responsible for any modifications or changes that SBCNC has made to accommodate local conditions.

On the basis of (ISO) International Standards Organization/Technical Committee No. 71 evaluation of the SBC and a letter ballot member bodies of ISO/TC 71, Saudi Building Code Concrete Structures (SBC 304) has been approved to be added to the clause A.2 of ISO 19338, this implies that SBC 304 is deemed to satisfy ISO 19338.

The development process of SBC 304 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made on ACI, such as Durability Requirements, the simplified methods for the design of two-way slab system of Appendix C, expanding some topics such as Hot Weather, taking into considerations the properties of local material such as the Saudi steel and the engineering level for those involved in the building sector.

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CHAPTER 1
GENERAL REQUIREMENTS

SECTION 1.1
SCOPE

1.1.1 The Saudi Building Code for Concrete Structures referred to as SBC 304, provides minimum requirements for structural concrete design and construction. For structural concrete, the specified compressive strength as defined in Section 5.1 shall not be less than 20 MPa. No maximum specified compressive strength shall apply unless restricted by a specific code provision.

1.1.2 SBC 304 shall govern in all matters pertaining to design and construction of structural concrete.

1.1.3 SBC 304 shall govern in all matters pertaining to design, construction, and material properties wherever SBC 304 is in conflict with requirements contained in other standards referenced in SBC 304.

1.1.4 For special structures, such as arches, tanks, reservoirs, bins and silos, blast-resistant structures, and chimneys, provisions of SBC 304 shall govern where applicable.

1.1.5 SBC 304 does not govern design and installation of portions of concrete piles, drilled piers, and caissons embedded in ground except for structures in regions of high seismic risk or assigned to high seismic performance or design categories. See Section 21.10.4 for requirements for concrete piles, drilled piers, and caissons in structures in regions of high seismic risk or assigned to high seismic performance or design categories.

1.1.6 SBC 304 does not govern design and construction of soil-supported slabs, unless the slab transmits vertical loads or lateral forces from other portions of the structure to the soil.

1.1.7 Concrete on steel form deck

1.1.7.1 Design and construction of structural concrete slabs cast on stay-in-place, noncomposite steel form deck are governed by SBC 304.

1.1.7.2 SBC 304 does not govern the design of structural concrete slabs cast on stay-in-place, composite steel form deck. Concrete used in the construction of such slabs shall be governed by Parts 1, 2, and 3 of this requirement, where applicable.

1.1.8 Special provisions for earthquake resistance

1.1.8.1 In regions of low seismic risk where the seismic design loads are computed using provisions for ordinary concrete systems, in accordance with Table 10.2 of SBC 301, provisions of Chapter 21 shall not apply.

1.1.8.2 In seismic regions where the seismic design loads are computed using provisions for intermediate or special concrete systems, in accordance with

Table 10.2 of SBC 301, provisions of Chapters 1 through 18 shall apply except as modified by the provisions of Chapter 21. See Section 21.2.1

1.1.8.3 The seismic design category of a structure, shall be regulated by SBC 301.

SECTION 1.2
CONSTRUCTION DOCUMENTS

1.2.1 Copies of design drawings, typical details, and specifications for all structural concrete construction shall bear the seal of a registered structural engineer. These drawings, details, and specifications shall show:

(a) Name and date of issue of code and supplement to which design conforms;

(b) Live load and other loads used in design;

(c) Specified compressive strength of concrete at stated ages or stages of construction for which each part of structure is designed;

(d) Specified strength or grade of reinforcement;

(e) Size and location of all structural elements, concrete cover, reinforcement, and anchors;

(f) Provision for dimensional changes resulting from creep, shrinkage, and temperature;

(g) Magnitude and location of prestressing forces;

(h) Anchorage length of reinforcement and location and length of lap splices;

(i) Type and location of mechanical and welded splices of reinforcement;

(j) Details and location of all contraction or isolation joints;

(k) Minimum concrete compressive strength at time of post-tensioning;

(l) Stressing sequence for post-tensioning tendons;

(m) Statement if slab on grade is designed as a structural diaphragm, see Section 21.10.3.4.

1.2.2 Calculations pertinent to design shall be filed with the drawings. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. Model analysis shall be permitted to supplement calculations.

1.2.3 Building official means the officer or other designated authority charged with the administration and enforcement of SBC 304, or his duly authorized representative.

SECTION 1.3
INSPECTION

1.3.1 The special inspection of concrete elements of buildings and structures and concreting operations shall be as required by SBC 302.

SECTION 1.4
APPROVAL OF SPECIAL SYSTEMS OF DESIGN OR CONSTRUCTION

Sponsors of any system of design or construction within the scope of SBC 304, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by SBC 304, shall have the right to present the data on which their design is based to the building official or to a board of examiners appointed by the building official. This board shall be composed of competent engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of SBC 304. These rules when approved by the building official and promulgated shall be of the same force and effect as the provisions of SBC 304.

CHAPTER 2
DEFINITIONS

SECTION 2.1
DEFINITIONS

The following terms are defined for general use in SBC 304. Specialized definitions appear in individual chapters.

Admixture. Material other than water, aggregate, or hydraulic cement, used as an ingredient of concrete and added to concrete before or during its mixing to modify its properties.

Aggregate. Granular material, such as sand, gravel, crushed stone, and iron blast-furnace slag, used with a cementing medium to form a hydraulic cement concrete or mortar.

***Aggregate, lightweight*.** Aggregate with a dry, loose weight of 1100 kg/m^3 or less.

Anchorage device. In post-tensioning, the hardware used for transferring a post-tensioning force from the prestressing steel to the concrete.

Anchorage zone. In post-tensioned members, the portion of the member through which the concentrated prestressing force is transferred to the concrete and distributed more uniformly across the section. Its extent is equal to the largest dimension of the cross section. For anchorage devices located away from the end of a member, the anchorage zone includes the disturbed regions ahead of and behind the anchorage devices.

Basic monostrand anchorage device. Anchorage device used with any single strand or a single 16 mm or smaller diameter bar that satisfies 18.21.1 and the anchorage device requirements of ACI 423.6, "Specification for Unbonded Single-Strand Tendons."

Basic multistrand anchorage device. Anchorage device used with multiple strands, bars, or wires, or with single bars larger than 16 mm diameter, that satisfies 18.21.1 and the bearing stress and minimum plate stiffness requirements of AASHTO Bridge Specifications, Division I, Articles 9.21.7.2.2 through 9.21.7.2.4.

Bonded tendon. Tendon in which prestressing steel is bonded to concrete either directly or through grouting.

Building official. See 1.2.3.

Cementitious materials. Materials as specified in Chapter 3, which have cementing value when used in concrete either by themselves, such as portland cement, blended hydraulic cements, and expansive cement, or such materials in combination with fly ash, other raw or calcined natural pozzolans, silica fume, and/or ground granulated blast-furnace slag.

Column. Member with a ratio of height-to-least lateral dimension exceeding 3 used primarily to support axial compressive load.

Composite concrete flexural members. Concrete flexural members of precast or cast-in-place concrete elements, or both, constructed in separate placements but so interconnected that all elements respond to loads as a unit.

Compression-controlled section. *A* cross section in which the net tensile strain in the extreme tension steel at nominal strength is less than or equal to the compression-controlled strain limit.

Compression-controlled strain limit. The net tensile strain at balanced strain conditions. See 10.3.3.

Concrete. Mixture of portland cement or any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixtures.

Concrete, specified compressive strength of (f_c'). Compressive strength of concrete used in design and evaluated using 150 × 300 mm cylindrical specimens in accordance with provisions of Chapter 5, expressed in Megapascals (MPa). Whenever the quantity f_c' is under a radical sign, square root of numerical value only is intended, and result has units of Megapascals (MPa). See Section 5.1.2.

Concrete, structural lightweight. Concrete containing lightweight aggregate that conforms to 3.3 and has an air-dry unit weight as determined by "Test Method for Unit Weight of Structural Lightweight Concrete" (ASTM C 567), not exceeding 1850 kg/m^3. In SBC 304, a lightweight concrete without natural sand is termed "all-lightweight concrete" and lightweight concrete in which all of the fine aggregate consists of normal weight sand is termed "sand-lightweight concrete."

Contraction joint. Formed, sawed, or tooled groove in a concrete structure to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure.

Curvature friction. Friction resulting from bends or curves in the specified prestressing tendon profile.

Deformed reinforcement. Deformed reinforcing bars, bar mats, deformed wire, welded plain wire fabric, and welded deformed wire fabric conforming to 3.5.3.

Development length. Length of embedded reinforcement required to develop the design strength of reinforcement at a critical section. See 9.3.3.

Duct. *A* conduit (plain or corrugated) to accommodate prestressing steel for post-tensioned installation. Requirements for post-tensioning ducts are given in 18.17.

Effective depth of section (d). Distance measured from extreme compression fiber to centroid of tension reinforcement.

Effective prestress. Stress remaining in prestressing steel after all losses have occurred.

Embedment length. Length of embedded reinforcement provided beyond a critical section.

Extreme tension steel. The reinforcement (prestressed or nonprestressed) that is the farthest from the extreme compression fiber.

Isolation joint. *A* separation between adjoining parts of a concrete structure, usually a vertical plane, at a designed location such as to interfere least with performance of the structure, yet such as to allow relative movement in three directions and avoid formation of cracks elsewhere in the concrete and through which all or part of the bonded reinforcement is interrupted.

Jacking force. In prestressed concrete, temporary force exerted by device that introduces tension into prestressing steel.

Load, dead. Dead weight supported by a member, as defined by the SBC 301 (without load factors).

Load, factored. Load, multiplied by appropriate load factors, used to proportion members by the strength design method of SBC 304. See 8.1.1 and 9.2.

Load, live. Live load specified by the SBC 301 (without load factors).

Load, service. Load specified by the SBC 301 (without load factors).

Modulus of elasticity. Ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material. See 8.5.

Moment frame. Frame in which members and joints resist forces through flexure, shear, and axial force. Moment frames shall be categorized as follows:

Intermediate moment frame. *A* cast-in-place frame complying with the requirements of 21.2.2.3 and 21.12 in addition to the requirements for ordinary moment frames.

Ordinary moment frame. A cast-in-place or precast concrete frame complying with the requirements of Chapters 1 through 18.

Special moment frame. A cast-in-place frame complying with the requirements of 21.2 through 21.5, or a precast frame complying with the requirements of 21.2 through 21.6. In addition, the requirements for ordinary moment frames shall be satisfied.

Net tensile strain. The tensile strain at nominal strength exclusive of strains due to effective prestress, creep, shrinkage, and temperature.

Pedestal. Upright compression member with a ratio of unsupported height to average least lateral dimension not exceeding 3.

Plain concrete. Structural concrete with no reinforcement or with less reinforcement than the minimum amount specified for reinforced concrete.

Plain reinforcement. Reinforcement that does not conform to definition of deformed reinforcement. See 3.5.4.

Post-tensioning. Method of prestressing in which prestressing steel is tensioned after concrete has hardened.

Precast concrete. Structural concrete element cast elsewhere than its final position in the structure.

Prestressed concrete. Structural concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.

Prestressing steel. High-strength steel element such as wire, bar, or strand, or a bundle of such elements, used to impart prestress forces to concrete.

Pretensioning. Method of prestressing in which prestressing steel is tensioned before concrete is placed.

Reinforced concrete. Structural concrete reinforced with no less than the minimum amounts of prestressing steel or nonprestressed reinforcement specified in Chapters 1 through 21 and Appendices A through C.

Reinforcement. Material that conforms to 3.5, excluding prestressing steel unless specifically included.

Registered design professional. An individual who is registered or licensed to practice the respective design profession in the Kingdom.

Reshores. Shores placed snugly under a concrete slab or other structural member after the original forms and shores have been removed from a larger area, thus requiring the new slab or structural member to deflect and support its own weight and existing construction loads applied prior to the installation of the reshores.

Sheathing. A material encasing prestressing steel to prevent bonding of the prestressing steel with the surrounding concrete, to provide corrosion protection, and to contain the corrosion inhibiting coating.

Shores. Vertical or inclined support members designed to carry the weight of the formwork, concrete, and construction loads above.

Span length - See 8.7.

Special anchorage device. Anchorage device that satisfies 18.15.1 and the standardized acceptance tests of AASHTO "Standard Specifications for Highway Bridges," Division II, Article 10.3.2.3.

Spiral reinforcement. Continuously wound reinforcement in the form of a cylindrical helix.

Splitting tensile strength (f_{ct}). Tensile strength of concrete determined in accordance with ASTM C 496 as described in "Specification for Lightweight aggregates for Structural Concrete" (ASTM C 330). See 5.1.4.

Stirrup. Reinforcement used to resist shear and torsion stresses in a structural member; typically bars, wires, or welded wire fabric (plain or deformed) either single leg or bent into L, U, or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. (The term "stirrups" is usually applied to lateral reinforcement in flexural members and the term ties to those in compression members.) See also *Tie*.

Strength, design. Nominal strength multiplied by a strength reduction factor ϕ. See 9.3.

Strength, nominal. Strength of a member or cross section calculated in accordance with provisions and assumptions of the strength design method of SBC 304 before application of any strength reduction factors. See 9.3.1.

Strength, required. Strength of a member or cross section required to resist factored loads or related internal moments and forces in such combinations as are stipulated in SBC 304. See Section 9.1.1.

Stress. Intensity of force per unit area.

Structural concrete. All concrete used for structural purposes including plain and reinforced concrete.

Structural walls. Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shear wall is a structural wall. Structural walls shall be categorized as follows:

> ***Intermediate precast structural wall.*** A wall complying with all applicable requirements of Chapters 1 through 18 in addition to 21.13.
> ***Ordinary reinforced concrete structural wall.*** A wall complying with the requirements of Chapters 1 through 18.
> ***Special precast structural wall.*** *A* precast wall complying with the requirements of 21.8. In addition, the requirements of ordinary reinforced concrete structural walls and the requirements of 21.2 shall be satisfied.
> ***Special reinforced concrete structural wall.*** A cast-in-place wall complying with the requirements of 21.2 and 21.7 in addition to the requirements for ordinary reinforced concrete structural walls.

Tendon. In pretensioned applications, the tendon is the prestressing steel. In post-tensioned applications, the tendon is a complete assembly consisting of anchorages, prestressing steel, and sheathing with coating for unbonded applications or ducts with grout for bonded applications.

Tension-controlled section. *A* cross section in which the net tensile strain in the extreme tension steel at nominal strength is greater than or equal to 0.005.

Tie. Loop of reinforcing bar or wire enclosing longitudinal reinforcement. A continuously wound bar or wire in the form of a circle, rectangle, or other polygon shape without re-entrant corners is acceptable. See also ***Stirrup***.

Transfer. Act of transferring stress in prestressing steel from jacks or pretensioning bed to concrete member.

Unbonded Tendon. Tendon in which the prestressing steel is prevented from bonding to the concrete and is free to move relative to the concrete. The prestressing force is permanently transferred to the concrete at the tendon ends by the anchorages only.

Wall. Member, usually vertical, used to enclose or separate spaces.

Wobble friction. In prestressed concrete, friction caused by unintended deviation of prestressing sheath or duct from its specified profile.

Yield strength. Specified minimum yield strength or yield point of reinforcement. Yield strength or yield point shall be determined in tension according to applicable ASTM standards as modified by 3.5 of SBC 304.

CHAPTER 3
MATERIALS

SECTION 3.0
NOTATION

f_y = specified yield strength of nonprestressed reinforcement, MPa.

SECTION 3.1
TESTS OF MATERIALS

3.1.1 The Building Official shall have the right to order testing of any materials used in concrete construction to determine if materials are of quality specified.

3.1.2 Tests of materials and of concrete shall be made in accordance with standards listed in 3.8.

3.1.3 A complete record of tests of materials and of concrete shall be retained by the inspector for 5 years after completion of the project, and made available for inspection during the progress of the work.

SECTION 3.2
CEMENTS

3.2.1 Cement shall conform to one of the following specifications:

(a) "Specification for Portland Cement" (ASTM C 150);

(b) "Specification for Blended Hydraulic Cements" (ASTM C 595M), excluding Types S and SA which are not intended as principal cementing constituents of structural concrete;

(c) "Specification for Expansive Hydraulic Cement" (ASTM C 845).

3.2.2 Cement used in the work shall correspond to that on which selection of concrete proportions was based. See 5.2.

SECTION 3.3
AGGREGATES

3.3.1 Concrete aggregates shall conform to one of the following specifications:

(a) "Specification for Concrete Aggregates" (ASTM C 33);

(b) "Specification for Lightweight Aggregates for Structural Concrete" (ASTM C 330).

Exception: Aggregates that have been shown by special test or actual service to produce concrete of adequate strength and durability and approved by the Building Official.

3.3.2 Nominal maximum size of coarse aggregate shall be not larger than:

(a) 1/5 the narrowest dimension between sides of forms, nor

(b) 1/3 the depth of slabs, nor

(c) 3/4 the minimum clear spacing between individual reinforcing bars or wires, bundles of bars, individual tendons, bundled tendons, or ducts.

These limitations shall not apply if, in the judgment of the engineer, workability and methods of consolidation are such that concrete can be placed without honey-combs or voids.

SECTION 3.4
WATER

3.4.1 Water used in mixing or curing concrete shall be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances deleterious to concrete or reinforcement.

3.4.2 Mixing water for prestressed concrete, reinforced concrete or for concrete that will contain aluminum embedments, including that portion of mixing water contributed in the form of free moisture on aggregates, shall not contain deleterious amounts of chloride ion. See Section 4.4.1.

3.4.3 Nonpotable water shall not be used in concrete unless the following are satisfied:

3.4.3.1 The following limits of in water for concrete (mixing or curing) shall not be exceeded: alkali carbonate and bicarbonate 1000 ppm, chlorides 1000 ppm, sulfates 3000 ppm, alkalis 600 ppm, and pH 4 (minimum).

3.4.3.2 Mortar test cubes made with nonpotable mixing water containing more than 2000 ppm of total dissolved solids shall have 7-day and 28-day strengths equal to at least 90 percent of strengths of similar specimens made with potable water. Strength test comparison shall be made on mortars, identical except for the mixing water, prepared and tested in accordance with ASTM C 109.

SECTION 3.5
STEEL REINFORCEMENT

3.5.1 Reinforcement shall be deformed reinforcement, except that plain reinforcement shall be permitted for spirals or prestressing steel; and reinforcement consisting of structural steel, steel pipe, or steel tubing shall be permitted as specified in SBC 304.

3.5.2 Welding of reinforcing bars shall not be considered as a recommended practice in SBC 304.

3.5.3 Deformed reinforcement

3.5.3.1 Deformed reinforcing bars shall conform to one of the following specifications:

(a) "Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement" ASTM A 615M;

(b) "Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete

Reinforcement" ASTM A 706M;

3.5.3.2 Deformed reinforcing bars with a specified yield strength f_y exceeding 420 MPa shall be permitted, provided f_y shall be the stress corresponding to a strain of 0.35 percent and the bars otherwise conform to one of the ASTM specifications listed in 3.5.3.1. See 9.4.

3.5.3.3 Bar mats for concrete reinforcement shall conform to "Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement" (ASTM A 184M). Reinforcing bars used in bar mats shall conform to one of the specifications listed in Section 3.5.3.1.

3.5.3.4 Deformed wire for concrete reinforcement shall conform to "Specification for Steel Wire, Deformed, for Concrete Reinforcement" (ASTM A 496), except that wire shall not be smaller than size D4 and for wire with a specified yield strength f_y exceeding 420 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent if the yield strength specified in the design exceeds 420 MPa.

3.5.3.5 Welded plain wire fabric for concrete reinforcement shall conform to "Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement" (ASTM A 185), except that for wire with a specified yield strength f_y exceeding 420 MPa, f_y, shall be the stress corresponding to a strain of 0.35 percent if the yield strength specified in the design exceeds 420 MPa. Welded intersections shall not be spaced farther apart than 300 mm in direction of calculated stress, except for wire fabric used as stirrups in accordance with Section 12.13.2.

3.5.3.6 Welded deformed wire fabric for concrete reinforcement shall conform to "Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement" (ASTM A 497), except that for wire with a specified yield strength f_y exceeding 420 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent if the yield strength specified in the design exceeds 420 MPa. Welded intersections shall not be spaced farther apart than 400 mm in direction of calculated stress, except for wire fabric used as stirrups in accordance with 12.13.2.

3.5.3.7 Galvanized reinforcing bars shall comply with "Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement" (ASTM A 767M). Epoxy-coated reinforcing bars shall comply with "Specification for Epoxy-Coated Reinforcing Steel Bars" (ASTM A 775M) or with "Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars" (ASTM A 934M). Bars to be galvanized or epoxy-coated shall conform to one of the specifications listed in 3.5.3.1.

3.5.3.8 Epoxy-coated wires and welded wire fabric shall comply with "Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement" (ASTM A 884M). Wires to be epoxy-coated shall conform to 3.5.3.4 and welded wire fabric to be epoxy-coated shall conform to 3.5.3.5 or 3.5.3.6.

3.5.4 Plain reinforcement

3.5.4.1 Plain bars for spiral reinforcement shall conform to the specification listed in 3.5.3.1(a) or (b).

3.5.4.2 Plain wire for spiral reinforcement shall conform to "Specification for Steel Wire, Plain, for Concrete Reinforcement" (ASTM A 82), except that for wire with a specified yield strength f_y exceeding 420 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent if the yield strength specified in the design exceeds 420 MPa.

3.5.5 Prestressing steel

3.5.5.1 Steel for prestressing shall conform to one of the following specifications:

(a) Wire conforming to "Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete" (ASTM A 421);

(b) Low-relaxation wire conforming to "Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete" including Supplement "Low-Relaxation Wire" (ASTM A 421);

(c) Strand conforming to "Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete" (ASTM A 416M);

(d) Bar conforming to "Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete" (ASTM A 722).

3.5.5.2 Wire, strands, and bars not specifically listed in ASTM A 421, A 416M, or A 722 are allowed provided they conform to minimum requirements of these specifications and do not have properties that make them less satisfactory than those listed in ASTM A 421, A 416M, or A 722.

3.5.6 Structural steel, steel pipe, or tubing

3.5.6.1 Structural steel used with reinforcing bars in composite compression members meeting requirements of 10.16.7 or 10.16.8 shall conform to one of the following specifications:

(a) "Specification for Carbon Structural Steel" (ASTM A 36M);

(b) "Specification for High-Strength Low-Alloy Structural Steel" (ASTM A 242M);

(c) "Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel" (ASTM A 572M);

(d) “Specification for High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in (100 mm) Thick” (ASTM A 588M).

3.5.6.2 Steel pipe or tubing for composite compression members composed of a steel encased concrete core meeting requirements of 10.16.6 shall conform to one of the following specifications:

(a) Grade B of "Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless" (ASTM A 53);

(b) "Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes" (ASTM A 500);

(c) "Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing" (ASTM A 501).

SECTION 3.6
ADMIXTURE

3.6.1 Admixtures to be used in concrete shall be subject to prior approval by the engineer.

3.6.2 An admixture shall be shown capable of maintaining essentially the same composition and performance throughout the work as the product used in establishing concrete proportions in accordance with 5.2.

3.6.3 Calcium chloride or admixtures containing chloride from other than impurities from admixture ingredients shall not be used in prestressed concrete, in concrete containing embedded aluminum, or in concrete cast against stay-in-place galvanized steel forms. See Section 4.3.2 and 4.4.1.

3.6.4 Air-entraining admixtures shall conform to "Specification for Air-Entraining Admixtures for Concrete" (ASTM C 260).

3.6.5 Water-reducing admixtures, retarding admixtures, accelerating admixtures, water-reducing and retarding admixtures, and water-reducing and accelerating admixtures shall conform to "Specification for Chemical Admixtures for Concrete" (ASTM C 494) or "Specification for Chemical Admixtures for Use in Producing Flowing Concrete" (ASTM C 1017).

3.6.6 Fly ash or other pozzolans used as admixtures shall conform to "Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete" (ASTM C 618).

3.6.7 Ground granulated blast-furnace slag used as an admixture shall conform to "Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars" (ASTM C 989).

3.6.8 Admixtures used in concrete containing ASTM C 845 expansive cements shall be compatible with the cement and produce no deleterious effects.

3.6.9 Silica fume used as an admixture shall conform to ASTM C 1240.

SECTION 3.7
STORAGE OF MATERIALS

3.7.1 Cementitious materials and aggregates shall be stored in such manner as to prevent deterioration or intrusion of foreign matter.

3.7.2 Any material that has deteriorated or has been contaminated shall not be used for concrete.

SECTION 3.8
REFERENCED STANDARDS

3.8.1 The following Standards of the American Society for Testing and Materials are declared to be part of SBC 304 as if fully set forth herein:

A 36/ A 36M-00a	Standard Specification for Carbon Structural Steel
A 53/ A 53M-99b	Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless
A 82-97a	Standard Specification for Steel Wire, Plain, for Concrete Reinforcement
A 108-99	Standard Specification for Steel Bars, Carbon, Cold-Finished, Standard Quality
A 184/ A 184M-96	Standard Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement
A 185-97	Standard Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement
A 242/ A 242M-00a	Standard Specification for High-Strength Low-Alloy Structural Steel
A 307-97	Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength
A 416/ A 416M-99	Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
A 421/ A 421 M-98a	Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
A 496-97a	Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement
A 497-99	Standard Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement
A 500-99	Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
A 501-99	Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
A 572/ A 572M-00	Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
A 588/ A 588M-00	Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in. (100 mm) Thick
A 615/ A 615M-00	Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A 706/ A 706M-00	Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A 722/ A 722M-98	Standard Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete
A 767/ A 767M-00b	Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement
A 775/	Standard Specification for Epoxy-Coated Steel Reinforcing

A 775M-00	Bars
A 884/ A 884M-99	Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement
A 934/ A 934M-00	Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars
C 31/ C 31 M-98	Standard Practice for Making and Curing Concrete Test Specimens in the Field
C 33-99a	Standard Specification for Concrete Aggregates
C 39/ C 39M-99	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 42/ C 42M-99	Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 94/ C 94M-00	Standard Specification of Ready-Mixed Concrete
C109/ C 109M-99	Standard Test Method for Compress Strength of Hydraulic Cement Mortars (Using 50-mm Cube Specimens)
C 144-99	Standard Specification for Aggregate for Masonry Mortar
C 150-99a	Standard Specification for Portland Cement
C 172-99	Standard Practice for Sampling Freshly-Mixed Concrete
C 192/ C 192M-98	Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory
C 260-00	Standard Specification for Air-Entraining Admixtures for Concrete
C 330-99	Standard Specification for Lightweight Aggregates for Structural Concrete
C 494/ C 494M-99a	Standard Specification for Chemical Admixtures for Concrete
C 496-96	Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C567-99a	Standard Test Method for Density of Structural Lightweight Concrete
C 595-00	Standard Specification for Blended Hydraulic Cements
C 618-99	Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Concrete
C 685-98a	Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing
C 845-96	Standard Specification for Expansive Hydraulic Cement
C 989-99	Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars
C 1017/	Standard Specification for Chemical Admixtures for Use in

C 1017M-98	Producing Flowing Concrete
C 1218/ C 1218M-99	Standard Test Method for Water-Soluble Chloride in Mortar and Concrete
C 1240-00	Standard Specification for Use of Silica Fume as a Mineral Admixture in Hydraulic Cement Concrete, Mortar, and Grout

3.8.2 "Structural Welding Code-Reinforcing Steel" (ANSI/AWS D1.4-98) of the American Welding Society is declared to be part of SBC 304 as if fully set forth herein.

3.8.3 "Specification for Unbonded Single Strand Tendons (ACI 423.6-01) and Commentary (423.6R-01)" is declared to be part of SBC 304 as if fully set forth herein.

3.8.4 Articles 9.21.7.2 and 9.21.7.3 of Division I and Article 10.3.2.3 of Division II of AASHTO "Standard Specification for Highway Bridges" (AASHTO 16th Edition, 1996) are declared to be a part of SBC 304 as if fully set forth herein.

3.8.5 "Evaluating the Performance of Post-Installed Mechanical Anchors in Concrete (ACI 355.2-01)" is declared to be part of SBC 304 as if fully set forth herein, for the purpose cited in Appendix D.

3.8.6 "Structural Welding Code-Steel (AWS D 1.1:2000)" of the American Welding Society is declared to be part of SBC 304 as if fully set forth herein.

3.8.7 "Acceptance Criteria for Moment Frames Based on Structural Testing (ACI T1.1-01)," is declared to be part of SBC 304 as if fully set forth herein.

CHAPTER 4
DURABILITY REQUIREMENTS

SECTION 4.0
NOTATION

f'_c = specified compressive strength of concrete, MPa

f'_{cr} = required average compressive strength, MPa

SECTION 4.1
WATER-CEMENTITIOUS MATERIALS RATIO

4.1.1 The water-cementitious materials ratios specified in Tables 4.3.1 and 4.4.2 shall be calculated using the weight of cement meeting ASTM C 150, C 595M or C 845 plus the weight of fly ash or other pozzolans meeting ASTM C 618, slag meeting ASTM C 989 and silica fume meeting ASTM C 1240, if any.

SECTION 4.2
FREEZING AND THAWING EXPOSURE

Not applicable in the Kingdom.

SECTION 4.3
SULFATE EXPOSURES

4.3.1 Concrete to be exposed to sulfate-bearing groundwater or soils shall conform to the requirements of Table 4.3.1 or shall be concrete prepared with a cement that provides sulfate resistance and that has a maximum water-cementitious materials ratio, minimum cementitious materials content and minimum compressive strength from Table 4.3.1.

TABLE 4.3.1 – REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-BEARING SOILS OR WATER

Sulfate exposure	Water soluble sulfate (SO_4) in soil, percent by weight	Sulfate (SO_4) in water, ppm	Cement type	Maximum water-cementitious materials ratio, by weight	Minimum cementitious materials content, kg/m^3	Minimum f'_c, MPa
Negligible	$0.00 \leq SO_4 < 0.10$	$0 \leq SO_4 < 150$	–	–	–	–
Moderate	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 < 1500$	II	0.50	330	28
Severe+	$0.20 \leq SO_4 \leq 2.00$	$1500 \leq SO_4 \leq 10{,}000$	V	0.45	350	30
Very severe+	$SO_4 > 2.00$	$SO_4 > 10{,}000$	V plus pozzolan++	0.45	350	30

+ If sulfate ions are associated with magnesium ions, supplementary protection, such as application of a barrier coating, is required.

++Pozzolan that conforms to relevant ASTM standards or that is shown to improve the sulfate resistance by service records should only be used.

SECTION 4.4
CORROSION PROTECTION OF REINFORCEMENT

4.4.1 For corrosion protection of reinforcement in concrete, maximum water-soluble chloride ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the concrete ingredients including water, aggregates, cementitious materials, and admixtures shall not exceed the limits of Table 4.4.1. When testing is performed to determine the water-soluble chloride ion content, test procedures shall conform to ASTM C 1218.

TABLE 4.4.1 – MAXIMUM CHLORIDE ION CONTENT FOR CORROSION PROTECTION OF REINFORCEMENT

Type of member	Maximum water-soluble chloride ion (cl^-) in concrete, percent by weight of cement*
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

* Determined according to ASTM C 1218.

4.4.2 If concrete with reinforcement will be exposed to chlorides from soil, groundwater, seawater, or spray from these sources, requirements of Table 4.4.2 for water-cementitious materials ratio, cementitious materials content, cement type and concrete strength, and the minimum cover over reinforcing steel requirements of 7.7 shall be satisfied. See 18.16 for unbonded tendons.

4.4.3 For the permanently submerged, tidal, splash and spray zones of marine structures, the requirements for very severe exposure in Table 4.4.2 shall be satisfied.

4.4.4 For concrete structures near to or on the coast and exposed to airborne salt but not in direct contact with seawater, the requirements for severe exposure in Table 4.4.2 shall be satisfied.

4.4.5 For superstructures in coastal areas and not directly exposed to airborne salt, the requirements for moderate exposure in Table 4.4.2 shall be satisfied.

TABLE 4.4.2 –
REQUIREMENTS FOR CONCRETE EXPOSED TO CHLORIDE-BEARING SOIL AND WATER

Chloride exposure	Water soluble chloride (cl^-) in soil, percent by weight	Water soluble chloride (cl^-) in water, ppm	Cement type	Maximum water-cementitious materials ratio	Minimum cementitious materials content, kg/m^3	Minimum f'_c, MPa
Negligible	Upto 0.05	Up to 500	–	–	–	–
Moderate	0.05 to 0.1	500 to 2,000	–	0.50	330	28
Severe	0.1 to 0.5	2,000 to 10,000	I	0.45	350	30
Very severe	More than 0.5	More than 10,000	I + pozzolan+	0.40	370	35

+Pozzolan that conforms to relevant standards shall only be used.

SECTION 4.5
SULFATE PLUS CHLORIDE EXPOSURES

4.5.1 If concrete is exposed to both chlorides and sulfates, the lowest applicable maximum water-cementitious materials ratio and highest minimum cementitious materials content of Tables 4.3.1 and 4.4.2 shall be selected. The corresponding highest f'_c shall be the governing value for quality control purposes. The cement type shall be the one required by Table 4.4.2.

SECTION 4.6
SABKHA EXPOSURES

4.6.1 Concrete structures exposed to sabkha shall meet the requirements for very severe exposure in Table 4.4.2, except that the water-cementitious materials ratio shall not be more than 0.35. In addition, the exposed surfaces shall be protected by appropriate means, such as tanking or epoxy-based coating.

SECTION 4.7
SALT WEATHERING

4.7.1 Concrete structures amenable to salt weathering shall be protected by applying an appropriate barrier coating.

CHAPTER 5
CONCRETE QUALITY, MIXING AND PLACING

SECTION 5.0
NOTATION

f_c' = specified compressive strength of concrete, MPa

f_{cr}' = required average compressive strength of concrete used as the basis for selection of concrete proportions, MPa

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, MPa

s = standard deviation, MPa

SECTION 5.1
GENERAL

5.1.1 Concrete shall be proportioned to provide an average compressive strength as prescribed in Section 5.3.2 and shall satisfy the durability criteria of Chapter 4. Concrete shall be produced to minimize the frequency of strengths below f_c', as prescribed in Section 5.6.3.3. For concrete designed and constructed in accordance with the code, f_c' shall not be less than 20 MPa (cylinder standard).

5.1.2 Requirements for f_c' shall be based on tests of 150 x 300 mm cylinders made and tested as prescribed in Section 5.6.3.

5.1.3 Unless otherwise specified, f_c' shall be based on 28-day tests. If other than 28 days, test age for f_c' shall be as indicated in design drawings or specifications.

5.1.4 Where design criteria in Section 9.5.2.3, 11.2, and 12.2.4 provide for use of a splitting tensile strength value of concrete, laboratory tests shall be made in accordance with "Specification for Lightweight Aggregates for Structural Concrete" (ASTM C 330) to establish value of f_{ct} corresponding to specified value of f_c'.

5.1.5 Splitting tensile strength tests shall not be used as a basis for field acceptance of concrete.

SECTION 5.2
SELECTION OF CONCRETE PROPORTIONS

5.2.1 Proportions of materials for concrete shall be established to provide:

(a) Workability and consistency to permit concrete to be worked readily into forms and around reinforcement under conditions of placement to be employed, without segregation or excessive bleeding;

(b) Resistance to special exposures as required by Chapter 4;

(c) Conformance with strength test requirements of 5.6.

5.2.2 Where different materials are to be used for different portions of proposed work, each combination shall be evaluated.

5.2.3 Concrete proportions shall be established in accordance with Seciton 5.3 or, alternatively, 5.4, and shall meet applicable requirements of Chapter 4.

SECTION 5.3
PROPORTIONING ON THE BASIS OF FIELD EXPERIENCE OR TRIAL MIXTURES OR BOTH

5.3.1 Standard deviation

5.3.1.1 Where a concrete production facility has test records, a standard deviation shall be established. Test records from which a standard deviation is calculated:

(a) Shall represent materials, quality control procedures, and conditions similar to those expected and changes in materials and proportions within the test records shall not have been more restricted than those for proposed work;

(b) Shall represent concrete produced to meet a specified strength or strengths f'_c within 7 MPa of that specified for proposed work;

(c) Shall consist of at least 30 consecutive tests or two groups of consecutive tests totaling at least 30 tests as defined in Section 5.6.2.3 (spanning over a period of not less than 45 days), except as provided in Section 5.3.1.2.

5.3.1.2 Where a concrete production facility does not have test records meeting requirements of 5.3.1.1, but does have a record based on 15 to 29 consecutive tests, a standard deviation shall be established as the product of the calculated standard deviation and modification factor of Table Section 5.3.1.2. To be acceptable, test record shall meet requirements (a) and (b) of Section 5.3.1.1, and represent only a single record of consecutive tests that span a period of not less than 45 calendar days.

TABLE 5.3.1.2
MODIFICATION FACTOR FOR STANDARD DEVIATION WHEN LESS THAN 30 TESTS ARE AVAILABLE

No. of tests*	Modification factor for standard deviation†
Less than 15	Use table 5.3.2.2
15	1.16
20	1.08
25	1.03
30 or more	1.00

* Interpolate for intermediate numbers of tests.
† Modified standard deviation to be used to determine required average strength f'_{cr} from 5.3.2.1.

5.3.2 Required average strength

5.3.2.1 Required average compressive strength f'_{cr} used as the basis for selection of concrete proportions shall be determined from Table 5.3.2.1 using the standard deviation calculated in accordance with Section 5.3.1.1 or 5.3.1.2.

TABLE 5.3.2.1
REQUIRED AVERAGE COMPRESSIVE STRENGTH WHEN DATA ARE AVAILABLE TO ESTABLISH A STANDARD DEVIATION

Specified compressive strength, f'_c, MPa	Required average compressive strength, f'_{cr}, MPa
$f'_c \leq 35$	Use the larger value computed from Eq. (5-1) and (5-2) $f'_{cr} = f'_c + 1.34s$ (5-1) $f'_{cr} = f'_c + 2.33s - 3.45$ (5-2)
Over 35	Use the larger value computed from Eq. (5-1) and (5-3) $f'_{cr} = f'_c + 1.34s$ (5-1) $f'_{cr} = 0.90f'_c + 2.33s$ (5-3)

5.3.2.2 When a concrete production facility does not have field strength test records for calculation of standard deviation meeting requirements of Section 5.3.1.1 or 5.3.1.2, required average strength f'_{cr} shall be determined from Table 5.3.2.2 and documentation of average strength shall be in accordance with requirements of 5.3.3.

TABLE 5.3.2.2
REQUIRED AVERAGE COMPRESSIVE STRENGTH WHEN DATA ARE NOT AVAILABLE TO ESTABLISH A STANDARD DEVIATION

Specified compressive strength, f'_c, MPa	**Required average compressive strength, f'_{cr}, MPa**
20 to 35	$f'_c + 8.5$
Over 35	$1.10f'_c + 5.0$

5.3.3 **Documentation of average strength.** Documentation that proposed concrete proportions will produce an average compressive strength equal to or greater than required average compressive strength (see 5.3.2) shall consist of a field strength test record, several strength test records, or trial mixtures.

5.3.3.1 When test records are used to demonstrate that proposed concrete proportions will produce the required average strength f'_{cr} (see 5.3.2), such records shall represent materials and conditions similar to those expected. Changes in materials, conditions, and proportions within the test records shall not have been more restricted than those for proposed work. For the purpose of documenting average strength potential, test records consisting of less than 30 but not less than 10 consecutive tests are acceptable provided test records encompass a period of time not less than 45 days. Required concrete proportions shall be permitted to be established by interpolation between the strengths and proportions of two or more test records, each of which meets other requirements of this section.

5.3.3.2 When an acceptable record of field test results is not available, concrete proportions established from trial mixtures meeting the following restrictions shall be permitted:

(a) Combination of materials shall be those for proposed work;

(b) Trial mixtures having proportions and consistencies required for proposed work shall be made using at least three different water-cementitious materials ratios or cementitious materials contents that will produce a range of strengths encompassing the required average strength f'_{cr} ;

(c) Trial mixtures shall be designed to produce a slump within ± 20 mm of maximum permitted, and for air-entrained concrete, within ± 0.5 percent of maximum allowable air content;

(d) For each water-cementitious materials ratio or cementitious materials content, at least three test cylinders for each test age shall be made and cured in accordance with "Method of Making and Curing Concrete Test Specimens in the Laboratory" (ASTM C 192). Cylinders shall be tested at 28 days or at test age designated for determination of f'_c.

(e) From results of cylinder tests a curve shall be plotted showing the relationship between water-cementitious materials ratio or cementitious materials content and compressive strength at designated test age;

(f) Maximum water-cementitious materials ratio or minimum cementitious materials content for concrete to be used in proposed work shall be that shown by the curve to produce the average strength required by 5.3.2, unless a lower water-cementitious materials ratio or higher strength is required by Chapter 4.

SECTION 5.4
PROPORTIONING WITHOUT FIELD EXPERIENCE OR TRIAL MIXTURES

5.4.1 If data required by 5.3 are not available, concrete proportions shall be based upon other experience or information, if approved by the registered design professional. The required average compressive strength f'_{cr} of concrete produced with materials similar to those proposed for use shall be at least 8.5 MPa greater than the specified compressive strength f'_c. This alternative shall not be used for specified compressive strengths greater than 35 MPa.

5.4.2 Concrete proportioned by this section shall conform to the durability requirements of Chapter 4 and to compressive strength test criteria of 5.6.

SECTION 5.5
AVERAGE STRENGTH REDUCTION

As data become available during construction, it shall be permitted to reduce the amount by which f'_{cr} must exceed the specified value of f'_c, provided:

(a) Thirty or more test results are available and average of test results exceeds that required by Section 5.3.2.1, using a standard deviation calculated in accordance with Section 5.3.1.1; or

(b) Fifteen to 29 test results are available and average of test results exceeds that required by Section 5.3.2.1 using a standard deviation calculated in accordance with Section 5.3.1.2; and

(c) Special exposure requirements of Chapter 4 are met.

SECTION 5.6
EVALUATION AND ACCEPTANCE OF CONCRETE

5.6.1 Concrete shall be tested in accordance with the requirements of Section 5.6.2 through 5.6.5. Qualified field testing technicians shall perform tests on fresh concrete at the job site, prepare specimens required for curing under field conditions, prepare specimens required for testing in the laboratory, and record the temperature of the fresh concrete when preparing specimens for strength tests. Qualified laboratory technicians shall perform all required laboratory tests.

5.6.2 Frequency of testing

5.6.2.1 Samples for strength tests of each class of concrete placed each day shall be taken not less than once a day, nor less than once for each 120 m^3 of concrete, nor less than once for each 500 m^2 of surface area for slabs or walls.

5.6.2.2 On a given project, if total volume of concrete is such that frequency of testing required by Section 5.6.2.1 would provide less than five strength tests for a given class of concrete, tests shall be made from at least five randomly selected batches or from each batch if fewer than five batches are used.

5.6.2.3 A strength test shall be the average of the strengths of two cylinders made from the same sample of concrete and tested at 28 days or at test age designated for determination of f'_c .

5.6.3 Laboratory-cured specimens

5.6.3.1 Samples for strength tests shall be taken in accordance with "Method of Sampling Freshly Mixed Concrete" (ASTM C 172).

5.6.3.2 Cylinders for strength tests shall be molded and laboratory-cured in accordance with "Practice for Making and Curing Concrete Test Specimens in the Field" (ASTM C 31) and tested in accordance with "Test Method for Compressive Strength of Cylindrical Concrete Specimens" (ASTM C 39).

5.6.3.3 Strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met:

(a) Every arithmetic average of any three consecutive strength tests equals or exceeds f'_c

(b) No individual strength test (average of two cylinders) falls below f'_c by more than 3.5 MPa when f'_c is 35 MPa or less; or by more than 0.10 f'_c when f'_c is more than 35 MPa.

5.6.3.4 If either of the requirements of 5.6.3.3 are not met, steps shall be taken to increase the average of subsequent strength test results. Requirements of 5.6.5 shall be observed if requirement of 5.6.3.3(b) is not met.

5.6.4. **Field-cured specimens**

5.6.4.1 If required by the building official, results of strength tests of cylinders cured under field conditions shall be provided.

5.6.4.2 Field-cured cylinders shall be cured under field conditions in accordance with "Practice for Making and Curing Concrete Test Specimens in the Field" (ASTM C 31).

5.6.4.3 Field-cured test cylinders shall be molded at the same time and from the same samples as laboratory-cured test cylinders.

5.6.4.4 Procedures for protecting and curing concrete shall be improved when strength of field-cured cylinders at test age designated for determination of f_c' is less than 85 percent of that of companion laboratory-cured cylinders. The 85 percent limitation shall not apply if field-cured strength exceeds f_c' by more than 3.5 MPa.

5.6.5 **Investigation of low-strength test results**

5.6.5.1 If any strength test (see 5.6.2.4) of laboratory-cured cylinders falls below specified value of f_c' by more than the values given in Section 5.6.3.3(b) or if tests of field-cured cylinders indicate deficiencies in protection and curing (see 5.6.4.4), steps shall be taken to assure that load-carrying capacity of the structure is not jeopardized.

5.6.5.2 If the likelihood of low-strength concrete is confirmed and calculations indicate that load-carrying capacity is significantly reduced, tests of cores drilled from the area in question in accordance with "Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete" (ASTM C 42M) shall be permitted. In such cases, three cores shall be taken for each strength test that falls below the values given in 5.6.3.3(b).

5.6.5.3 Cores shall be prepared for transport and storage by wiping drilling water from their surfaces and placing the cores in watertight bags or containers immediately after drilling. Cores shall be tested no earlier than 48 h and not later than 7 days after coring unless approved by the registered design professional.

5.6.5.4 Concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to at least 85 percent of f_c' and if no single core is less than 75 percent of f_c'. Additional testing of cores extracted from locations represented by erratic core strength results shall be permitted.

5.6.5.5 If criteria of 5.6.5.4 are not met and if the structural adequacy remains in doubt, the responsible authority shall be permitted to order a strength evaluation in accordance with Chapter 20 for the question-able portion of the structure, or take other appropriate action.

SECTION 5.7
PREPARATION OF EQUIPMENT AND PLACE OF DEPOSIT

5.7.1 Preparation before concrete placement shall include the following:

(a) All equipment for mixing and transporting concrete shall be clean;

(b) All debris shall be removed from spaces to be occupied by concrete;

(c) Forms shall be clean and properly coated;

(d) Masonry filler units that will be in contact with concrete shall be in a saturated surface dry condition;

Reinforcement shall be thoroughly clean of deleterious coatings, oil, dust, etc.

(f) Water shall be removed from place of deposit before concrete is placed unless a tremie is to be used or unless otherwise permitted by the building official;

(g) All laitance and other unsound material shall be removed before additional concrete is placed against hardened concrete.

SECTION 5.8
MIXING

5.8.1 All concrete shall be mixed until there is a uniform distribution of materials and shall be discharged completely before mixer is recharged.

5.8.2 Ready-mixed concrete shall be mixed and delivered in accordance with requirements of "Specification for Ready-Mixed Concrete" (ASTM C 94) or "Specification for Concrete Made by Volumetric Batching and Continuous Mixing" (ASTM C 685).

5.8.3 Job-mixed concrete shall be mixed in accordance with the following:

(a) Mixing shall be done in a batch mixer of approved type;

(b) Mixer shall be rotated at a speed recommended by the manufacturer;

(c) Mixing shall be continued for at least 1-1/2 minutes after all materials are in the drum, unless a shorter time is shown to be satisfactory by the mixing uniformity tests of "Specification for Ready-Mixed Concrete" (ASTM C 94);

(d) Materials handling, batching, and mixing shall conform to applicable provisions of "Specification for Ready-Mixed Concrete" (ASTM C 94);

(e) A detailed record shall be kept to identify:

(1) number of batches produced;
(2) proportions of materials used;
(3) approximate location of final deposit in structure;
(4) time and date of mixing and placing.

SECTION 5.9
CONVEYING

5.9.1 Concrete shall be conveyed from mixer to place of final deposit by methods that will prevent separation or loss of materials.

5.9.2 Conveying equipment shall be capable of providing a supply of concrete at site of placement without separation of ingredients and without interruptions sufficient to permit loss of plasticity between successive increments.

SECTION 5.10
PLACING

5.10.1 Concrete shall be deposited as nearly as practical in its final position to avoid segregation due to rehandling or flowing.

5.10.2 Concreting shall be carried on at such a rate that concrete is at all times plastic and flows readily into spaces between reinforcement.

5.10.3 Concrete that has partially hardened or been contaminated by foreign materials shall not be deposited in the structure.

5.10.4 Retempering of concrete with water shall not be permitted.

5.10.5 After concreting is started, it shall be carried on as a continuous operation until placing of a panel or section, as defined by its boundaries or predetermined joints, is completed except as permitted or prohibited by 6.4.

5.10.6 Top surfaces of vertically formed lifts shall be generally level.

5.10.7 When construction joints are required, joints shall be made in accordance with 6.4.

5.10.8 All concrete shall be thoroughly consolidated by suitable means during placement and shall be thoroughly worked around reinforcement and embedded fixtures and into corners of forms.

SECTION 5.11
CURING

5.11.1 Concrete shall be maintained above 10°C and in a moist condition for at least the first 7 days after placement, except when cured in accordance with 5.11.3.

5.11.2 During hot weather conditions, extra precautions should be taken to prevent surface from drying as required by section 5.13.7.

5.11.3 **Accelerated curing**

5.11.3.1 Curing by high pressure steam, steam at atmospheric pressure, heat and moisture, or other accepted processes, shall be permitted to accelerate strength gain and reduce time of curing.

5.11.3.2 Accelerated curing shall provide a compressive strength of the concrete at the load stage considered at least equal to required design strength at that load stage.

5.11.3.3 Curing process shall be such as to produce Concrete with a durability at least equivalent to the curing method of 5.11.1 or 5.11.2.

5.11.4 When required by the engineer or architect, supplementary strength tests in accordance with 5.6.4 shall be performed to assure that curing is satisfactory.

SECTION 5.12
COLD WEATHER REQUIREMENTS

Not applicable in the Kingdom.

SECTION 5.13
HOT WEATHER REQUIREMENTS

5.13.1 During hot weather conditions, proper attention shall be given to ingredients, production methods, handling, placing, and curing to prevent excessive concrete temperature or water evaporation that could impair the strength, durability and serviceability of the member or structure.

5.13.2 The temperature of fresh concrete shall be kept as low as practicable but shall not exceed 35 $^{\circ}$C at the time of placing.

5.13.3 The use of chemical admixtures, such as retarders and water reducers, shall be considered to offset the negative effects of hot weather.

5.13.4 Steps must be taken to transport, place, consolidate, and finish the concrete at the fastest possible rate.

1. Discharge of concrete shall be completed as soon as possible after the initial mixing at the batching plant. However, it should not be more than two hours provided retarding admixtures are used.
2. Unless otherwise required, concrete shall be proportioned for a slump of not less than 75 mm at the time of placing to permit prompt placement and effective consolidation in the form.
3. Concreting shall be done at the lowest ambient temperature, preferably early in the morning or late in the afternoon.
4. Delivery of concrete to the jobsite shall be scheduled so that it will be placed promptly on arrival.
5. The construction activity should be carefully planned to avoid cold joints. If construction joints become necessary, they shall be made in accordance with Section 6.4 of SBC 304.

5.13.5 Retempering of concrete by the addition of water to compensate for the loss of workability shall not be allowed.

5.13.6. All necessary precautions shall be taken to prevent plastic shrinkage cracking. In particular, precautions should be taken during placing of concrete to avoid excessive evaporation of mix water.

5.13.7 Curing of concrete shall commence as soon as the surfaces are finished and it shall continue for at least the first seven days.

Moist curing for the entire curing period is preferred. However, if moist curing cannot be continued beyond three days, concrete shall be protected from drying with curing paper, heat-reflecting plastic sheets, or membrane-forming curing compounds.

5.13.8 Tests on fresh concrete and specimen preparation shall be strictly in accordance with the relevant ASTM standards by qualified technicians.

Air temperature, concrete temperature, and general weather conditions at the time of concrete placement shall be recorded.

Inspection of concrete shall be detailed and emphasized in the project specifications to ascertain that adequate precautions are taken to minimize the adverse effects of hot weather on concrete properties.

CHAPTER 6
FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION JOINTS

SECTION 6.1
DESIGN OF FORMWORK

6.1.1 Forms shall result in a final structure that conforms to shapes, lines, and dimensions of the members as required by the design drawings and specifications.

6.1.2 Forms shall be substantial and sufficiently tight to prevent leakage of mortar.

6.1.3 Forms shall be properly braced or tied together to maintain position and shape.

6.1.4 Forms and their supports shall be designed so as not to damage previously placed structure.

6.1.5 Design of formwork shall include consideration of the following factors:

(a) Rate and method of placing concrete;

(b) Construction loads, including vertical, horizontal, and impact loads;

(c) Special form requirements for construction of shells, folded plates, domes, architectural concrete or similar types of elements.

6.1.6 Forms for prestressed concrete members shall be designed and constructed to permit movement of the member without damage during application of prestressing force.

SECTION 6.2
REMOVAL OF FORMS, SHORES, AND RESHORING

6.2.1 Removal of forms

Forms shall be removed in such a manner as not to impair safety and serviceability of the structure. Concrete exposed by form removal shall have sufficient strength not to be damaged by removal operation.

6.2.2 Removal of shores and reshoring

The provisions of Section 6.2.2.1 through 6.2.2.3 shall apply to slabs and beams except where cast on the ground.

6.2.2.1 Before starting construction, the contractor shall develop a procedure and schedule for removal of shores and installation of reshores and for calculating the loads transferred to the structure during the process.

(a) The structural analysis and concrete strength data used in planning and implementing form removal and shoring shall be furnished by the contractor to the building official when so requested;

(b) No construction loads shall be supported on, nor any shoring removed from, any part of the structure under construction except when that portion of the structure in combination with remaining forming and shoring system has sufficient strength to support safely its weight and loads placed thereon;

(c) Sufficient strength shall be demonstrated by structural analysis considering proposed loads, strength of forming and shoring system, and concrete strength data. Concrete strength data shall be based on tests of field-cured cylinders or, when approved by the building official, on other procedures to evaluate concrete strength.

6.2.2.2 No construction loads exceeding the combination of superimposed dead load plus specified live load shall be supported on any unshored portion of the structure under construction, unless analysis indicates adequate strength to support such additional loads.

6.2.2.3 Form supports for prestressed concrete members shall not be removed until sufficient prestressing has been applied to enable prestressed members to carry their dead load and anticipated construction loads.

SECTION 6.3
CONDUITS AND PIPES EMBEDDED IN CONCRETE

6.3.1 Conduits, pipes, and sleeves of any material not harmful to concrete and within limitations of 6.3 shall be permitted to be embedded in concrete with approval of the engineer, provided they are not considered to replace structurally the displaced concrete, except as provided in Section 6.3.6.

6.3.2 Conduits and pipes of aluminum shall not be embedded in structural concrete unless effectively coated or covered to prevent aluminum-concrete reaction or electrolytic action between aluminum and steel.

6.3.3 Conduits, pipes, and sleeves passing through a slab, wall, or beam shall not impair significantly the strength of the construction.

6.3.4 Conduits and pipes, with their fittings, embedded within a column shall not displace more than 4 percent of the area of cross section on which strength is calculated or which is required for fire protection.

6.3.5 Except when drawings for conduits and pipes are approved by the structural engineer, conduits and pipes embedded within a slab, wall, or beam (other than those merely passing through) shall satisfy Section 6.3.5.1 through 6.3.5.3.

6.3.5.1 They shall not be larger in outside dimension than 1/3 the overall thickness of slab, wall, or beam in which they are embedded.

6.3.5.2 They shall not be spaced closer than 3 diameters or widths on center.

6.3.5.3 They shall not impair significantly the strength of the construction.

6.3.6 Conduits, pipes, and sleeves shall not be considered as replacing structurally in compression the displaced concrete provided in 6.3.6.1 through 6.3.6.3.

6.3.6.1 They are not exposed to rusting or other deterioration.

6.3.6.2 They are of uncoated or galvanized iron or steel not thinner than standard Schedule 40 steel pipe.

6.3.6.3 They have a nominal inside diameter not over 50 mm and are spaced not less than 3 diameters on centers.

6.3.7 Pipes and fittings shall be designed to resist effects of the material, pressure, and temperature to which they will be subjected.

6.3.8 No liquid, gas, or vapor, except water not exceeding $30^{\circ}C$ nor 0.3 MPa pressure, shall be placed in the pipes until the concrete has attained its design strength.

6.3.9 In solid slabs, piping, unless it is for radiant heating or snow melting, shall be placed between top and bottom reinforcement.

6.3.10 Concrete cover for pipes, conduits, and fittings shall not be less than 40 mm for concrete exposed to earth or weather, nor less than 20 mm. for concrete not exposed to weather or in contact with ground.

6.3.11 Reinforcement with an area not less than 0.002 times area of concrete section shall be provided normal to piping.

6.3.12 Piping and conduit shall be so fabricated and installed that cutting, bending, or displacement of reinforcement from its proper location will not be required.

SECTION 6.4
CONSTRUCTION JOINTS

6.4.1 Surface of concrete construction joints shall be cleaned and laitance removed.

6.4.2 Immediately before new concrete is placed, all construction joints shall be wetted and standing water removed.

6.4.3 Construction joints shall be so made and located as not to impair the strength of the structure. Provision shall be made for transfer of shear and other forces through construction joints. See Section 11.7.9.

6.4.4 Construction joints in floors shall be located within the middle third of spans of slabs, beams, and girders. Joints in girders shall be offset a minimum distance of two times the width of intersecting beams.

6.4.5 Beams, girders, or slabs supported by columns or walls shall not be cast or erected until concrete in the vertical support members is no longer plastic.

6.4.6 Beams, girders, haunches, drop panels, and capitals shall be placed monolithically as part of a slab system, unless otherwise shown in design drawings or specifications.

CHAPTER 7
DETAILS OF REINFORCEMENT

SECTION 7.0
NOTATION

d = distance from extreme compression fiber to centroid of tension reinforcement, mm

d_b = nominal diameter of bar, wire, or prestressing strand, mm

f'_{ci} = compressive strength of concrete at time of initial prestress, MPa

f_y = specified yield strength of nonprestressed reinforcement, MPa

ℓ_d = development length, mm. See Chapter 12

SECTION 7.1
STANDARD HOOKS

The term standard hook as used in this code shall mean one of the following:

7.1.1 180-deg bend plus $4d_b$ extension, but not less than 60 mm at free end of bar.

7.1.2 90-deg bend plus $12d_b$ extension at free end of bar.

7.1.3 For stirrup and tie hooks

(a) Dia 16 mm bar and smaller, 90-deg bend plus $6d_b$ extension at free end of bar; or

(b) Dia 20 mm, Dia 22 mm, and Dia 25 mm bar, 90-deg bend plus $12d_b$ extension at free end of bar; or

(c) Dia 25 mm bar and smaller, 135-deg bend plus $6d_b$ extension at free end of bar.

7.1.4 Seismic hooks as defined in Section 21.1

SECTION 7.2
MINIMUM BEND DIAMETERS

7.2.1 Diameter of bend measured on the inside of the bar, other than for stirrups and ties in sizes Dia 10 mm through Dia 16 mm, shall not be less than the values in Table 7.2.

7.2.2 Inside diameter of bend for stirrups and ties shall not be less than $4d_b$ for Dia 16 mm bar and smaller. For bars larger than Dia 16 mm, diameter of bend shall be in accordance with Table 7.2.

7.2.3 Inside diameter of bend in welded wire fabric (plain or deformed) for stirrups and ties shall not be less than $4d_b$ for deformed wire larger than WD 7.0 and $2d_b$ for all other wires. Bends with inside diameter of less than $8d_b$ shall not be less than $4d_b$ from nearest welded intersection.

Table 7.2 - MINIMUM DIAMETERS OF BEND

Bar size	Minimum diameter
Dia 10 mm through Dia 25 mm	$6d_b$
Dia 28 mm, Dia 32 mm, and Dia 36 mm	$8d_b$
Dia 40 mm and larger	$10d_b$

SECTION 7.3
BENDING

7.3.1 All reinforcement shall be bent cold, unless otherwise permitted by the engineer.

7.3.2 Reinforcement partially embedded in concrete shall not be field bent, except as shown on the design drawings or permitted by the engineer.

SECTION 7.4
SURFACE CONDITIONS OF REINFORCEMENT

7.4.1 At the time concrete is placed, reinforcement shall be free from mud, oil, or other nonmetallic coatings that decrease bond. Epoxy coating of steel reinforcement in accordance with standards referenced in Section 3.5.3.7 and 3.5.3.8 shall be permitted.

7.4.2 Except for prestressing steel, steel reinforcement with rust, mill scale, or a combination of both shall be considered satisfactory, provided the minimum dimensions (including height of deformations) and weight of a hand-wire-brushed test specimen comply with applicable ASTM specifications referenced in 3.5.

7.4.3 Prestressing steel shall be clean and free of oil, dirt, scale, pitting and excessive rust. A light coating of rust shall be permitted.

SECTION 7.5
PLACING REINFORCEMENT

7.5.1 Reinforcement, including tendons, and post-tensioning ducts shall be accurately placed and adequately supported before concrete is placed, and shall be secured against displacement within tolerances permitted in Section 7.5.2.

7.5.2 Unless otherwise specified by the registered design professional, reinforcement, including tendons, and post-tensioning ducts shall be placed within the tolerances in Sections 7.5.2.1 and 7.5.2.2.

7.5.2.1 Tolerance for depth d and minimum concrete cover in flexural members, walls and compression members shall be as follows:

	Tolerance on d	Tolerance on minimum concrete cover
$d \leq 200\,mm$	±10 mm	-10 mm
$d > 200\,mm$	±15 mm	-15 mm

except that tolerance for the clear distance to formed soffits shall be minus 5 mm and tolerance for cover shall not exceed minus 1/3 the minimum concrete cover required in the design drawings or specifications.

7.5.2.2 Tolerance for longitudinal location of bends and ends of reinforcement shall be ±50 mm except the tolerance shall be ±15 mm at the discontinuous ends of brackets and corbels, and ±25 mm at the discontinuous ends of other members. The tolerance for minimum concrete cover of 7.5.2.1 shall also apply at discontinuous ends of members.

7.5.3 Welded wire fabric (with wire size not greater than WD 6.5) used in slabs not exceeding 3 m in span shall be permitted to be curved from a point near the top of slab over the support to a point near the bottom of slab at mid-span, provided such reinforcement is either continuous over, or securely anchored at support.

7.5.4 Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the engineer.

SECTION 7.6
SPACING LIMITS FOR REINFORCEMENT

7.6.1 The minimum clear spacing between parallel bars in a layer shall be d_b, but not less than 25 mm. See also Section 3.3.2.

7.6.2 Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above bars in the bottom layer with clear distance between layers not less than 25 mm.

7.6.3 In spirally reinforced or tied reinforced compression members, clear distance between longitudinal bars shall be not less than $1.5d_b$ nor less than 40 mm. See also Section 3.3.2.

7.6.4 Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.

7.6.5 In walls and slabs other than concrete joist construction, primary flexural reinforcement shall not be spaced farther apart than two times the wall or slab thickness, nor farther apart than 300 mm.

7.6.6 Bundled bars

7.6.6.1 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four in any one bundle.

7.6.6.2 Bundled bars shall be enclosed within stirrups or ties.

7.6.6.3 Bars larger than Dia 32 mm shall not be bundled in beams.

7.6.6.4 Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least $40d_b$ stagger.

7.6.6.5 Where spacing limitations and minimum concrete cover are based on bar diameter d_b, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

7.6.7 Tendons and ducts

7.6.7.1 Center-to-center spacing of pretensioning tendons at each end of a member shall be not less than $4d_b$ for strands, or $5d_b$ for wire, except that if concrete strength at transfer of prestress, f'_{ci}, is 28 MPa or more, minimum center to center spacing of strands shall be 50 mm. See also 3.3.2. Closer vertical spacing and bundling of tendons shall be permitted in the middle portion of a span.

7.6.7.2 Bundling of post-tensioning ducts shall be permitted if shown that concrete can be satisfactorily placed and if provision is made to prevent the prestressing steel, when tensioned, from breaking through the duct.

SECTION 7.7
CONCRETE PROTECTION FOR REINFORCEMENT

7.7.1 **Cast-in-place concrete (nonprestressed).** The following minimum concrete cover shall be provided for reinforcement, but shall not be less than required by Section 7.7.5 and 7.7.7:

	Minimum cover, mm
(a) Concrete cast against and permanently exposed to earth	75
(b) Concrete exposed to earth or weather:	
Dia 20 mm bars and larger..	50
Dia 18 mm bar, WD 12.0 wire, and smaller.............................	40
(c) Concrete not exposed to weather or in contact with ground:	
Slabs, walls, joists:	
Dia 40 mm bars and larger...	40
Bars with diameters smaller than 40 mm	20
Beams, Columns:	
Primary reinforcement, ties, stirrups, spirals	40
Shells, folded plate members:	
Dia 20 mm bar and Larger...	20
Dia 18 mm WD 12.0 wire, and smaller....................................	15

7.7.2 **Cast-in-place concrete (prestressed).** The following minimum concrete cover shall be provided for prestressed and nonprestressed reinforcement, ducts, and end fittings, but shall not be less than required by Section 7.7.5, 7.7.5.1, and 7.7.7:

	Minimum cover, mm
(a) Concrete cast against and permanently exposed to earth	75
(b) Concrete exposed to earth or weather:	

Wall panels, slabs, joists.. 25
Other members 40

(c) Concrete not exposed to weather or in contact with ground:
Slabs, walls, joists.. 40
Bars with diameters smaller than 40 mm 20
Beams, columns:
Primary reinforcement.. 40
Ties, stirrups, spirals 25
Shells, folded plate members:
Dia 18 mm bar, WD 12.0 wire, and smaller 10
Other reinforcement .... d_b but not less than 20

7.7.3 **Precast concrete (manufactured under plant control conditions).** The following minimum concrete cover shall be provided for prestressed and nonprestressed reinforcement, ducts, and end fittings, but shall not be less than required by Section 7.7.5, 7.7.5.1, and 7.7.7:

Minimum cover, mm

(a) Concrete exposed to earth or weather:
Wall panels:
Dia 40 mm bars and larger and prestressing tendons larger than 40 mm diameter... 40
Dia 36 mm bar and smaller, prestressing 40 mm diameter and smaller, WD 12.0 wire and smaller.................. 20
Other members:
Dia 40 mm bars and larger bars and prestressing tendons larger than 40 mm diameter... 50
Dia 20 mm through Dia 36 mm bars, prestressing tendons larger than 16 mm diameter through 40 mm diameter............ 40
Dia 16 mm bar and smaller, prestressing 16 mm diameter and smaller, WD 12.0 wire, and smaller 30

(b) Concrete not exposed to weather or in contact with ground:
Slabs, walls, joists:
Dia 40 mm bars and larger and prestressing tendons larger than 40 mm diameter... 30
Prestressing tendons 40 mm diameter and smaller................. 20
Dia 36 mm bar and smaller, WD 12.0 wire, and smaller 15
Beams, columns:
Primary reinforcement……. .... d_b but not less than 16 and need not exceed 40
Ties, stirrups, spirals.. 10
Shells, folded plate members:
Prestressing tendons... 20
Dia 20 mm bar and larger 15
Dia 16 mm bar and smaller, WD 12.0 wire, and smaller......... 10

7.7.4 **Bundled bars.** For bundled bars, minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 50 mm; except for concrete cast against and permanently exposed to earth, where minimum cover shall be 75 mm.

7.7.5 **Corrosive environments.** In corrosive environments or other severe exposure conditions, amount of concrete protection shall be suitably increased, and denseness and nonporosity of protecting concrete shall be considered, or other protection shall be provided.

7.7.5.1 For prestressed concrete members exposed to corrosive environments or other severe exposure conditions, and which are classified as Class T or C in 18.3.3, minimum cover to the prestressed reinforcement shall be increased 50 percent. This requirement shall be permitted to be waived if the pre-compressed tensile zone is not in tension under sustained loads.

7.7.6 **Future extensions.** Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

7.7.7 **Fire protection.** When Chapter 4 of SBC 801 requires a thickness of cover for fire protection greater than the minimum concrete cover specified in 7.7 of this code, such greater thicknesses shall be used.

SECTION 7.8
SPECIAL REINFORCEMENT DETAILS FOR COLUMNS

7.8.1 **Offset bars.** Offset bent longitudinal bars shall conform to the following:

7.8.1.1 Slope of inclined portion of an offset bar with axis of column shall not exceed 1 in 6.

7.8.1.2 Portions of bar above and below an offset shall be parallel to axis of column.

7.8.1.3 Horizontal support at offset bends shall be provided by lateral ties, spirals, or parts of the floor construction. Horizontal support provided shall be designed to resist $1\frac{1}{2}$ times the horizontal component of the computed force in the inclined portion of an offset bar. Lateral ties or spirals, if used, shall be placed not more than 150 mm from points of bend.

7.8.1.4 Offset bars shall be bent before placement in the forms. See Section 7.3.

7.8.1.5 Where a column face is offset 75 mm or greater, longitudinal bars shall not be offset bent. Separate dowels, lap spliced with the longitudinal bars adjacent to the offset column faces, shall be provided. Lap splices shall conform to 12.17.

7.8.2 **Steel cores.** Load transfer in structural steel cores of composite compression members shall be provided by the following:

7.8.2.1 Ends of structural steel cores shall be accurately finished to bear at end bearing splices, with positive provision for alignment of one core above the other in concentric contact.

7.8.2.2 At end bearing splices, bearing shall be considered effective to transfer not more than 50 percent of the total compressive stress in the steel core.

7.8.2.3 Transfer of stress between column base and footing shall be designed in accordance with 15.8.

7.8.2.4 Base of structural steel section shall be designed to transfer the total load from the entire composite member to the footing; or, the base shall be designed to transfer the load from the steel core only, provided ample concrete section is available for transfer of the portion of the total load carried by the reinforced concrete section to the footing by compression in the concrete and by reinforcement.

SECTION 7.9
CONNECTIONS

7.9.1 At connections of principal framing elements (such as beams and columns), enclosure shall be provided for splices of continuing reinforcement and for anchorage of reinforcement terminating in such connections.

7.9.2 Enclosure at connections shall consist of external concrete or internal closed ties, spirals, or stirrups.

SECTION 7.10
LATERAL REINFORCEMENT FOR COMPRESSION MEMBERS

7.10.1 Lateral reinforcement for compression members shall conform to the provisions of Section 7.10.4 and 7.10.5 and, where shear or torsion reinforcement is required, shall also conform to provisions of Chapter 11.

7.10.2 Lateral reinforcement requirements for composite compression members shall conform to 10.16. Lateral reinforcement requirements for tendons shall conform to 18.11.

7.10.3 It shall be permitted to waive the lateral reinforcement requirements of Section 7.10, 10.16, and 18.11 where tests and structural analysis show adequate strength and feasibility of construction.

7.10.4 **Spirals.** Spiral reinforcement for compression members shall conform to 10.9.3 and to the following:

7.10.4.1 Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled to permit handling and placing without distortion from designed dimensions.

7.10.4.2 For cast-in-place construction, size of spirals shall not be less than 10 mm diameter.

7.10.4.3 Clear spacing between spirals shall not exceed 75 mm, nor be less than 25 mm. See also Section 3.3.2.

7.10.4.4 Anchorage of spiral reinforcement shall be provided by $1\frac{1}{2}$ extra turns of spiral bar or wire at each end of a spiral unit.

7.10.4.5 Spiral reinforcement shall be spliced, if needed, by any one of the following methods:

(a) Lap splices not less than the larger of 300 mm and the length indicated in one of (1) through (5) below:

(1) deformed uncoated bar or wire.. $48d_b$

(2) plain uncoated bar or wire.. $72d_b$

(3) epoxy-coated deformed bar or wire ……………………..……... $72d_b$

(4) plain uncoated bar or wire with a standard stirrup or tie hook in accordance with 7.1.3 at ends of lapped spiral reinforcement. The hooks shall be embedded within the core confined by the spiral reinforcement ……..…..……….….…… $48d_b$

(5) epoxy-coated deformed bar or wire with a standard stirrup or tie hook in accordance with 7.1.3 at ends of lapped spiral reinforcement. The hooks shall be embedded within the core confined by the spiral reinforcement…………………….……….. $48d_b$

(b) Full mechanical or welded splices in accordance with Section 12.14.3.

7.10.4.6 Spirals shall extend from top of footing or slab in any story to level of lowest horizontal reinforcement in members supported above.

7.10.4.7 Where beams or brackets do not frame into all sides of a column, ties shall extend above termination of spiral to bottom of slab or drop panel.

7.10.4.8 In columns with capitals, spirals shall extend to a level at which the diameter or width of capital is two times that of the column.

7.10.4.9 Spirals shall be held firmly in place and true to line.

7.10.5 **Ties.** Tie reinforcement for compression members shall conform to the following:

7.10.5.1 All nonprestressed bars shall be enclosed by lateral ties, at least Dia 10 mm in size for longitudinal bars Dia 32 mm or smaller, and at least Dia 12 mm in size for Dia 32 mm bars and larger and bundled longitudinal bars. Deformed wire or welded wire fabric of equivalent area shall be permitted.

7.10.5.2 Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie bar or wire diameters, or least dimension of the compression member.

7.10.5.3 Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 deg and no bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie shall be permitted.

7.10.5.4 Ties shall be located vertically not more than one-half a tie spacing above the top of footing or slab in any story, and shall be spaced as provided herein to not more than one-half a tie spacing below the lowest horizontal reinforcement in slab or drop panel above.

7.10.5.5 Where beams or brackets frame from four directions into a column, termination of ties not more than 75 mm below lowest reinforcement in shallowest of such beams or brackets shall be permitted.

7.10.5.6 Where anchor bolts are placed in the top of columns or pedestals, the bolts shall be enclosed by lateral reinforcement that also surrounds at least four vertical bars of the column or pedestal. The lateral reinforcement shall be distributed within 130 mm of the top of the column or pedestal, and shall consist of at least two Dia 14 mm or three Dia 10 mm bars.

SECTION 7.11
LATERAL REINFORCEMENT FOR FLEXURAL MEMBERS

7.11.1 Compression reinforcement in beams shall be enclosed by ties or stirrups satisfying the size and spacing limitations in Section 7.10.5 or by welded wire fabric of equivalent area. Such ties or stirrups shall be provided throughout the distance where compression reinforcement is required.

7.11.2 Lateral reinforcement for flexural framing members subject to stress reversals or to torsion at supports shall consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement.

7.11.3 Closed ties or stirrups shall be formed in one piece by overlapping standard stirrup or tie end hooks around a longitudinal bar, or formed in one or two pieces lap spliced with a Class B splice (lap of $1.3\ell_d$) or anchored in accordance with Section 12.13.

SECTION 7.12
SHRINKAGE AND TEMPERATURE REINFORCEMENT

7.12.1 Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement shall be provided in structural slabs where the flexural reinforcement extends in one direction only.

7.12.1.1 Shrinkage and temperature reinforcement shall be provided in accordance with either Section 7.12.2 or 7.12.3.

7.12.1.2 Where shrinkage and temperature movements are significantly restrained, the requirements of Section 8.2.4 and 9.2.3 shall be considered.

7.12.2 Deformed reinforcement conforming to Section 3.5.3 used for shrinkage and temperature reinforcement shall be provided in accordance with the following:

7.12.2.1 Area of shrinkage and temperature reinforcement shall provide at least the following ratios of reinforcement area to gross concrete area, but not less than 0.0014:

(a) Slabs where Grade 300 or 350 deformed bars are used 0.0020

(b) Slabs where Grade 420 deformed bars or welded wire fabric (plain or deformed) are used 0.0018

(c) Slabs where reinforcement with yield stress exceeding 420 MPa measured at a yield strain of 0.35 percent is used $\dfrac{0.0018 \times 420}{f_y}$

7.12.2.2 Shrinkage and temperature reinforcement shall be spaced not farther apart than four times the slab thickness, nor farther apart than 300 mm.

7.12.2.3 At all sections where required, reinforcement for shrinkage and temperature stresses shall develop the specified yield strength f_y in tension in accordance with Chapter 12.

7.12.3 Prestressing steel conforming to 3.5.5 used for shrinkage and temperature reinforcement shall be provided in accordance with the following:

7.12.3.1 Tendons shall be proportioned to provide a minimum average compressive stress of 1.0 MPa on gross concrete area using effective prestress, after losses, in accordance with 18.6.

7.12.3.2 Spacing of tendons shall not exceed 2 m.

7.12.3.3 When spacing of tendons exceeds 1.4 m, additional bonded shrinkage and temperature reinforcement conforming to Section 7.12.2 shall be provided between the tendons at slab edges extending from the slab edge for a "distance" equal to the tendon spacing.

SECTION 7.13
REQUIREMENTS FOR STRUCTURAL INTEGRITY

7.13.1 In the detailing of reinforcement and connections, members of a structure shall be effectively tied together to improve integrity of the overall structure.

7.13.2 For cast-in-place construction, the following shall constitute minimum requirements:

7.13.2.1 In joist construction, at least one bottom bar shall be continuous or shall be spliced with a Class A tension splice or a mechanical or welded splice satisfying Section 12.14.3 and at noncontinuous supports shall be terminated with a standard hook.

7.13.2.2 Beams along the perimeter of the structure shall have continuous reinforcement consisting of:

(a) at least one-sixth of the tension reinforcement required for negative moment at the support, but not less than two bars; and

(b) at least one-quarter of the tension reinforcement required for positive moment at midspan, but not less than two bars.

7.13.2.3 Where splices are needed to provide the required continuity, the top reinforcement shall be spliced at or near midspan and bottom reinforcement shall be spliced at or near the support. Splices shall be Class A tension splices or mechanical or welded splices satisfying Section 12.14.3. The continuous reinforcement required in Section 7.13.2.2(a) and 7.13.2.2(b) shall be enclosed by the corners of U-stirrups having not less than 135-deg hooks around the continuous top bars, or by one-piece closed stirrups with not less than 135-deg hooks around one of the continuous top bars. Stirrups need not be extended through any joints.

7.13.2.4 In other than perimeter beams, when stirrups as defined in Section 7.13.2.3 are not provided, at least one-quarter of the positive moment reinforcement required at midspan, but not less than two bars, shall be continuous or shall be spliced over or near the support with a Class A tension splice or a mechanical or welded splice satisfying 12.14.3, and at noncontinuous supports shall be terminated with a standard hook.

7.13.2.5 For two-way slab construction, see Section 13.3.8.5.

7.13.3 For precast concrete construction, tension ties shall be provided in the transverse, longitudinal, and vertical directions and around the perimeter of the structure to effectively tie elements together. The provisions of 16.5 shall apply.

7.13.4 For lift-slab construction, see Section 13.3.8.6 and 18.12.6.

CHAPTER 8
ANALYSIS AND DESIGN – GENERAL CONSIDERATIONS

SECTION 8.0
NOTATION

A_s = area of nonprestressed tension reinforcement, mm^2

A_s' = area of compression reinforcement, mm^2

b = width of compression face of member, mm

d = distance from extreme compression fiber to centroid of tension reinforcement, mm

E_c = modulus of elasticity of concrete, MPa. See 8.5.1

E_s = modulus of elasticity of reinforcement, MPa. See 8.5.2 and 8.5.3

f_c' = specified compressive strength of concrete, MPa

f_y = specified yield strength of nonprestressed reinforcement, MPa

ℓ_n = clear span for positive moment or shear and average of adjacent clear spans for negative moment

V_c = nominal shear strength provided by concrete

w_c = unit weight of concrete, kg/m^3

w_u = factored load per unit length of beam or per unit area of slab

β_1 = factor defined in 10.2.7.3

ε_t = net tensile strain in extreme tension steel at nominal strength

ρ = ratio of nonprestressed tension reinforcement
= A_s / bd

ρ' = ratio of nonprestressed compression reinforcement
= A_s' / bd

ρ_b = reinforcement ratio producing balanced strain conditions. See 10.3.2

ϕ = strength reduction factor. See 9.3

SECTION 8.1
DESIGN METHOD

8.1.1 In design of structural concrete, members shall be proportioned for adequate strength in accordance with provisions of SBC 304, using load factors and strength reduction factors specified in Chapter 9.

8.1.2 Design of reinforced concrete using the provisions of Appendix B, Alternative Provisions for Reinforced and Prestressed Concrete Flexural and Compression Members, shall be permitted.

8.1.3 Anchors within the scope of Appendix D, Anchoring to Concrete, installed in concrete to transfer loads between connected elements shall be designed using Appendix D.

SECTION 8.2
LOADING

8.2.1 Design provisions of SBC 304 are based on the assumption that structures shall be designed to resist all applicable loads.

8.2.2 Service loads shall be in accordance with the SBC 301.

8.2.3 In design for wind and earthquake loads, integral structural parts shall be designed to resist the total lateral loads.

8.2.4 Consideration shall be given to effects of forces due to prestressing, crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete, and unequal settlement of supports.

SECTION 8.3
METHOD OF ANALYSIS

8.3.1 All members of frames or continuous construction shall be designed for the maximum effects of factored loads as determined by the theory of elastic analysis, except as modified according to 8.4. It shall be permitted to simplify design by using the assumptions specified in Section 8.6 through 8.9.

8.3.2 Except for prestressed concrete, approximate methods of frame analysis shall be permitted for buildings of usual types of construction, spans, and story heights.

8.3.3 As an alternate to frame analysis, the following approximate moments and shears shall be permitted for design of continuous beams and one-way slabs (slabs reinforced to resist flexural stresses in only one direction), provided:

(a) There are two or more spans;

(b) Spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 percent;

(c) Loads are uniformly distributed;

(d) Unit live load does not exceed three times unit dead load; and

(e) Members are prismatic.

(f) Supports are rigid.

Positive moment
End spans
Discontinuous end unstrained... $w_u \ell_n^2 / 11$
Discontinuous end integral with support .... $w_u \ell_n^2 / 14$
Interior spans .... $w_u \ell_n^2 / 16$

Negative moments at exterior face of the first interior support

Two spans………………………………………………………… $w_u \ell_n^2 / 9$

More than two spans ………………………………………………… $w_u \ell_n^2 / 10$

Negative moment at other faces of interior supports ………………….. $w_u \ell_n^2 / 11$

Negative moment at face of all supports for

Slabs with spans not exceeding 3 m; and beams where ratio of sum of column stiffnesses to beam stiffness exceeds eight at each end of the span ……………………………………………… $w_u \ell_n^2 / 12$

Negative moment at interior face of exterior support for members built integrally with supports

Where support is spandrel beam……………………………….… $w_u \ell_n^2 / 24$

Where support is a column……………………………………… $w_u \ell_n^2 / 16$

Shear in end members at face of first interior support …..…………..$1.15 w_u \ell_n / 2$

Shear at face of all other supports…………………………….……. $w_u \ell_n / 2$

8.3.4 Strut-and-tie models shall be permitted to be used in the design of structural concrete. See Appendix A.

SECTION 8.4
REDISTRIBUTION OF NEGATIVE MOMENTS IN CONTINUOUS FLEXURAL MEMBERS

8.4.1 Except where approximate values for moments are used, it shall be permitted to increase or decrease negative moments calculated by elastic theory at supports of continuous flexural members for any assumed loading arrangement by not more than $1000\varepsilon_t$ percent, with a maximum of 20 percent.

8.4.2 The modified negative moments shall be used for calculating moments at sections within the spans.

8.4.3 Redistribution of negative moments shall be made only when ε_t is equal to or greater than 0.0075 at the section at which moment is reduced.

SECTION 8.5
MODULUS OF ELASTICITY

8.5.1 Modulus of elasticity E_c for concrete shall be permitted to be taken as $w_c^{1.5} 0.043\sqrt{f_c'}$ (in MPa) for values of w_c between 1500 and 2500 kg/m^3. For normal weight concrete, E_c shall be permitted to be taken as $4700\sqrt{f_c'}$.

8.5.2 Modulus of elasticity E_s for nonprestressed reinforcement shall be permitted to be taken as 200000 MPa.

8.5.3 Modulus of elasticity E_s for prestressing steel shall be determined by tests or supplied by the manufacturer.

SECTION 8.6
STIFFNESS

8.6.1 Use of any set of reasonable assumptions shall be permitted for computing relative flexural and torsional stiffnesses of columns, walls, floors, and roof systems. The assumptions adopted shall be consistent throughout analysis.

8.6.2 Effect of haunches shall be considered both in determining moments and in design of members.

SECTION 8.7
SPAN LENGTH

8.7.1 Span length of members not built integrally with supports shall be considered as the clear span plus the depth of the member, but need not exceed distance between centers of supports.

8.7.2 In analysis of frames or continuous construction for determination of moments, span length shall be taken as the distance center-to-center of supports.

8.7.3 For beams built integrally with supports, design on the basis of moments at faces of support shall be permitted.

8.7.4 It shall be permitted to analyze solid or ribbed slabs built integrally with supports, with clear spans not more than 3 m, as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and width of beams otherwise neglected.

SECTION 8.8
COLUMNS

8.8.1 Columns shall be designed to resist the axial forces from factored loads on all floors or roof and the maximum moment from factored loads on a single adjacent span of the floor or roof under consideration. Loading condition giving the maximum ratio of moment to axial load shall also be considered.

8.8.2 In frames or continuous construction, consideration shall be given to the effect of unbalanced floor or roof loads on both exterior and interior columns and of eccentric loading due to other causes.

8.8.3 In computing gravity load moments in columns, it shall be permitted to assume far ends of columns built integrally with the structure to be fixed.

8.8.4 Resistance to moments at any floor or roof level shall be provided by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffnesses and conditions of restraint.

SECTION 8.9
ARRANGEMENT OF LIVE LOAD

8.9.1 It shall be permitted to assume that:

- **(a)** The live load is applied only to the floor or roof under consideration;
- **(b)** The far ends of columns built integrally with the structure are considered to be fixed.

8.9.2 It shall be permitted to assume that the arrangement of live load is limited to combinations of:

- **(a)** Factored dead load on all spans with full factored live load on two adjacent spans;
- **(b)** Factored dead load on all spans with full factored live load on alternate spans.

SECTION 8.10
T- BEAM CONSTRUCTION

8.10.1 In T-beam construction, the flange and web shall be built integrally or otherwise effectively bonded together.

8.10.2 Width of slab effective as a T-beam flange shall not exceed one-quarter of the span length of the beam, and the effective overhanging flange width on each side of the web shall not exceed:

- **(a)** eight times the slab thickness;
- **(b)** one-half the clear distance to the next web.

8.10.3 For beams with a slab on one side only, the effective overhanging flange width shall not exceed:

- **(a)** one-twelfth the span length of the beam;
- **(b)** six times the slab thickness;
- **(c)** one-half the clear distance to the next web.

8.10.4 Isolated beams, in which the T-shape is used to provide a flange for additional compression area, shall have a flange thickness not less than one-half the width of web and an effective flange width not more than four times the width of web.

8.10.5 Where primary flexural reinforcement in a slab that is considered as a T-beam flange (excluding joist construction) is parallel to the beam, reinforcement perpendicular to the beam shall be provided in the top of the slab in accordance with the following:

8.10.5.1 Transverse reinforcement shall be designed to carry the factored load on the overhanging slab width assumed to act as a cantilever. For isolated beams, the full width of overhanging flange shall be considered. For other T-beams, only the effective over-hanging slab width need be considered.

8.10.5.2 Transverse reinforcement shall be spaced not farther apart than three times the slab thickness, nor farther apart than 300 mm.

SECTION 8.11
JOIST CONSTRUCTION

8.11.1 Joist construction consists of a monolithic combination of regularly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions.

8.11.2 Ribs shall be not less than 100 mm width, and shall have a depth of not more than $3\frac{1}{2}$ times the minimum width of rib.

8.11.3 Clear spacing between ribs shall not exceed 800 mm.

8.11.4 Joist construction not meeting the limitations of 8.11.1 through 8.11.3 shall be designed as slabs and beams.

8.11.5 When permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to that of the specified strength of concrete in the joists are used:

8.11.5.1 For shear and negative moment strength computations, it shall be permitted to include the vertical shells of fillers in contact with the ribs. Other portions of fillers shall not be included in strength computations.

8.11.5.2 Slab thickness over permanent fillers shall be not less than one-twelfth the clear distance between ribs, nor less than 40 mm.

8.11.5.3 In one-way joists, reinforcement normal to the ribs shall be provided in the slab as required by Section 7.12.

8.11.6 When removable forms or fillers not complying with Section 8.11.5 are used:

8.11.6.1 Slab thickness shall be not less than one-twelfth the clear distance between ribs, nor less than 50 mm.

8.11.6.2 Reinforcement normal to the ribs shall be provided in the slab as required for flexure, considering load concentrations, if any, but not less than required by Section 7.12.

8.11.7 Where conduits or pipes as permitted by 6.3 are embedded within the slab, slab thickness shall be at least 25 mm greater than the total overall depth of the conduits or pipes at any point. Conduits or pipes shall not impair significantly the strength of the construction.

8.11.8 For joist construction, contribution of concrete to shear strength V_c shall be permitted to be 10 percent more than that specified in Chapter 11. It shall be permitted to increase shear strength using shear reinforcement or by widening the ends of ribs.

SECTION 8.12
SEPARATE FLOOR FINISH

8.12.1 A floor finish shall not be included as part of a structural member unless placed monolithically with the floor slab or designed in accordance with requirements of Chapter 17.

8.12.2 It shall be permitted to consider all concrete floor finishes as part of required cover or total thickness for nonstructural considerations

CHAPTER 9
STRENGTH AND SERVICEABILITY REQUIREMENTS

SECTION 9.0
NOTATION

A_g = gross area of section, mm^2

$A_s^{'}$ = area of compression reinforcement, mm^2

b = width of compression face of member, mm

c = distance from extreme compression fiber to neutral axis, mm

d = distance from extreme compression fiber to centroid of tension reinforcement, mm

$d^{'}$ = distance from extreme compression fiber to centroid of compression reinforcement, mm

d_s = distance from extreme tension fiber to centroid of tension reinforcement, mm

d_t = distance from extreme compression fiber to extreme tension steel, mm

D = dead loads, or related internal moments and forces

E = load effects of seismic forces, or related internal moments and forces

E_c = modulus of elasticity of concrete, MPa. See 8.5.1

$f_c^{'}$ = specified compressive strength of concrete, MPa

$\sqrt{f_c^{'}}$ = square root of specified compressive strength of concrete, MPa

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, MPa

f_r = modulus of rupture of concrete, MPa

f_y = specified yield strength of nonprestressed reinforcement, MPa

F = loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces

h = overall thickness of member, mm

H = loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces

I_{cr} = moment of inertia of cracked section transformed to concrete, mm^4

I_e = effective moment of inertia for computation of deflection, mm^4

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4

ℓ = span length of beam or one-way slab, as defined in 8.7; clear projection of cantilever, mm

ℓ_n = length of clear span in long direction of two-way construction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases, mm

L = live loads, or related internal moments and forces

L_r = roof live load, or related internal moments and forces

M_a = maximum moment in member at stage deflection is computed, N-mm

M_{cr} = cracking moment, N-mm. See 9.5.2.3

P_b = nominal axial load strength at balanced strain conditions, N. See 10.3.2

P_n = nominal axial load strength at given eccentricity, N.
R = rain load, or related internal moments and forces
T = cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete
U = required strength to resist factored loads or related internal moments and forces
W = wind load, or related internal moments and forces
w_c = weight of concrete, kg/m^3
y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, mm
α = ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of beam. See Chapter 13
α_m = average value of a for all beams on edges of a panel
β = ratio of clear spans in long to short direction of two-way slabs
ε_t = net tensile strain in extreme tension steel at nominal strength
λ = multiplier for additional long-term deflection as defined in 9.5.2.5
ζ = time-dependent factor for sustained load. See 9.5.2.5
ρ = ratio of nonprestressed tension reinforcement, = A_s / bd
ρ' = reinforcement ratio for nonprestressed compression reinforcement, = A_s' / bd
ρ_b = reinforcement ratio producing balanced strain conditions. See 10.3.2
ϕ = strength reduction factor. See 9.3

SECTION 9.1
GENERAL

9.1.1 Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in SBC 304.

9.1.2 Members also shall meet all other requirements of SBC 304 to ensure adequate performance at service load levels.

SECTION 9.2
REQUIRED STRENGTH

9.2.1 Required strength U shall be at least equal to the effects of factored loads in Eq. (9-1) through (9-7). The effect of one or more loads not acting simultaneously shall be investigated.

$$U = 1.4\,(D + F) \qquad (9\text{-}1)$$

$$U = 1.4\,(D + F + T) + 1.7(L + H) + 0.5\,(L_r \text{ or } R) \qquad (9\text{-}2)$$

$$U = 1.2D + 1.6\,(L_r \text{ or } R) + (1.0\,L \text{ or } 0.8\,W) \qquad (9\text{-}3)$$

$$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } R) \qquad (9\text{-}4)$$

$$U = 1.2D + 1.0E + 1.0L \qquad (9\text{-}5)$$

$$U = 0.9D + 1.6W + 1.6H \qquad (9\text{-}6)$$

$$U = 0.9D + 1.0E + 1.6H \qquad (9\text{-}7)$$

except as follows:

(a) The load factor on L in Eq. (9-3) to (9-5) shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where the live load value of 5 kN/m^2 to be consistent with the Saudi Building Code for Loading (SBC 301).

(b) The load factor on H shall be set equal to zero in Eq. (9-6) and (9-7) if the structural action due to H counteracts that due to W or E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

9.2.2 If resistance to impact effects is taken into account in design, such effects shall be included with live load L.

9.2.3 Estimations of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change shall be based on a realistic assessment of such effects occurring in service.

9.2.4 For a structure in a flood area, the flood load and load combinations of SBC 301 shall be used.

9.2.5 For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum prestressing steel jacking force.

SECTION 9.3
DESIGN STRENGTH

9.3.1 Design strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of SBC 304, multiplied by the strength reduction factors ϕ in 9.3.2, 9.3.4, and 9.3.5.

9.3.2 Strength reduction factor ϕ shall be as follows:

9.3.2.1 Tension-controlled sections as defined in 10.3.4………………….……….. 0.90

See also 9.3.2.7)

9.3.2.2 Compression-controlled sections, as defined in 10.3.3:

(a) Members with spiral reinforcement conforming to 10.9.3 …… 0.70

(b) Other reinforced members ……………………….………0.65

For sections in which the net tensile strain in the extreme tension steel at nominal strength is between the limits for compression-controlled and tension-controlled sections, ϕ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as the net tensile strain in the extreme tension steel at nominal strength increases from the compression-controlled strain limit to 0.005.

Alternatively, when Appendix B is used, for members in which f_y does not exceed 420 MPa, with symmetric reinforcement, and with $(h - d' - d_s)/h$ not less than 0.70, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ to zero. For other reinforced members, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ or ϕP_b, whichever is smaller, to zero.

9.3.2.3 Shear and torsion ……………………………….……………. 0.75

9.3.2.4 Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models) ……………….…………………………….. 0.65

9.3.2.5 Post-tensioned anchorage zones ……………………………... 0.85

9.3.2.6 Strut-and-tie models (Appendix A), and struts, ties, nodal zones, and bearing areas in such models …………………...............................…… 0.75

9.3.2.7 Flexure sections without axial load in pretensioned members where strand embedment is less than the development length as provided in 12.9.1.1 ……………………………………...………………….. 0.75

9.3.3 Development lengths specified in Chapter 12 do not require a ϕ-factor.

9.3.4 In structures that rely on special moment resisting frames or special reinforced concrete structural walls to resist earthquake effects, the strength reduction factors ϕ shall be modified as given in (a) through (c):

(a) The strength reduction factor for shear shall be 0.60 for any structural member that is designed to resist earthquake effects if its nominal shear strength is less than the shear corresponding to the development of the nominal flexural strength of the member. The nominal flexural strength shall be determined considering the most critical factored axial loads and including earthquake effects;

(b) The strength reduction factor for shear in diaphragms shall not exceed the minimum strength reduction factor for shear used for the vertical components of the primary lateral-force-resisting system;

(c) The strength reduction factor for shear in joints and diagonally reinforced coupling beams shall be 0.85.

9.3.5 Strength reduction factor ϕ for flexure, compression, shear, and bearing of structural plain concrete shall be 0.55.

SECTION 9.4
DESIGN STRENGTH FOR REINFORCEMENT

Designs shall not be based on yield strength of reinforcement f_y in excess of 550 MPa, except for prestressing steel.

SECTION 9.5
CONTROL OF DEFLECTIONS

9.5.1 Reinforced concrete members subjected to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that adversely affect strength or serviceability of a structure.

9.5.2 One-way construction (nonprestressed)

9.5.2.1 Minimum thickness stipulated in Table 9.5(a) shall apply for one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation of deflection indicates a lesser thickness can be used without adverse effects.

TABLE 9.5(a)
MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED

		Minimum thickness, *h*		
	Simply supported	**One end continuous**	**Both ends continuous**	**Cantilever**
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$

Notes:

1) Span length ℓ is in mm.
2) Values given shall be used directly for members with normal weight concrete (w_c = 2300 kg/m^3) and Grade 420 reinforcement for other conditions, the values shall be modified as follows:
   a) For structural lightweight concrete having unit weight in the range 1500-2000 kg/m³, the values shall be multiplied by (1.65 - 0.0003 w_c) but not less than 1.09, where w_c is the unit weight in kg/m^3.
   b) For f_y other than 420 MPa, the values shall be multiplied by (0.4 + f_y/700).

9.5.2.2 Where deflections are to be computed, deflections that occur immediately on application of load shall be computed by usual methods or formulas for elastic deflections, considering effects of cracking and reinforcement on member stiffness.

9.5.2.3 Unless stiffness values are obtained by a more comprehensive analysis, immediate deflection shall be computed with the modulus of elasticity E_c for concrete as specified in 8.5.1 (normalweight or lightweight concrete) and with the effective moment of inertia as follows, but not greater than I_g.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (9\text{-}8)$$

where:

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad (9\text{-}9)$$

and for normalweight concrete,

$$f_r = 0.7\sqrt{f_c'} \qquad (9\text{-}10)$$

When lightweight aggregate concrete is used, one of the following modifications shall apply:

(a) When f_{ct} is specified and concrete is proportioned in accordance with 5.2, f_r shall be modified by substituting $1.8 f_{ct}$ for $\sqrt{f_c'}$, but the value of $1.8 f_{ct}$ shall not exceed $\sqrt{f_c'}$

(b) When f_{ct} is not specified, f_r shall be multiplied by 0.75 for all-lightweight concrete, and 0.85 for sand-lightweight concrete. Linear interpolation shall be permitted if partial sand replacement is used.

9.5.2.4 For continuous members, effective moment of inertia shall be permitted to be taken as the average of values obtained from Eq. (9-8) for the critical positive and negative moment sections. For prismatic members, effective moment of inertia shall be permitted to be taken as the value obtained from Eq. (9-8) at midspan for simple and continuous spans, and at support for cantilevers.

9.5.2.5 Unless values are obtained by a more comprehensive analysis, additional long-term deflection resulting from creep and shrinkage of flexural members (normalweight or lightweight concrete) shall be determined by multiplying the immediate deflection caused by the sustained load considered, by the factor

$$\lambda = \frac{\zeta}{1 + 50\rho'} \tag{9-11}$$

where ρ' shall be the value at midspan for simple and continuous spans, and at support for cantilevers. It shall be permitted to assume the time-dependent factor for ζ sustained loads to be equal to:

5 years or more 2.0
12 months 1.4
6 Months 1.2
3 months 1.0

9.5.2.6 Deflection computed in accordance with 9.5.2.2 through 9.5.2.5 shall not exceed limits stipulated in Table 9.5(b).

TABLE 9.5(b)
MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

<table>
<tr><th>Type of member</th><th>Deflection to be considered</th><th>Deflection limitation</th></tr>
<tr><td>Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections</td><td>Immediate deflection due to live load L</td><td>$\ell/180$*</td></tr>
<tr><td>Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections</td><td>Immediate deflection due to live load L</td><td>$\ell/360$</td></tr>
<tr><td>Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections</td><td rowspan="2">That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)**</td><td>$\ell/480$‡</td></tr>
<tr><td>Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections</td><td>$\ell/240$§</td></tr>
</table>

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

** Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.3, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

‡ Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

§ Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

9.5.3 Two-way construction (nonprestressed)

9.5.3.1 Section 9.5.3 shall govern the minimum thickness of slabs or other two-way construction designed in accordance with the provisions of Chapter 13 and conforming with the requirements of 13.6.1.2. The thickness of slabs without interior beams spanning between the supports on all sides shall satisfy the requirements of Section 9.5.3.2 or 9.5.3.4. The thickness of slabs with beams spanning between the supports on all sides shall satisfy requirements of Section 9.5.3.3 or 9.5.3.4.

9.5.3.2 For slabs without interior beams spanning between the supports and having a ratio of long to short span not greater than 2, the minimum thickness shall be in accordance with the provisions of Table 9.5(c) and shall not be less than the following values:

(a) Slabs without drop panels as defined in Section 13.3.7.1 and 13.3.7.2 — 120 mm

(b) Slabs with drop panels as defined in Section 13.3.7.1 and 13.3.7.2 — 100 mm

TABLE 9.5(c)-MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS

Yield strength f_y MPa*	Without drop panels†			With drop panels†		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams‡		Without edge beams	With edge beams‡	
300	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
420	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
520	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$	$\ell_n/31$	$\ell_n/34$	$\ell_n/34$

* For values of reinforcement yield strength between the values given in the table, minimum thickness shall be determined by linear interpolation.
† Drop panel is defined in 13.3.7.1 and 13.3.7.2
‡ Slabs with beams between columns along exterior edges. The value of α for the edge beam shall not be less than 0.8.

9.5.3.3 For slabs with beams spanning between the supports on all sides, the minimum thickness shall be as follows:

(a) For α_m equal to or less than 0.2, the provisions of Section 9.5.3.2 shall apply;

(b) For α_m greater than 0.2 but not greater than 2.0, the thickness shall not be less than

$$h = \frac{\ell_n\left(0.8 + \frac{f_y}{1500}\right)}{36 + 5\beta(\alpha_m - 0.2)} \tag{9-12}$$

and not less than 120 mm;

(c) For α_m greater than 2.0, the thickness shall not be less than

$$h = \frac{\ell_n\left(0.8 + \frac{f_y}{1500}\right)}{36 + 9\beta} \tag{9-13}$$

and not less than 90 mm;

(d) At discontinuous edges, an edge beam shall be provided with a stiffness ratio α not less than 0.80 or the minimum thickness required by Eq. (9-12) or (9-13) shall be increased by at least 10 percent in the panel with a discontinuous edge.

9.5.3.4 Slab thickness less than the minimum thickness required by Section 9.5.3.1, 9.5.3.2, and 9.5.3.3 shall be permitted to be used if shown by computation that the deflection will not exceed the limits stipulated in Table 9.5(b). Deflections shall be computed taking into account size and shape of the panel, conditions of support, and nature of restraints at the panel edges. The modulus of elasticity of concrete E_c shall be as specified in Section 8.5.1. The effective moment of inertia shall be that given by Eq. (9-8); other values shall be permitted to be used if they result in computed deflections in reasonable agreement with results of comprehensive tests. Additional long-term deflection shall be computed in accordance with Section 9.5.2.5.

9.5.4 Prestressed concrete construction

9.5.4.1 For flexural members designed in accordance with provisions of Chapter 18, immediate deflection shall be computed by usual methods or formulas for elastic deflections, and the moment of inertia of the gross concrete section shall be permitted to be used for Class U flexural members, as defined in Section 18.3.3.

9.5.4.2 For Class C and Class T flexural members, as defined in Section 18.3.3, deflection calculations shall be based on a cracked transformed section analysis. It shall be permitted to base computations on a bilinear moment-deflection relationship, or an effective moment of inertia as defined by Eq. (9-8).

9.5.4.3 Additional long-term deflection of prestressed concrete members shall be computed taking into account stresses in concrete and steel under sustained load and including effects of creep and shrinkage of concrete and relaxation of steel.

9.5.4.4 Deflection computed in accordance with Section 9.5.4.1 or 9.5.4.2, and 9.5.4.3 shall not exceed limits stipulated in Table 9.5(b).

9.5.5 Composite construction

9.5.5.1 Shored construction

If composite flexural members are supported during construction so that, after removal of temporary supports, dead load is resisted by the full composite section, it shall be permitted to consider the composite member equivalent to a monolithically cast member for computation of deflection. For nonprestressed members, the portion of the member in compression shall determine whether values in Table 9.5(a) for normalweight or lightweight concrete shall apply. If deflection is computed, account shall be taken of curvatures resulting from differential shrinkage of precast and cast-in-place components, and of axial creep effects in a prestressed concrete member.

9.5.5.2 Unshored construction

If the thickness of a nonprestressed precast flexural member meets the requirements of Table 9.5(a), deflection need not be computed. If the thickness of a nonprestressed composite member meets the requirements of Table 9.5(a), it is not required to compute deflection occurring after the member becomes composite, but the long-term deflection of the precast member shall be investigated for magnitude and duration of load prior to beginning of effective composite action.

9.5.5.3 Deflection computed in accordance with Section 9.5.5.1 or 9.5.5.2 shall not exceed limits stipulated in Table 9.5(b).

CHAPTER 10
FLEXURAL AND AXIAL LOADS

SECTION 10.0
NOTATION

a = depth of equivalent rectangular stress block as defined in 10.2.7.1, mm

A_b = area of an individual horizontal bar or wire, mm^2

A_c = area of core of spirally reinforced compression member measured to outside diameter of spiral, mm^2

A_g = gross area of section, mm^2

A_s = area of nonprestressed tension reinforcement, mm^2

$A_{s,\min}$ = minimum amount of flexural reinforcement, mm^2 , See 10.5

A_{st} = total area of longitudinal reinforcement, (bars or steel shapes), mm^2

A_t = area of structural steel shape, pipe, or tubing in a composite section, mm^2

A_1 = loaded area, mm^2

A_2 = the area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal, mm^2

b = width of compression face of member, mm

b_w = web width, mm

c = distance from extreme compression fiber to neutral axis, mm

c_c = clear cover from the nearest surface in tension to the surface of the flexural tension reinforcement, mm

C_m = a factor relating actual moment diagram to an equivalent uniform moment diagram

d = distance from extreme compression fiber to centroid of tension reinforcement, mm

E_c = modulus of elasticity of concrete, MPa. See 8.5.1

E_s = modulus of elasticity of reinforcement, MPa. See 8.5.2 or 8.5.3

EI = flexural stiffness of compression member. See Eq. (10-11) and Eq. (10-12), N-mm^2

f_c' = specified compressive strength of concrete, MPa

f_s = calculated stress in reinforcement at service loads, MPa

f_y = specified yield strength of nonprestressed reinforcement, MPa

h = overall thickness of member, mm

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4

I_{se} = moment of inertia of reinforcement about centroidal axis of member cross section, mm^4

I_t = moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross section, mm^4

k = effective length factor for compression members

ℓ_c = length of compression member in a frame, measured from center to center of the joints in the frame, mm

ℓ_u = unsupported length of compression member, mm

M_c = factored moment to be used for design of compression member, N-mm

M_s = moment due to loads causing appreciable sway, N-mm

M_u = factored moment at section, N-mm

M_1 = smaller factored end moment on a compression member, positive if member is bent in single curvature, negative if bent in double curvature, N-mm

M_{1ns} = factored end moment on a compression member at the end at which M_1 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm

M_{1s} = factored end moment on compression member at the end at which M_1 acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm

M_2 = larger factored end moment on compression member, always positive, N-mm

$M_{2,\min}$ = minimum value of M_2, N-mm

M_{2ns} = factored end moment on compression member at the end at which M_2 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm

M_{2s} = factored end moment on compression member at the end at which M_2 acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm

P_b = nominal axial load strength at balanced strain conditions. See 10.3.2, N

P_c = critical load. See Eq. (10-10), N

P_n = nominal axial load strength at given eccentricity, N

P_o = nominal axial load strength at zero eccentricity, N

P_u = factored axial load at given eccentricity, $N \le \phi P_n$

Q = stability index for a story. See 10.11.4

r = radius of gyration of cross section of a compression member, mm

s = center-to-center spacing of flexural tension reinforcement nearest to the extreme tension face, mm (where there is only one bar or wire nearest to the extreme tension face, s is the width of the extreme tension face.)

s_{sk} = spacing of skin reinforcement, mm

V_u = factored horizontal shear in a story, N

β_1 = factor defined in 10.2.7.3

β_d = (a) for nonsway frames, β_d is the ratio of the maximum factored axial sustained load to the maximum factored axial load associated with the same load combination;
(b) for sway frames, except as required in (c) of this definition, β_d is the ratio of the maximum factored sustained shear within a story to the maximum factored shear in that story;
(c) for stability checks of sway frames carried out in accordance with 10.13.6, β_d is the ratio of the maximum factored sustained axial load to the maximum factored axial load

δ_{ns} = moment magnification factor for frames braced against sidesway, to reflect effects of member curvature between ends of compression member

δ_s = moment magnification factor for frames not braced against sidesway, to reflect lateral drift resulting from lateral and gravity loads

Δ_o = relative lateral deflection between the top and bottom of a story due to V_u, computed using a first-order elastic frame analysis and stiffness values satisfying 10.11.1, mm

ε_t = net tensile strain in extreme tension steel at nominal strength

ρ = ratio of nonprestressed tension reinforcement
= A_s / bd

ρ_b = reinforcement ratio producing balanced strain conditions. See 10.3.2

ρ_S = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member

ϕ = strength reduction factor. See 9.3

ϕ_k = stiffness reduction factor. See R10.12.3

SECTION 10.1
SCOPE

10.1.1 Provisions of Chapter 10 shall apply for design of members subject to flexure or axial loads or to combined flexure and axial loads.

SECTION 10.2
DESIGN ASSUMPTIONS

10.2.1 Strength design of members for flexure and axial loads shall be based on assumptions given in Sections 10.2.2 through 10.2.7, and on satisfaction of applicable conditions of equilibrium and compatibility of strains.

10.2.2 Strain in reinforcement and concrete shall be assumed directly proportional to the distance from the neutral axis, except that, for deep beams as defined in Section 10.7.1, an analysis that considers a nonlinear distribution of strain shall be used. Alternatively, it shall be permitted to use a strut-and-tie model. See Sections 10.7, 11.8, and Appendix A.

10.2.3 Maximum usable strain at extreme concrete compression fiber shall be assumed equal to 0.003.

10.2.4 Stress in reinforcement below specified yield strength f_y for grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.

10.2.5 Tensile strength of concrete shall be neglected in axial and flexural calculations of reinforced concrete, except when meeting requirements of 18.4.

10.2.6 The relationship between concrete compressive stress distribution and concrete strain shall be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

10.2.7 Requirements of Section 10.2.6 are satisfied by an equivalent rectangular concrete stress distribution defined by the following:

10.2.7.1 Concrete stress of $0.85 f_c^{'}$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fiber of maximum compressive strain.

10.2.7.2 Distance c from the fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

10.2.7.3 Factor β_1 shall be taken as 0.85 for concrete strengths $f_c^{'}$ up to and including 30 MPa. For strengths above 30 MPa, β_1 shall be reduced continuously at a rate of 0.05 for each 7 MPa of strength in excess of 30 MPa, but β_1 shall not be taken less than 0.65.

SECTION 10.3
GENERAL PRINCIPLES AND REQUIREMENTS

10.3.1 Design of cross sections subject to flexure or axial loads, or to combined flexure and axial loads, shall be based on stress and strain compatibility using assumptions in 10.2.

10.3.2 Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as concrete in compression reaches its assumed ultimate strain of 0.003.

10.3.3 Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit at the time the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 420 reinforcement, and for all prestressed reinforcement, it shall be permitted to set the compression-controlled strain limit equal to 0.002.

10.3.4 Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.

10.3.5 For nonprestressed flexural members and nonprestressed members with axial load less than $0.10 f_c^{'} A_g$, the net tensile strain ε_t at nominal strength shall not be less than 0.005.

10.3.5.1 Use of compression reinforcement shall be permitted in conjunction with additional tension reinforcement to increase the strength of flexural members.

10.3.6 Design axial load strength ϕP_n of compression members shall not be taken greater than the following:

10.3.6.1 For nonprestressed members with spiral reinforcement conforming to 7.10.4 or composite members conforming to 10.16:

$$\phi P_{n,\max} = 0.85\phi\left[0.85 f_c^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right] \quad (10\text{-}1)$$

10.3.6.2 For nonprestressed members with tie reinforcement conforming to 7.10.5:

$$\phi P_{n,\max} = 0.80\phi\left[0.85 f_c^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right] \quad (10\text{-}2)$$

10.3.6.3 For prestressed members, design axial load strength ϕP_n shall not be taken greater than 0.85 (for members with spiral reinforcement) or 0.80 (for members with tie reinforcement) of the design axial load strength at zero eccentricity ϕP_o.

10.3.7 Members subject to compressive axial load shall be designed for the maximum moment that can accompany the axial load. The factored axial load P_u at given eccentricity shall not exceed that given in 10.3.6. The maximum factored moment M_u shall be magnified for slenderness effects in accordance with 10.10.

SECTION 10.4
DISTANCE BETWEEN LATERAL SUPPORTS OF FLEXURAL MEMBERS

10.4.1 Spacing of lateral supports for a beam shall not exceed 50 times the least width b of compression flange or face.

10.4.2 Effects of lateral eccentricity of load shall be taken into account in determining spacing of lateral supports.

SECTION 10.5
MINIMUM REINFORCEMENT OF FLEXURAL MEMBERS

10.5.1 At every section of a flexural member where tensile reinforcement is required by analysis, except as provided in Section 10.5.2, and 10.5.3, the area A_s provided shall not be less than that given by

$$A_{s,\min} = \frac{\sqrt{f_c^{'}}}{4 f_y} b_w d \quad (10\text{-}3)$$

and not less than $1.4 b_w d / f_y$

10.5.2 For statically determinate members with a flange in tension, the area $A_{s,\min}$ shall be equal to or greater than the value given by Eq. (10-3) with b_w replaced by either $2b_w$ or the width of the flange, whichever is smaller.

10.5.3 For structural slabs and footings of uniform thickness the minimum area of tensile reinforcement in the direction of the span shall be the same as that required by 7.12. Maximum spacing of this reinforcement shall not exceed three times the thickness, nor 300 mm.

SECTION 10.6
DISTRIBUTION OF FLEXURAL REINFORCEMENT IN BEAMS AND ONE-WAY SLABS

10.6.1 This section prescribes rules for distribution of flexural reinforcement to control flexural cracking in beams and in one-way slabs (slabs reinforced to resist flexural stresses in only one direction).

10.6.2 Distribution of flexural reinforcement in two-way slabs shall be as required by 13.3.

10.6.3 Flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross section as required by 10.6.4.

10.6.4 The spacing s of reinforcement closest to a surface in tension shall not exceed that given by

$$s = \frac{95{,}000}{f_s} - 2.5c_c \qquad (10\text{-}4)$$

but not greater than $300(252/f_s)$.

Calculated stress f_s (in MPa) in reinforcement at service load shall be computed as the unfactored moment divided by the product of steel area and internal moment arm. It shall be permitted to take f_s as 60 percent of specified yield strength.

10.6.5 Provisions of Section 10.6.4 are not sufficient for structures subject to very aggressive exposure or designed to be watertight. For such structures, special investigations and precautions are required.

10.6.6 Where flanges of T-beam construction are in tension, part of the flexural tension reinforcement shall be distributed over an effective flange width as defined in Section 8.10, or a width equal to one-tenth the span, whichever is smaller. If the effective flange width exceeds one-tenth the span, some longitudinal reinforcement shall be provided in the outer portions of the flange.

10.6.7 If the effective depth d of a beam or joist exceeds 900 mm, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance $d/2$ nearest the flexural tension reinforcement. The spacing s_{sk} between longitudinal bars or wires of the skin reinforcement shall not exceed the least of $d/6$, 300 mm, and $1000A_b/(d-750)$. It shall be permitted to include such reinforcement in strength computations if a strain compatibility analysis is made to determine stress in the individual bars or wires. The total area of longitudinal skin reinforcement in both faces need not exceed one-half of the required flexural tensile reinforcement.

SECTION 10.7
DEEP BEAMS

10.7.1 Deep beams are members loaded on one face and supported on the opposite face so that compression struts can develop between the loads and the supports, and have either:

(a) clear spans, ℓ_n equal to or less than four times the overall member depth; or

(b) regions loaded with concentrated loads within twice the member depth from the face of the support.

Deep beams shall be designed either taking into account nonlinear distribution of strain, or by Appendix A. (See also Section 11.8.1 and 12.10.6.). Lateral buckling shall be considered.

10.7.2 Shear strength of deep beams shall be in accordance with 11.8.

10.7.3 Minimum flexural tension reinforcement shall conform to 10.5.

10.7.4 Minimum horizontal and vertical reinforcement in the side faces of deep beams shall satisfy either A.3.3 or Sections 11.8.4 and 11.8.5.

SECTION 10.8
DESIGN DIMENSIONS FOR COMPRESSION MEMBERS

10.8.1 **Isolated compression member with multiple spirals**. Outer limits of the effective cross section of a compression member with two or more interlocking spirals shall be taken at a distance outside the extreme limits of the spirals equal to the minimum concrete cover required by Section 7.7.

10.8.2 **Compression member built monolithically with wall.** Outer limits of the effective cross section of a spirally reinforced or tied reinforced compression member built monolithically with a concrete wall or pier shall be taken not greater than 40 mm outside the spiral or tie reinforcement.

10.8.3 **Equivalent circular compression member.** As an alternative to using the full gross area for design of a compression member with a square, octagonal, or other shaped cross section, it shall be permitted to use a circular section with a diameter equal to the least lateral dimension of the actual shape. Gross area considered, required percentage of reinforcement, and design strength shall be based on that circular section.

10.8.4 **Limits of section.** For a compression member with a cross section larger than required by considerations of loading, it shall be permitted to base the minimum reinforcement and strength on a reduced effective area A_g not less than one-half the total area. This provision shall not apply in regions of high seismic risk.

SECTION 10.9
LIMITS FOR REINFORCEMENT OF COMPRESSION MEMBERS

10.9.1 Area of longitudinal reinforcement for non-composite compression members shall be not less than 0.01 nor more than 0.08 times gross area A_g of section.

10.9.2 Minimum number of longitudinal bars in compression members shall be 4 for bars within rectangular or circular ties, 3 for bars within triangular ties, and 6 for bars enclosed by spirals conforming to Section 10.9.3.

10.9.3 Ratio of spiral reinforcement ρ_S shall be not less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_y} \qquad (10\text{-}5)$$

where f_y is the specified yield strength of spiral reinforcement but not more than 420 MPa.

SECTION 10.10
SLENDERNESS EFFECTS IN COMPRESSION MEMBERS

10.10.1 Except as allowed in 10.10.2, the design of compression members, restraining beams, and other supporting members shall be based on the factored forces and moments from a second-order analysis considering material nonlinearity and cracking, as well as the effects of member curvature and lateral drift, duration of the loads, shrinkage and creep, and interaction with the supporting foundation. The dimensions of each member cross section used in the analysis shall be within 10 percent of the dimensions of the members shown on the design drawings or the analysis shall be repeated. The analysis procedure shall have been shown to result in prediction of strength in substantial agreement with the results of comprehensive tests of columns in statically indeterminate reinforced concrete structures.

10.10.2 As an alternate to the procedure prescribed in Section 10.10.1, it shall be permitted to base the design of compression members, restraining beams, and other supporting members on axial forces and moments from the analyses described in 10.11.

SECTION 10.11
MAGNIFIED MOMENTS - GENERAL

10.11.1 The factored axial forces P_u the factored moments M_1 and M_2 at the ends of the column, and, where required, the relative lateral story deflections Δ_o shall be computed using an elastic first-order frame analysis with the section properties determined taking into account the influence of axial loads, the presence of cracked regions along the length of the member, and effects of duration of the loads. Alternatively, it shall be permitted to use the following properties for the members in the structure:

(a) Modulus of elasticity.......................... E_c from 8.5.1

(b) Moments of inertia

Beams .... $0.35\, I_g$

Columns .... $0.70\, I_g$

Walls -Uncracked .... $0.70\, I_g$

-Cracked .... $0.35\, I_g$

Flat plates and flat slabs .... $0.25\, I_g$

(c) Area ....$1.0\, A_g$

The moments of inertia shall be divided by $(1+\beta_d)$

(a) When sustained lateral loads act; or

(b) For stability checks made in accordance with Section 10.13.6.

10.11.2 It shall be permitted to take the radius of gyration r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

10.11.3 **Unsupported length of compression members**

10.11.3.1 The unsupported length ℓ_u of a compression member shall be taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support in the direction being considered.

10.11.3.2 Where column capitals or haunches are present, the unsupported length shall be measured to the lower extremity of the capital or haunch in the plane considered.

10.11.4 Columns and stories in structures shall be designated as nonsway or sway columns or stories. The design of columns in nonsway frames or stories shall be based on Section 10.12. The design of columns in sway frames or stories shall be based on 10.13.

10.11.4.1 It shall be permitted to assume a column in a structure is nonsway if the increase in column end moments due to second-order effects does not exceed 5 percent of the first-order end moments.

10.11.4.2 It also shall be permitted to assume a story within a structure is nonsway if:

$$Q = \frac{\sum P_u \Delta_o}{V_u \ell_c} \qquad (10\text{-}6)$$

is less than or equal to 0.05, where $\sum P_u$ and V_u are the total vertical load and the story shear, respectively, in the story in question and Δ_o is the first-order relative deflection between the top and bottom of that story due to V_u .

10.11.5 Where an individual compression member in the frame has a slenderness $k\ell_u / r$ of more than 100, Section 10.10.1 shall be used to compute the forces and moments in the frame.

10.11.6 For compression members subject to bending about both principal axes, the moment about each axis shall be magnified separately based on the conditions of restraint corresponding to that axis.

SECTION 10.12
MAGNIFIED MOMENTS – NONSWAY FRAMES

10.12.1 For compression members in nonsway frames, the effective length factor k shall be taken as 1.0, unless analysis shows that a lower value is justified. The calculation of k shall be based on the E and I values used in Section 10.11.1.

10.12.2 In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k\ell_u}{r} \le 34 - 12(M_1 / M_2) \qquad (10\text{-}7)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

10.12.3 Compression members shall be designed for the factored axial load P_u and the moment amplified for the effects of member curvature M_c as follows:

$$M_c = \delta_{ns} M_2 \tag{10-8}$$

where

$$\delta_{ns} = \frac{C_m}{1 - \dfrac{P_u}{0.75 P_c}} \geq 1.0 \tag{10-9}$$

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \tag{10-10}$$

EI shall be taken as

$$EI = \frac{(0.2 E_c I_g + E_s I_{se})}{1 + \beta_d} \tag{10-11}$$

$$EI = \frac{0.4 E_c I_g}{1 + \beta_d} \tag{10-12}$$

10.12.3.1 For members without transverse loads between supports, C_m shall be taken as

$$C_m = 0.6 + 0.4 \frac{M_1}{M_2} \geq 0.4 \tag{10-13}$$

where M_1/M_2 is positive if the column is bent in single curvature. For members with transverse loads between supports, C_m shall be taken as 1.0.

10.12.3.2 The factored moment M_2 in Eq. (10-8) shall not be taken less than

$$M_{2,\min} = P_u (15 + 0.03h) \tag{10-14}$$

about each axis separately, where 15 and h are in millimeters. For members for which $M_{2,\min}$ exceeds M_2, the value of C_m in Eq. (10-13) shall either be taken equal to 1.0, or shall be based on the ratio of the computed end moments M_1 and M_2.

SECTION 10.13
MAGNIFIED MOMENTS – SWAY FRAMES

10.13.1 For compression members not braced against sidesway, the effective length factor k shall be determined using E and I values in accordance with Section 10.11.1 and shall not be less than 1.0.

10.13.2 For compression members not braced against sidesway, it shall be permitted to neglect the effects of slenderness when $k\ell_u/r$ *is* less than 22.

10.13.3 The moments M_1 and M_2 at the ends of an individual compression member shall be taken as

$$M_1 = M_{1ns} + \delta_s M_{1s} \qquad (10\text{-}15)$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \qquad (10\text{-}16)$$

where $\delta_s M_{1s}$ and $\delta_s M_{2s}$ shall be computed according to 10.13.4.

10.13.4 Calculation of $\delta_s M_s$

10.13.4.1 The magnified sway moments $\delta_s M_s$ shall be taken as the column end moments calculated using a second-order elastic analysis based on the member stiffnesses given in Section 10.11.1.

10.13.4.2 Alternatively, it shall be permitted to calculate $\delta_s M_s$ as

$$\delta_s M_s = \frac{M_s}{1-Q} \geq M_s \qquad (10\text{-}17)$$

If δ_s calculated in this way exceeds 1.5, $\delta_s M_s$ shall be calculated using 10.13.4.1 or 10.13.4.3.

10.13.4.3 Alternatively, it shall be permitted to calculate the magnified sway moment $\delta_s M_s$ as

$$\delta_s M_s = \frac{M_s}{1 - \dfrac{\sum P_u}{0.75 \sum P_c}} \geq M_s \qquad (10\text{-}18)$$

where $\sum P_u$ is the summation for all the vertical loads in a story and $\sum P_c$ is the summation for all sway resisting columns in a story. P_c is calculated using Eq. (10-10) using k from 10.13.1 and EI from Eq. (10-11) or Eq. (10-12).

10.13.5 If an individual compression member has.

$$\frac{\ell_u}{r} > \frac{35}{\sqrt{\dfrac{P_u}{f_c^{'} A_g}}} \qquad (10\text{-}19)$$

it shall be designed for the factored axial load P_u and the moment M_c calculated using Section 10.12.3 in which M_1 and M_2 are computed in accordance with Section 10.13.3, β_d as defined for the load combination under consideration, and k as defined in Section 10.12.1.

10.13.6 In addition to load cases involving lateral loads, the strength and stability of the structure as a whole under factored gravity loads shall be considered.

(a) When $\delta_s M_s$ is computed from 10.13.4.1, the ratio of second-order lateral deflections to first-order lateral deflections for 1.4 dead load and 1.7 live load plus lateral load applied to the structure shall not exceed 2.5;

(b) When $\delta_s M_s$ is computed according to 10.13.4.2, the value of Q computed using $\sum P_u$ for 1.4 dead load plus 1.7 live load shall not exceed 0.60;

(c) When $\delta_s M_s$ is computed from 10.13.4.3, δ_s computed using $\sum P_u$ and

$\sum P_c$ corresponding to the factored dead and live loads shall be positive and shall not exceed 2.5.

In cases (a), (b), and (c) above, β_d shall be taken as the ratio of the maximum factored sustained axial load to the maximum factored axial load.

10.13.7 In sway frames, flexural members shall be designed for the total magnified end moments of the compression members at the joint.

SECTION 10.14
AXIALLY LOADED MEMBERS SUPPORTING SLAB SYSTEM

10.14.1 Axially loaded members supporting a slab system included within the scope of Section 13.1 shall be designed as provided in Chapter 10 and in accordance with the additional requirements of Chapter 13.

SECTION 10.15
TRANSMISSION OF COLUMN LOADS THROUGH FLOOR SYSTEM

When the specified compressive strength of concrete in a column is greater than 1.4 times that specified for a floor system, transmission of load through the floor system shall be provided by Section 10.15.1, 10.15.2, or 10.15.3.

10.15.1 Concrete of strength specified for the column shall be placed in the floor at the column location. Top surface of the column concrete shall extend 600 mm into the slab from face of column. Column concrete shall be well integrated with floor concrete, and shall be placed in accordance with Section 6.4.5 and 6.4.6.

10.15.2 Strength of a column through a floor system shall be based on the lower value of concrete strength with vertical dowels and spirals as required.

10.15.3 For columns laterally supported on four sides by beams of approximately equal depth or by slabs, it shall be permitted to base strength of the column on an assumed concrete strength in the column joint equal to 75 percent of column concrete strength plus 35 percent of floor concrete strength. In the application of Section 10.15.3, the ratio of column concrete strength to slab concrete strength shall not be taken greater than 2.5 for design.

SECTION 10.16
COMPOSITE COMPRESSION MEMBERS

10.16.1 Composite compression members shall include all such members reinforced longitudinally with structural steel shapes, pipe, or tubing with or without longitudinal bars.

10.16.2 Strength of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

10.16.3 Any axial load strength assigned to concrete of a composite member shall be transferred to the concrete by members or brackets in direct bearing on the composite member concrete.

10.16.4 All axial load strength not assigned to concrete of a composite member shall be developed by direct connection to the structural steel shape, pipe, or tube.

10.16.5 For evaluation of slenderness effects, radius of gyration of a composite section shall be not greater than the value given by

$$r = \sqrt{\frac{(E_c I_g / 5) + E_s I_t}{(E_c A_g / 5) + E_s A_t}} \qquad (10\text{-}20)$$

and, as an alternative to a more accurate calculation, EI in Eq. (10-10) shall be taken either as Eq. (10-11) or

$$EI = \frac{(E_c I_g / 5)}{1 + \beta_d} + E_s I_t \qquad (10\text{-}21)$$

10.16.6 **Structural steel encased concrete core**

10.16.6.1 For a composite member with a concrete core encased by structural steel, the thickness of the steel encasement shall be not less than

$$b\sqrt{\frac{f_y}{3E_s}} \text{ for each face of width } b$$

or

$$h\sqrt{\frac{f_y}{8E_s}} \text{ for circular sections of diameter } h$$

10.16.6.2 Longitudinal bars located within the encased concrete core shall be permitted to be used in computing A_t and I_t.

10.16.7 **Spiral reinforcement around structural steel core.** A composite member with spirally reinforced concrete around a structural steel core shall conform to Section 10.16.7.1 through 10.16.7.5.

10.16.7.1 Specified compressive strength of concrete f_c' shall not be less than that given in 1.1.1.

10.16.7.2 Design yield strength of structural steel core shall be the specified minimum yield strength for the grade of structural steel used but not to exceed 350 MPa.

10.16.7.3 Spiral reinforcement shall conform to 10.9.3.

10.16.7.4 Longitudinal bars located within the spiral shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.16.7.5 Longitudinal bars located within the spiral shall be permitted to be used in computing A_t and I_t.

10.16.8 **Tie reinforcement around structural steel core.** A composite member with laterally tied concrete around a structural steel core shall conform to 10.16.8.1 through 10.16.8.8.

10.16.8.1 Specified compressive strength of concrete f_c' shall not be less than that given in 1.1.1.

10.16.8.2 Design yield strength of structural steel core shall be the specified minimum yield strength for the grade of structural steel used but not to exceed 350 MPa.

10.16.8.3 Lateral ties shall extend completely around the structural steel core.

10.16.8.4 Lateral ties shall have a diameter not less than 0.02 times the greatest side dimension of composite member, except that ties shall not be smaller than Dia. 10 mm and are not required to be larger than Dia. 16 mm. Welded wire fabric of equivalent area shall be permitted.

10.16.8.5 Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or 0.5 times the least side dimension of the composite member.

10.16.8.6 Longitudinal bars located within the ties shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.16.8.7 A longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not farther apart than one-half the least side dimension of the composite member.

10.16.8.8 Longitudinal bars located within the ties shall be permitted to be used in computing A_t for strength but not in computing I_t for evaluation of slenderness effects.

SECTION 10.17
BEARING STRENGTH

10.17.1 Design bearing strength of concrete shall not exceed $\phi\left(0.85 f_c^{'} A_1\right)$, except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area shall be permitted to be multiplied by $\sqrt{A_2 / A_1}$ but not more than 2.

10.17.2 Section 10.17 does not apply to post-tensioning anchorages.

CHAPTER 11
SHEAR AND TORSION

SECTION 11.0
NOTATION

a = shear span, distance between concentrated load and face of support, mm
A_c = area of concrete section resisting shear transfer mm^2
A_{cp} = area enclosed by outside perimeter of concrete cross section, mm^2. See 11.6.1
A_f = area of reinforcement in bracket or corbel resisting factored moment $[V_u a + N_{uc}(h-d)]$, mm^2
A_g = gross area of section, mm^2. For a hollow section, A_g is the area of the concrete only and does not include the area of the void(s). See 11.6.1
A_h = area of shear reinforcement parallel to flexural tension reinforcement, mm^2
A_ℓ = total area of longitudinal reinforcement to resist torsion, mm^2
A_n = area of reinforcement in bracket or corbel resisting tensile force N_{uc}, mm^2
A_o = gross area enclosed by shear flow path, mm^2
A_{oh} = area enclosed by centerline of the outermost closed transverse torsional reinforcement, mm^2
A_{ps} = area of prestressed reinforcement in tension zone, mm^2
A_s = area of nonprestressed tension reinforcement, mm^2
A_t = area of one leg of a closed stirrup resisting torsion within a distance s, mm^2
A_v = area of shear reinforcement within a distances, or area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s for deep flexural members, mm^2
A_{vf} = area of shear-friction reinforcement, mm^2
A_{vh} = area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2, mm^2
b = width of compression face of member, mm
b_o = perimeter of critical section for slabs and footings, mm
b_t = width of that part of cross section containing the closed stirrups resisting torsion, mm.
b_w = web width, or diameter of circular section, mm
b_1 = width of the critical section defined in 11.12.1.2 measured in the direction of the span for which moments are determined, mm
b_2 = width of the critical section defined in 11.12.1.2 measured in direction perpendicular to b_1, mm
c_1 = Size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm
c_2 = size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which moments are being determined, mm
d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, but need not be less than $0.80h$ for circular sections and prestressed members, mm
f_c' = specified compressive strength of concrete, MPa

$\sqrt{f_c'}$ = square root of specified compressive strength of concrete, MPa

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, MPa

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, MPa

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, MPa. (In a composite member, f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both pre-stress and moments resisted by precast member acting alone)

f_{pe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads, MPa

f_{pu} = specified tensile strength of prestressng steel, MPa

f_y = specified yield strength of nonprestressed reinforcement, MPa

f_{yh} = specified yield strength of circular tie, hoop, or spiral reinforcement, MPa

f_{yv} = yield strength of closed transverse torsional reinforcement, MPa

$f_{y\ell}$ = yield strength of longitudinal torsional reinforcement, MPa

h = overall thickness of member, mm

h_v = total depth of shearhead cross section, mm

h_w = total height of wall from base to top, mm

I = moment of inertia of section resisting externally applied factored loads, mm^4

ℓ_n = clear span measured face-to-face of supports, mm

ℓ_v = length of shearhead arm from centroid of concentrated load or reaction, mm

ℓ_w = horizontal length of wall, mm

M_{cr} = moment causing flexural cracking at section due to externally applied loads. See 11.4.2.1

M_m = modified moment, N-mm

$M_{\max}$ = maximum factored moment at section due to externally applied loads, N-mm

M_p = required plastic moment strength of shear-head cross section, N-mm

M_u = factored moment at section, N-mm

M_v = moment resistance contributed by shearhead reinforcement, N-mm

N_u = factored axial load normal to cross section occurring simultaneously with V_u $or\ T_u$ to be taken as positive for compression, N

N_{uc} = factored tensile force applied at top of bracket or corbel acting simultaneously with V_u ,to be taken as positive for tension, N

p_{cp} = outside perimeter of the concrete cross section, mm. See 11.6.1

p_h = perimeter of centerline of outermost closed transverse torsional reinforcement, mm

s = spacing of shear or torsion reinforcement measured in a direction parallel to longitudinal reinforcement, mm

s_1 = spacing of vertical reinforcement in wall, mm

s_2 = spacing of shear or torsion reinforcement measured in a direction perpendicular to longitudinal reinforcement or spacing of horizontal reinforcement in wall, mm

t = thickness of a wall of a hollow section, mm

T_n = nominal torsional moment strength, N-mm

T_u = factored torsional moment at section, N-mm

V_c = nominal shear strength provided by concrete,

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, N

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web, N

V_d = shear force at section due to unfactored dead load, N

V_l = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}, N

V_n = nominal shear strength, N

V_p = vertical component of effective prestress force at section, N

V_s = nominal shear strength provided by shear reinforcement, N

V_u = factored shear force at section, N

v_n = nominal shear stress, MPa. See 11.12.6.2

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, mm

α = angle between inclined stirrups and longitudinal axis of member

α_f = angle between shear-friction reinforcement and shear plane

α_s = constant used to compute V_c in slabs and footings

α_v = ratio of flexural stiffness of shearhead arm to that of the surrounding composite slab section. See 11.12.4.5

β_c = ratio of long side to short side of concentrated load or reaction area

β_p = constant used to compute V_c in prestressed slabs

γ_f = fraction of unbalanced moment transferred by flexure at slab-column connections. See 13.5.3.2

γ_v = fraction of unbalanced moment transferred by eccentricity of shear at slab-column connections. See 11.12.6.1

= $1-\gamma_f$

η = number of identical arms of shearhead

θ = angle of compression diagonals in truss analogy for torsion

λ = correction factor related to unit weight of concrete

μ = coefficient of friction. See 11.7.4.3

ρ = ratio of nonprestressed tension reinforcement

= A_s / bd

ρ_h = ratio of horizontal shear reinforcement area to gross concrete area of vertical section

ρ_n = ratio of vertical shear reinforcement area to gross concrete area of horizontal section

ρ_w = $A_s / b_w d$

ϕ = strength reduction factor. See 9.3

SECTION 11.1
SHEAR STRENGTH

11.1.1 Except for members designed in accordance with Appendix A, design of cross sections subject to shear shall be based on:

$$\phi V_n \geq V_u \qquad (11\text{-}1)$$

where V_u is the factored shear force at the section considered and V_n is nominal shear strength computed by:

$$V_n = V_c + V_s \qquad (11\text{-}2)$$

where V_c is nominal shear strength provided by concrete in accordance with Section 11.3, 11.4, or 11.12, and V_s is nominal shear strength provided by shear reinforcement in accordance with Section 11.5.6, 11.10.9, or 11.12.

11.1.1.1 In determining shear strength V_n the effect of any openings in members shall be considered.

11.1.1.2 In determining shear strength V_c whenever applicable, effects of axial tension due to creep and shrinkage in restrained members shall be considered and effects of inclined flexural compression in variable depth members shall be permitted to be included.

11.1.2 The values of $\sqrt{f_c^{'}}$ used in this chapter shall not exceed 25/3 MPa except as allowed in 11.1.2.1.

11.1.2.1 Values of $\sqrt{f_c^{'}}$ greater than 25/3 MPa shall be permitted in computing V_c, V_{ci}; and V_{cw} for reinforced or prestressed concrete beams and concrete joist construction having minimum web reinforcement in accordance with Section 11.5.5.3, 11.5.5.4, or 11.6.5.2.

11.1.3 Computation of maximum factored shear force V_u at supports in accordance with Section 11.1.3.1 or 11.1.3.2 shall be permitted when all of the following conditions are satisfied:

(a) Support reaction, in direction of applied shear, introduces compression into the end regions of member;

(b) Loads are applied at or near the top of the member

No concentrated load occurs between face of support and location of critical section defined in Section 11.1.3.1 or 11.1.3.2.

11.1.3.1 For nonprestressed members, sections located less than a distance d from face of support shall be permitted to be designed for the same shear V_u as that computed at a distance d.

11.1.3.2 For prestressed members, sections located less than a distance $h/2$ from face of support shall be permitted to be designed for the same shear V_u as that computed at a distance $h/2$.

11.1.4 For deep beams, brackets and corbels, walls, and slabs and footings, the special provisions of Section 11.8 through 11.12 shall apply.

SECTION 11.2
LIGHTWEIGHT CONCRETE

11.2.1 Provisions for shear and torsion strength apply to normalweight concrete. When lightweight aggregate concrete is used, one of the following modifications shall apply to $\sqrt{f_c'}$ throughout Chapter 11, except Section 11.5.4.3, 11.5.6.9, 11.6.3.1, 11.12.3.2, and 11.12.4.8.

11.2.1.1 When f_{ct} is specified and concrete is proportioned in accordance with 5.2, $1.8\, f_{ct}$ shall be substituted for $\sqrt{f_c'}$ but the value of $1.8\, f_{ct}$ shall not exceed $\sqrt{f_c'}$.

11.2.1.2 When f_{ct} is not specified, all values of $\sqrt{f_c'}$ shall be multiplied by 0.75 for all-lightweight concrete and 0.85 for sand-lightweight concrete. Linear interpolation shall be permitted when partial sand replacement is used.

SECTION 11.3
SHEAR STRENGTH PROVIDED BY CONCRETE FOR NONPRESTRESSED MEMBERS

11.3.1 Shear strength V_c shall be computed by provisions of Section 11.3.1.1 through 11.3.1.3, unless a more detailed calculation is made in accordance with Section 11.3.2.

11.3.1.1 For members subject to shear and flexure only,

$$V_c = \frac{\sqrt{f_c'}}{6} b_w d \qquad (11\text{-}3)$$

11.3.1.2 For members subject to axial compression,

$$V_c = \left(1 + \frac{N_u}{14 A_g}\right)\left(\frac{\sqrt{f_c'}}{6}\right) b_w d \qquad (11\text{-}4)$$

Quantity N_u / A_g shall be expressed in MPa.

11.3.1.3 For members subject to significant axial tension, shear reinforcement shall be designed to carry total shear unless a more detailed analysis is made using Section 11.3.2.3.

11.3.2 Shear strength V_c shall be permitted to be computed by the more detailed calculation of Section 11.3.2.1 through 11.3.2.3.

11.3.2.1 For members subject to shear and flexure only,

$$V_c = \left(\sqrt{f_c'} + 120 \rho_w \frac{V_u d}{M_u}\right) \frac{b_w d}{7} \qquad (11\text{-}5)$$

but not greater than $0.3 \times \sqrt{f_c'} b_w d$. Quantity $V_u d / M_u$ shall not be taken greater than 1.0 in computing V_c by Eq. (11-5), where M_u is factored moment occurring simultaneously with V_u at section considered.

11.3.2.2 For members subject to axial compression, it shall be permitted to compute V_c using Eq. (11-5) with M_m substituted for M_u and $V_u d / M_u$ not then limited to 1.0, where

$$M_m = M_u - N_u \frac{(4h - d)}{8} \qquad (11\text{-}6)$$

However, V_c shall not be taken greater than

$$V_c = 0.3 \times \sqrt{f_c^{'}} \cdot b_w d \sqrt{1 + \frac{0.3N_u}{A_g}} \tag{11-7}$$

Quantity N_u / A_g shall be expressed in MPa. When M_m as computed by Eq. (11-6) is negative, V_c shall be computed by Eq. (11-7).

11.3.2.3 For members subject to significant axial tension,

$$V_c = \left(1 + \frac{0.3N_u}{A_g}\right)\frac{\sqrt{f_c^{'}}}{6} b_w d \tag{11-8}$$

but not less than zero, where N_u is negative for tension. Quantity N_u / A_g shall be expressed in MPa.

11.3.3 For circular members, the area used to compute V_c shall be taken as the product of the diameter and effective depth of the concrete section. It shall be permitted to take the effective depth as 0.8 times the diameter of the concrete section.

SECTION 11.4
SHEAR STRENGTH PROVIDED BY CONCRETE FOR PRESTRESSED MEMBERS

11.4.1 For members with effective prestress force not less than 40 percent of the tensile strength of flexural reinforcement, unless a more detailed calculation is made in accordance with Section 11.4.2,

$$V_c = \left(\frac{\sqrt{f_c^{'}}}{20} + 5\frac{V_u d}{M_u}\right) b_w d \tag{11-9}$$

but V_c need not be taken less than $(1/6)\sqrt{f_c^{'}} b_w d$ nor shall V_c be taken greater than $0.4\sqrt{f_c^{'}} b_w d$ nor the value given in Section 11.4.3 or 11.4.4. The quantity $V_u d / M_u$ shall not be taken greater than 1.0, where M_u, is factored moment occurring simultaneously with V_u at the section considered. When applying Eq. (11-9), d in the term $V_u d / M_u$ shall be the distance from extreme compression fiber to centroid of prestressed reinforcement.

11.4.2 Shear strength V_c shall be permitted to be computed in accordance with Sections 11.4.2.1 and 11.4.2.2, where V_c shall be the lesser of V_{ci} or V_{cw}.

11.4.2.1 Shear strength V_{ci} shall be computed by

$$V_{ci} = \frac{\sqrt{f_c^{'}}}{20} b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \tag{11-10}$$

but V_{ci} need not be taken less than $1/7\sqrt{f_c^{'}} b_w d$, where

$$M_{cr} = (I / y_t) \cdot \left(\frac{\sqrt{f_c'}}{2} + f_{pe} - f_d \right) \tag{11-11}$$

and values of $M_{\max}$ and V_i shall be computed from the load combination causing maximum moment to occur at the section.

11.4.2.2 Shear strength V_{cw} shall be computed by

$$V_{cw} = 0.3\left(\sqrt{f_c'} + f_{pc} \right) b_w d + V_p \tag{11-12}$$

Alternatively, V_{cw} shall be computed as the shear force corresponding to dead load plus live load that results in a principal tensile stress of $(1/3)\sqrt{f_c'}$ at the centroidal axis of member, or at the intersection of flange and web when the centroidal axis is in the flange. In composite members, the principal tensile stress shall be computed using the cross section that resists live load.

11.4.2.3 In Eq. (11-10) and (11-12), d shall be the distance from extreme compression fiber to centroid of prestressed reinforcement or $0.8h$, whichever is greater.

11.4.3 In a pretensioned member in which the section at a distance $h/2$ from face of support is closer to the end of member than the transfer length of the prestressing steel, the reduced prestress shall be considered when computing V_{cw}. This value of V_{cw} shall also be taken as the maximum limit for Eq. (11-9). The pre-stress force shall be assumed to vary linearly from zero at end of the prestressing steel, to a maximum at a distance from end of the prestressing steel equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

11.4.4 In a pretensioned member where bonding of some tendons does not extend to the end of member, a reduced prestress shall be considered when computing V_c in accordance with Section 11.4.1 or 11.4.2. The value of V_{cw} calculated using the reduced prestress shall also be taken as the maximum limit for Eq. (11-9). The prestress force due to tendons for which bonding does not extend to the end of member shall be assumed to vary linearly from zero at the point at which bonding commences to a maximum at a distance from this point equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

SECTION 11.5
SHEAR STRENGTH PROVIDED BY SHEAR REINFORCEMENT

11.5.1 Types of shear reinforcement

11.5.1.1 Shear reinforcement consisting of the following shall be permitted:

(a) Stirrups perpendicular to axis of member;

(b) Welded wire fabric with wires located perpendicular to axis of member;

(c) Spirals, circular ties, or hoops.

11.5.1.2 For nonprestressed members, shear reinforcement shall be permitted to also consist of:

(a) Stirrups making an angle of 45 deg or more with longitudinal tension reinforcement;

(b) Longitudinal reinforcement with bent portion making an angle of 30 deg or more with the longitudinal tension reinforcement;

(c) Combinations of stirrups and bent longitudinal reinforcement.

11.5.2 Design yield strength of shear reinforcement shall not exceed 420 MPa, except that the design yield strength of welded deformed wire fabric shall not exceed 550 MPa.

11.5.3 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fiber and shall be anchored at both ends according to 12.13 to develop the design yield strength of reinforcement.

11.5.4 Spacing limits for shear reinforcement

11.5.4.1 Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/2$ in nonprestressed members or $0.75h$ in prestressed members, nor 500 mm.

11.5.4.2 Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45 deg line, extending toward the reaction from mid-depth of member $d/2$ to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

11.5.4.3 When V_s exceeds $(1/3)\sqrt{f_c^{'}}b_w d$, maximum spacings given in Section 11.5.4.1 and 11.5.4.2 shall be reduced by one-half.

11.5.5 Minimum shear reinforcement

11.5.5.1 A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where factored shear force V_u exceeds one-half the shear strength provided by concrete ϕV_c except:

(a) Slabs and footings;

(b) Concrete joist construction defined by 8.11;

(c) Beams with total depth not greater than 250 mm, 2.5 times thickness of flange, or 0.5 the width of web, whichever is greatest.

11.5.5.2 Minimum shear reinforcement requirements of Section 11.5.5.1 shall be permitted to be waived if shown by test that required nominal flexural and shear strengths can be developed when shear reinforcement is omitted. Such tests shall simulate effects of differential settlement, creep, shrinkage, and temperature change, based on a realistic assessment of such effects occurring in service.

11.5.5.3 Where shear reinforcement is required by Section 11.5.5.1 or for strength and where 11.6.1 allows torsion to be neglected, the minimum area of shear reinforcement for prestressed (except as provided in 11.5.5.4) and nonprestressed members shall be computed by

$$A_v = \frac{1}{16}\sqrt{f_c^{'}}\frac{b_w s}{f_y} \qquad (11\text{-}13)$$

but shall not be less than $0.33 b_w s / f_y$ where b_w and s are in mm.

11.5.5.4 For prestressed members with an effective prestress force not less than 40 percent of the tensile strength of the flexural reinforcement, the area of shear reinforcement shall not be less than the smaller A_v from Eq. (11-13) or (11-14)

$$A_v = \frac{A_{ps} f_{pu} s}{80 f_y d} \sqrt{\frac{d}{b_w}} \tag{11-14}$$

11.5.6 **Design of shear reinforcement**

11.5.6.1 Where factored shear force V_u, exceeds shear strength ϕV_c, shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where shear strength V_s shall be computed in accordance with 11.5.6.2 through 11.5.6.9.

11.5.6.2 When shear reinforcement perpendicular to axis of member is used,

$$V_s = \frac{A_v f_y d}{s} \tag{11-15}$$

where A_v is the area of shear reinforcement within a distance s.

11.5.6.3 When circular ties, hoops, or spirals are used as shear reinforcement, V_s shall be computed using Eq. (11-15) where d shall be taken as the effective depth defined in Section 11.3.3. A_v shall be taken as two times the area of the bar in a circular tie, hoop, or spiral at a spacing s, and f_{yh} is the specified yield strength of circular tie, hoop, or spiral reinforcement.

11.5.6.4 When inclined stirrups are used as shear reinforcement,

$$V_s = \frac{A_v f_y \left(\sin\alpha + \cos\alpha\right) d}{s} \tag{11-16}$$

11.5.6.5 When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support,

$$V_s = A_v f_y \sin\alpha \tag{11-17}$$

but not greater, than, $(1/4)\sqrt{f_c'} b_w d$.

11.5.6.6 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, shear strength V_s shall be computed by Eq. (11-16).

11.5.6.7 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

11.5.6.8 Where more than one type of shear reinforcement is used to reinforce the same portion of a member, shear strength V_s shall be computed as the sum of the V_s values computed for the various types.

11.5.6.9 Shear strength V_s shall not be taken greater than $(2/3)\sqrt{f_c'} b_w d$.

SECTION 11.6
DESIGN FOR TORSION

11.6.1 **Threshold torsion.** It shall be permitted to neglect torsion effects when the factored torsional moment T_u is less than:

(a) For nonprestressed members:

$$\frac{\phi\sqrt{f_c'}}{12}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)$$

(b) For prestressed members:

$$\frac{\phi\sqrt{f_c'}}{12}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)\sqrt{1+\frac{3f_{pc}}{\sqrt{f_c'}}}$$

(c) For nonprestressed members subjected to an axial tensile or compressive force:

$$\frac{\phi\sqrt{f_c'}}{3}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)\sqrt{1+\frac{3N_u}{A_g\sqrt{f_c'}}}$$

For members cast monolithically with a slab, the overhanging flange width used in computing A_{cp} and p_{cp} shall conform to 13.2.4. For a hollow section, A_g shall be used in place of A_{cp} in 11.6.1, and the outer boundaries of the section shall conform to 13.2.4.

11.6.1.1 For isolated members with flanges and for members cast monolithically with a slab, the overhanging flange width used in computed A_{cp} and p_{cp} shall conform to 13.2.4, except that the overhanging flanges shall be neglected in cases where the parameter A_{cp}^2 / p_{cp} calculated for a beam with flanges is less than that computed for the same beam ignoring the flanges.

11.6.2 Calculation of factored torsional moment T_u

11.6.2.1 If the factored torsional moment T_u in a member is required to maintain equilibrium and exceeds the minimum value given in 11.6.1, the member shall be designed to carry that torsional moment in accordance with 11.6.3 through 11.6.6.

11.6.2.2 In a statically indeterminate structure where reduction of the torsional moment in a member can occur due to redistribution of internal forces upon cracking, the maximum factored torsional moment T_u shall be permitted to be reduced to the values given in (a), (b), or (c), as applicable:

(a) For nonprestressed members, at the sections described in 11.6.2.4:

$$\frac{\phi\sqrt{f_c'}}{3}\left(\frac{A_{cp}^2}{p_{cp}}\right)$$

(b) For prestressed members, at the sections described in 11.6.2.5:

$$\frac{\phi\sqrt{f_c'}}{3}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)\sqrt{1+\frac{3f_{pc}}{\sqrt{f_c'}}}$$

(c) For nonprestressed members subjected to an axial tensile or compressive force:

$$\frac{\phi\sqrt{f_c'}}{3}\left(\frac{A^2{}_{cp}}{p_{cp}}\right)\sqrt{1+\frac{3N_u}{A_g\sqrt{f_c'}}}$$

In (a), (b), or (c), the correspondingly redistributed bending moments and shears in the adjoining members shall be used in the design of these members. For hollow sections, A_{cp} shall not be replaced with A_g in 11.6.2.2.

11.6.2.3 Unless determined by a more exact analysis, it shall be permitted to take the torsional loading from a slab as uniformly distributed along the member.

11.6.2.4 In nonprestressed members, sections located less than a distance d from the face of a support shall be designed for not less than the torsion T_u computed at a distance d. If a concentrated torque occurs within this distance, the critical section for design shall be at the face of the support.

11.6.2.5 In prestressed members, sections located less than a distance $h/2$ from the face of a support shall be designed for not less than the torsion T_u computed at a distance $h/2$. If a concentrated torque occurs within this distance, the critical section for design shall be at the face of the support.

11.6.3 Torsional moment strength

11.6.3.1 The cross-sectional dimensions shall be such that:

(a) For solid sections:

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2+\left(\frac{T_u p_h}{1.7A_{oh}^2}\right)^2}\le\phi\left(\frac{V_c}{b_w d}+\frac{2}{3}\sqrt{f_c'}\right) \qquad (11\text{-}18)$$

(b) For hollow sections:

$$\left(\frac{V_u}{b_w d}\right)^2+\left(\frac{T_u p_h}{1.7A_{oh}^2}\right)^2\le\phi\left(\frac{V_c}{b_w d}+\frac{2}{3}\sqrt{f_c'}\right) \qquad (11\text{-}19)$$

11.6.3.2 If the wall thickness varies around the perimeter of a hollow section, Eq. (11-19) shall be evaluated at the location where the left-hand side of Eq. (11-19) is a maximum.

11.6.3.3 If the wall thickness is less than A_{oh}/p_h, the second term in Eq. (11-19) shall be taken as:

$$\left(\frac{T_u}{1.7A_{oh}t}\right)$$

where t is the thickness of the wall of the hollow section at the location where the stresses are being checked.

11.6.3.4 Design yield strength of nonprestressed torsion reinforcement shall not exceed 420 MPa.

11.6.3.5 The reinforcement required for torsion shall be determined from:

$$\phi T_n \geq T_u \qquad (11\text{-}20)$$

11.6.3.6 The transverse reinforcement for torsion shall be designed using:

$$T_n = \frac{2A_o A_t f_{yv}}{s} \cot\theta \qquad (11\text{-}21)$$

where A_o shall be determined by analysis except that it shall be permitted to take A_o equal to $0.85A_{oh}$; θ shall not be taken smaller than 30 deg nor larger than 60 deg. It shall be permitted to take θ equal to:

(a) 45 deg for nonprestressed members or members with less prestress than in (b); or

(b) 37.5 deg for prestressed members with an effective prestress force not less than 40 percent of the tensile strength of the longitudinal reinforcement.

11.6.3.7 The additional longitudinal reinforcement required for torsion shall not be less than:

$$A_\ell = \frac{A_t}{s} p_h \left(\frac{f_{yv}}{f_{y\ell}} \right) \cot^2\theta \qquad (11\text{-}22)$$

where θ shall be the same value used in Eq. (11-21) and A_t / s shall be taken as the amount computed from Eq. (11-21) not modified in accordance with Section 11.6.5.2 or 11.6.5.3.

11.6.3.8 Reinforcement required for torsion shall be added to that required for the shear, moment and axial force that act in combination with the torsion. The most restrictive requirements for reinforcement spacing and placement shall be met.

11.6.3.9 It shall be permitted to reduce the area of longitudinal torsion reinforcement in the flexural compression zone by an amount equal to $M_u / (0.9 d f_{y\ell})$ where M_u is the factored moment acting at the section in combination with T_u except that the reinforcement provided shall not be less than that required by Section 11.6.5.3 or 11.6.6.2.

11.6.3.10 In prestressed beams:

(a) The total longitudinal reinforcement including prestressing steel at each section shall resist the factored bending moment at that section plus an additional concentric longitudinal tensile force equal to $A_\ell f_{y\ell}$ based on the factored torsion at that section;

(b) The spacing of the longitudinal reinforcement including tendons shall satisfy the requirements in Section 11.6.6.2.

11.6.3.11 In prestressed beams, it shall be permitted to reduce the area of longitudinal torsional reinforcement on the side of the member in compression due to flexure below that required by Section 11.6.3.10 in accordance with Section 11.6.3.9.

11.6.4 Details of torsional reinforcement

11.6.4.1 Torsion reinforcement shall consist of longitudinal bars or tendons and one or more of the following:

(a) Closed stirrups or closed ties, perpendicular to the axis of the member;

(b) A closed cage of welded wire fabric with transverse wires perpendicular to the axis of the member;

(c) In nonprestressed beams, spiral reinforcement.

11.6.4.2 Transverse torsional reinforcement shall be anchored by one of the following:

(a) A 135 deg standard hook around a longitudinal bar;

(b) According to Section 12.13.2.1, 12.13.2.2, or 12.13.2.3 in regions where the concrete surrounding the anchorage is restrained against spalling by a flange or slab or similar member.

11.6.4.3 Longitudinal torsion reinforcement shall be developed at both ends.

11.6.4.4 For hollow sections in torsion, the distance from the centerline of the transverse torsional reinforcement to the inside face of the wall of the hollow section shall not be less than $0.5A_{oh}/p_h$.

11.6.5 Minimum torsion reinforcement

11.6.5.1 A minimum area of torsion reinforcement shall be provided in all regions where the factored torsional moment T_u exceeds the values specified in 11.6.1.

11.6.5.2 Where torsional reinforcement is required by 11.6.5.1, the minimum area of transverse closed stirrups shall be computed by:

$$\left(A_v + 2A_t\right) = \frac{1}{16}\sqrt{f_c'}\,\frac{b_w s}{f_{yv}} \qquad (11\text{-}23)$$

but shall not be less than $\left(0.33 b_w s\right)/f_{yv}$.

11.6.5.3 Where torsional reinforcement is required by 11.6.5.1, the minimum total area of longitudinal torsional reinforcement shall be computed by:

$$A_{\ell,\min} = \frac{5\sqrt{f_c'}A_{cp}}{12 f_{y\ell}} - \left(\frac{A_t}{s}\right)p_h \frac{f_{yv}}{f_{y\ell}} \qquad (11\text{-}24)$$

where A_t/s shall not be taken less than $\left(1/6\right)b_w/f_{yv}$.

11.6.6 Spacing of torsion reinforcement

11.6.6.1 The spacing of transverse torsion reinforcement shall not exceed the smaller of $p_h/8$ or 300 mm.

11.6.6.2 The longitudinal reinforcement required for torsion shall be distributed around the perimeter of the closed stirrups with a maximum spacing of 300 mm. The longitudinal bars or tendons shall be inside the stirrups. There shall be at least one longitudinal bar or tendon in each corner of the stirrups. Bars shall have a diameter at least 0.042 times the stirrup spacing, but not less than a Dia. 10 mm bar.

11.6.6.3 Torsion reinforcement shall be provided for a distance of at least $(b_t + d)$ beyond the point theoretically required.

SECTION 11.7
SHEAR-FRICTION

11.7.1 Provisions of Section 11.7 are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

11.7.2 Design of cross sections subject to shear transfer as described in 11.7.1 shall be based on Eq. (11-1), where V_n is calculated in accordance with provisions of Section 11.7.3 or 11.7.4.

11.7.3 A crack shall be assumed to occur along the shear plane considered. The required area of shear-friction reinforcement A_{vf} across the shear plane shall be designed using either 11.7.4 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests.

11.7.3.1 Provisions of Section 11.7.5 through 11.7.10 shall apply for all calculations of shear transfer strength.

11.7.4 Shear-friction design method

11.7.4.1 When shear-friction reinforcement is perpendicular to the shear plane, shear strength V_n shall be computed by

$$V_n = A_{vf} f_y \mu \tag{11-25}$$

where μ is coefficient of friction in accordance with 11.7.4.3.

11.7.4.2 When shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in shear-friction reinforcement, shear strength V_n shall be computed by

$$V_n = A_{vf} f_y \left(\mu \sin \alpha_f + \cos \alpha_f\right) \tag{11-26}$$

where α_f is angle between shear-friction reinforcement and shear plane.

11.7.4.3 The coefficient of friction μ in Eq. (11-25) and Eq. (11-26) shall be

Concrete placed monolithically	1.4λ
Concrete placed against hardened concrete with surface intentionally roughened as specified in 11.7.9	1.0λ
Concrete placed against hardened concrete not intentionally roughened	0.6λ
Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see 11.7.10)	0.7λ

where $\lambda = 1.0$ for normalweight concrete, 0.85 for sand-lightweight concrete and 0.75 for all lightweight concrete. Linear interpolation shall be permitted when partial sand replacement is used.

11.7.5 Shear strength V_n shall not be taken greater than $0.2 f_c' A_c$, nor $5.5 A_c$, in N, where A_c, is area of concrete section resisting shear transfer.

11.7.6 Design yield strength of shear-friction reinforcement shall not exceed 420 MPa.

11.7.7 Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane shall be permitted to be taken as additive to the force in the shear-friction reinforcement $A_{vf} f_y$ when calculating required A_{vf}.

11.7.8 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop the specified yield strength on both sides by embedment, hooks, or welding to special devices.

11.7.9 For the purpose of 11.7, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ interface shall be roughened to a full amplitude of approximately 5 mm.

11.7.10 When shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint.

SECTION 11.8
DEEP BEAMS

11.8.1 The provisions of 11.8 shall apply to members with clear spans, ℓ_n, equal to or less than four times the overall member depth or regions of beams loaded with concentrated loads within twice the member depth from the support that are loaded on one face and supported on the opposite face so that compression struts can develop between the loads and supports. See also Section 12.10.6.

11.8.2 Deep beams shall be designed using either nonlinear analysis as permitted in 10.7.1, or Appendix A.

11.8.3 Shear strength V_n for deep beams shall not exceed $(5/6)\sqrt{f_c'} b_w d$.

11.8.4 The area of shear reinforcement perpendicular to the span, A_v shall not be less than $0.0025 b_w s$, and s shall not exceed $d/5$, nor 300 mm.

11.8.5 The area of shear reinforcement parallel to the span, A_{vh}, shall not be less than $0.0015 b_w s_2$, and s_2 shall not exceed $d/5$, nor 300 mm.

11.8.6 It shall be permitted to provide reinforcement satisfying A.3.3 instead of the minimum horizontal and vertical reinforcement specified in Section 11.8.4 and 11.8.5.

SECTION 11.9
SPECIAL PROVISIONS FOR BRACKETS AND CORBELS

11.9.1 Brackets and corbels with a shear span-to-depth ratio a/d less than 2 shall be permitted to be designed using Appendix A. Design shall be permitted using Section 11.9.3 and 11.9.4 for brackets and corbels with:

(a) a/d not greater than 1, and

(b) subject to horizontal tensile force for N_{uc} not larger than V_u.

The requirements of Section 11.9.2, 11.9.3.2.1, 11.9.3.2.2, 11.9.5, 11.9.6, and 11.9.7 shall apply to design of brackets and corbels. Distance d shall be measured at the face of the support.

11.9.2 Depth at outside edge of bearing area shall not be less than $0.5d$.

11.9.3 Section at face of support shall be designed to resist simultaneously a shear V_u, a moment $[V_u \mathrm{a} + N_{uc}(h-d)]$, and a horizontal tensile force N_{uc}.

11.9.3.1 In all design calculations in accordance with 11.9, strength reduction factor ϕ shall be taken equal to 0.75

11.9.3.2 Design of shear-friction reinforcement A_{vt} to resist shear V_u shall be in accordance with 11.7.

11.9.3.2.1 For normalweight concrete, shear strength V_n shall not be taken greater than $0.2 f_c' b_w d$ nor $5.5 b_w d$ in N.

11.9.3.2.2 For all-lightweight or sand-lightweight concrete, shear strength V_n shall not be taken greater than $(0.2 - 0.07\mathrm{a}/d)\, f_c' b_w d$ nor $(5.5 - 1.9\mathrm{a}/d) b_w d$, in N.

11.9.3.3 Reinforcement A_f to resist moment $[V_u \mathrm{a} + N_{uc}(h-d)]$ shall be computed in accordance with 10.2 and 10.3.

11.9.3.4 Reinforcement A_n to resist tensile force N_{uc} shall be determined from $N_{uc} \le \phi A_n f_y$. Tensile force N_{uc} shall not be taken less than $0.2V_u$ unless special provisions are made to avoid tensile forces. Tensile force N_{uc} shall be regarded as a live load even when tension results from creep, shrinkage, or temperature change.

11.9.3.5 Area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$ or $(2A_{vf}/3 + A_n)$.

11.9.4 Closed stirrups or ties parallel to A_s, with a total area A_n not less than $0.5(A_s - A_n)$, shall be uniformly distributed within two-thirds of the effective depth adjacent to A_s.

11.9.5 Ratio $\rho = A_s / bd$ shall not be less than $0.04(f_c' / f_y)$.

11.9.6 At front face of bracket or corbel, primary tension reinforcement A_s shall be anchored by one of the following:

(a) By a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars;

(b) By bending primary tension bars A_s back to form a horizontal loop; or

(c) By some other means of positive anchorage.

11.9.7 Bearing area of load on bracket or corbel shall not project beyond straight portion of primary tension bars A_s, nor project beyond interior face of transverse anchor bar (if one is provided).

SECTION 11.10
SPECIAL PROVISIONS FOR WALLS

11.10.1 Design for shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in 11.12. Design for horizontal in-plane shear forces in a wall shall be in accordance with 11.10.2 through 11.10.9. Alternatively, it shall be permitted to design walls with a height not exceeding two times the length of the wall for horizontal shear forces in accordance with Appendix A and Sections 11.10.9.2 through 11.10.9.5.

11.10.2 Design of horizontal section for shear in plane of wall shall be based on Eq. (11-1) and (11-2), where shear strength V_c shall be in accordance with 11.10.5 or 11.10.6 and shear strength V_s shall be in accordance with 11.10.9.

11.10.3 Shear strength V_n at any horizontal section for shear in plane of wall shall not be taken greater than $(5/6)\sqrt{f_c'}hd$.

11.10.4 For design for horizontal shear forces in plane of wall, d shall be taken equal to $0.8\ell_w$ larger value of d, equal to the distance from extreme compression fiber to center of force of all reinforcement in tension, shall be permitted to be used when determined by a strain compatibility analysis.

11.10.5 Unless a more detailed calculation is made in accordance with 11.10.6, shear strength V_c shall not be taken greater than $(1/6)\sqrt{f_c'}hd$ for walls subject to N_u in compression, or V_c shall not be taken greater than the value given in 11.3.2.3 for walls subject to N_u in tension.

11.10.6 Shear strength V_c shall be permitted to be the lesser of the values computed from Eq. (11-29) or (11-30)

$$V_c = \frac{1}{4}\sqrt{f_c'}hd + \frac{N_u d}{4\ell_w} \qquad (11\text{-}29)$$

or

$$V_c = \left[0.5\sqrt{f_c'} + \frac{\ell_w \left(\sqrt{f_c'} + 2\frac{N_u}{\ell_w h} \right)}{\frac{M_u}{V_u} - \frac{\ell_w}{2}} \right] \frac{hd}{10} \tag{11-30}$$

where N_u is negative for tension. When $\left(M_u / V_u - \ell_w / 2 \right)$ is negative, Eq. (11-30) shall not apply.

11.10.7 Sections located closer to wall base than a distance $\ell_w / 2$ or one-half the wall height, whichever is less, shall be permitted to be designed for the same V_u as that computed at a distance $\ell_w / 2$ or one-half the height.

11.10.8 When factored shear force V_u is less than $\phi V_c / 2$ reinforcement shall be provided in accordance with Section 11.10.9 or in accordance with Chapter 14. When V_u exceeds $\phi V_c / 2$, wall reinforcement for resisting shear shall be provided in accordance with 11.10.9.

11.10.9 **Design of shear reinforcement for walls**

11.10.9.1 Where factored shear force V_u exceeds shear strength ϕV_c, horizontal shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where shear strength V_s shall be computed by

$$V_s = \frac{A_v f_y d}{s_2} \tag{11-31}$$

where A_v is area of horizontal shear reinforcement within a distance s_2 and distance d is in accordance with 11.10.4. Vertical shear reinforcement shall be provided in accordance with 11.10.9.4.

11.10.9.2 Ratio ρ_h of horizontal shear reinforcement area to gross concrete area of vertical section shall not be less than 0.0025.

11.10.9.3 Spacing of horizontal shear reinforcement s_2 shall not exceed $\ell_w / 5$, $3h$, nor 500 mm.

11.10.9.4 Ratio ρ_h of vertical shear reinforcement area to gross concrete area of horizontal section shall not be less than

$$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w} \right)\left(\rho_h - 0.0025 \right) \tag{11-32}$$

nor 0.0025, but need not be greater than the required horizontal shear reinforcement.

11.10.9.5 Spacing of vertical shear reinforcement s_1 shall not exceed $\ell_w / 3$, $3h$, nor 500 mm.

SECTION 11.11
TRANSFER OF MOMENTS TO COLUMNS

11.11.1 When gravity load, wind, earthquake, or other lateral forces cause transfer of moment at connections of framing elements to columns, the shear resulting from moment transfer shall be considered in the design of lateral reinforcement in the columns.

11.11.2 Except for connections not part of a primary seismic load-resisting system that are restrained on four sides by beams or slabs of approximately equal depth, connections shall have lateral reinforcement not less than that required by Eq. (11-13) within the column for a depth not less than that of the deepest connection of framing elements to the columns. See also 7.9.

SECTION 11.12
SPECIAL PROVISIONS FOR SLABS AND FOOTINGS

11.12.1 The shear strength of slabs and footings in the vicinity of columns, concentrated loads, or reactions is governed by the more severe of two conditions:

11.12.1.1 Beam action where each critical section to be investigated extends in a plane across the entire width. For beam action the slab or footing shall be designed in accordance with 11.1 through 11.5.

11.12.1.2 Two-way action where each of the critical sections to be investigated shall be located so that its perimeter b_o is a minimum but need not approach closer than $d/2$ to

(a) Edges or corners of columns, concentrated loads, or reaction areas; or

(b) Changes in slab thickness such as edges of capitals or drop panels.

For two-way action the slab or footing shall be designed in accordance with 11.12.2 through 11.12.6.

11.12.1.3 For square or rectangular columns, concentrated loads, or reaction areas, the critical sections with four straight sides shall be permitted.

11.12.2 The design of a slab or footing for two-way action is based on Eq. (11-1) and (11-2). V_c shall be computed in accordance with Section 11.12.2.1, 11.12.2.2, or 11.12.3.1. V_s shall be computed in accordance with Section 11.12.3. For slabs with shearheads, V_n shall be in accordance with Section 11.12.4. When moment is transferred between a slab and a column, 11.12.6 shall apply.

11.12.2.1 For nonprestressed slabs and footings, V_c shall be the smallest of (a), (b), and (c):

(a) $$V_c = \left(1 + \frac{2}{\beta_c}\right)\frac{\sqrt{f_c'}\,b_o d}{6} \qquad (11\text{-}33)$$

where β_c is the ratio of long side to short side of the column, concentrated load or reaction area;

(b) $V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\frac{\sqrt{f_c'} b_o d}{12}$ (11-34)

where α_s is 40 for interior columns, 30 for edge columns, 20 for corner columns; and

(c) $V_c = \frac{1}{3}\sqrt{f_c'} b_o d$ (11-35)

11.12.2.2 At columns of two-way prestressed slabs and footings that meet the requirements of 18.9.3

$$V_c = \left(\beta_p \sqrt{f_c'} + 0.3 f_{pc}\right) b_o d + V_p \quad (11\text{-}36)$$

where β_p, is the smaller of 0.29 or $(\alpha_s d / b_o + 1.5)/12$, α_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns, b_o is perimeter of critical section defined in 11.12.1.2, f_{pc} is the average value of f_{pc} for the two directions, and V_p is the vertical component of all effective prestress forces crossing the critical section. V_c shall be permitted to be computed by Eq. (11-36) if the following are satisfied; otherwise, Section 11.12.2.1 shall apply:

(a) No portion of the column cross section shall be closer to a discontinuous edge than 4 times the slab thickness;

(b) f_c' in Eq. (11-36) shall not be taken greater than 35 MPa; and

(c) f_{pc} in each direction shall not be less than 0.9 MPa, nor be taken greater than 3.5 MPa.

11.12.3 Shear reinforcement consisting of bars or wires and single- or multiple-leg stirrups shall be permitted in slabs and footings with an effective depth, d, greater than or equal to 150 mm, but not less than 16 times the shear reinforcement bar diameter. Shear reinforcement shall be in accordance with Section 11.12.3.1 through 11.12.3.4.

11.12.3.1 V_n shall be computed by Eq. (11-2), where V_c shall not be taken greater than $(1/6)\sqrt{f_c'} b_o d$, and the strength of shear reinforcement V_s shall be calculated in accordance with 11.5. The area of shear reinforcement A_v used in Eq. (11-15) is the cross-sectional area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the column section.

11.12.3.2 V_n shall not be taken greater than $(1/2)\sqrt{f_c'} b_o d$

11.12.3.3 The distance between the column face and the first line of stirrup legs that surround the column shall not exceed $d/2$. The spacing between adjacent stirrup legs in the first line of shear reinforcement shall not exceed $2d$ measured in a direction parallel to the column face. The spacing between successive lines of shear reinforcement that surround the column shall not exceed $d/2$ measured in a direction perpendicular to the column face.

11.12.3.4 Slab shear reinforcement shall satisfy the anchorage requirements of 12.13 and shall engage the longitudinal flexural reinforcement in the direction being considered.

11.12.4 Shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) shall be permitted in slabs. The provisions of Section 11.12.4.1 through 11.12.4.9 shall apply where shear due to gravity load is transferred at interior column supports. Where moment is transferred to columns, Section 11.12.6.3 shall apply.

11.12.4.1 Each shearhead shall consist of steel shapes fabricated by welding with a full penetration weld into identical arms at right angles. Shearhead arms shall not be interrupted within the column section.

11.12.4.2 A shearhead shall not be deeper than 70 times the web thickness of the steel shape.

11.12.4.3 The ends of each shearhead arm shall be permitted to be cut at angles not less than 30 deg with the horizontal, provided the plastic moment strength of the remaining tapered section is adequate to resist the shear force attributed to that arm of the shearhead.

11.12.4.4 All compression flanges of steel shapes shall be located within $0.3d$ of compression surface of slab.

11.12.4.5 The ratio α_v between the flexural stiffness of each shearhead arm and that of the surrounding composite cracked slab section of width $(c_2 + d)$ shall not be less than 0.15.

11.12.4.6 The plastic moment strength M_p required for each arm of the shearhead shall be computed by

$$M_p = \frac{V_u}{2\phi\eta}\left[h_v + \alpha_v\left(\ell_v - \frac{c_1}{2}\right)\right] \qquad (11\text{-}37)$$

where ϕ is the strength reduction factor for tension-controlled members, η is the number of arms, and ℓ_v is the minimum length of each shearhead arm required to comply with requirements of Section 11.12.4.7 and 11.12.4.8.

11.12.4.7 The critical slab section for shear shall be perpendicular to the plane of the slab and shall cross each shearhead arm at three-quarters the distance $[\ell_v - (c_1/2)]$ from the column face to the end of the shearhead arm. The critical section shall be located so that its perimeter b_o is a minimum, but need not be closer than the perimeter defined in 11.12.1.2(a).

11.12.4.8 V_n shall not be taken greater than $(1/3)\sqrt{f_c'}b_o d$ on the critical section defined in 11.12.4.7. When shearhead reinforcement is provided, V_n shall not be taken greater than $0.6\sqrt{f_c'}b_o d$ on the critical section defined in 11.12.1.2(a).

11.12.4.9 The moment resistance M_v contributed to each slab column strip by a shearhead shall not be taken greater than

$$M_v = \frac{\phi\alpha_v V_u}{2\eta}\left(\ell_v - \frac{c_1}{2}\right) \qquad (11\text{-}38)$$

where ϕ is the strength reduction factor for tension-controlled members, η is the number of arms, and ℓ_v is the length of each shearhead arm actually provided. However, M_v shall not be taken larger than the smaller of:

(a) 30 percent of the total factored moment required for each slab column strip;

(b) The change in column strip moment over the length ℓ_v;

(c) The value of M_p computed by Eq. (11-37).

11.12.4.10 When unbalanced moments are considered, the shearhead must have adequate anchorage to transmit M_p to the column.

11.12.5 Openings in slabs

When openings in slabs are located at a distance less than 10 times the slab thickness from a concentrated load or reaction area, or when openings in flat slabs are located within column strips as defined in Chapter 13, the critical slab sections for shear defined in Section 11.12.1.2 and 11.12.4.7 shall be modified as follows:

11.12.5.1 For slabs without shearheads, that part of the perimeter of the critical section that is enclosed by straight lines projecting from the centroid of the column, concentrated load, or reaction area and tangent to the boundaries of the openings shall be considered ineffective.

11.12.5.2 For slabs with shearheads, the ineffective portion of the perimeter shall be one-half of that defined in Section 11.12.5.1.

11.12.6 Transfer of moment in slab-column connections

11.12.6.1 When gravity load, wind, earthquake, or other lateral forces cause transfer of unbalanced moment M_u between a slab and a column, a fraction $\gamma_f M_u$ of the unbalanced moment shall be transferred by flexure in accordance with Section 13.5.3. The remainder of the unbalanced moment given by $\gamma_v M_u$ shall be considered to be transferred by eccentricity of shear about the centroid of the critical section defined in Section 11.12.1.2 where

$$\gamma_v = \left(1 - \gamma_f\right) \qquad (11\text{-}39)$$

11.12.6.2 The shear stress resulting from moment transfer by eccentricity of shear shall be assumed to vary linearly about the centroid of the critical sections defined in Section 11.12.1.2. The maximum shear stress due to the factored shear force and moment shall not exceed ϕv_n:

(a) For members without shear reinforcement:

$$\phi v_n = \phi V_c / \left(b_0 d\right) \qquad (11\text{-}40)$$

where V_c is as defined in 11.12.2.1 or 11.12.2.2.

(b) For members with shear reinforcement other than shearheads:

$$\phi v_n = \phi\left(V_c + V_s\right) / \left(b_o d\right) \qquad (11\text{-}41)$$

where V_c and V_s are defined in Section 11.12.3.1. The design shall take into account the variation of shear stress around the column. The shear stress due to factored shear force and moment shall not exceed $(1/6)\sqrt{f_c^{'}}$ at the critical section located $d/2$ outside the outermost line *of* stirrup legs that surround the column.

11.12.6.3 When shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) is provided, the sum of the shear stresses due to vertical load acting on the critical section defined by 11.12.4.7 and the shear stresses resulting from moment transferred by eccentricity of shear about the centroid of the critical section defined in Section 11.12.1.2(a) and 11.12.1.3 shall not exceed $\phi(1/3)\sqrt{f_c^{'}}$.

CHAPTER 12
DEVELOPMENT AND SPLICES OF REINFORCEMENT

SECTION 12.0
NOTATION

a	=	depth of equivalent rectangular stress block as defined in 10.2.7.1, mm
A_b	=	area of an individual bar, mm^2
A_s	=	area of nonprestressed tension reinforcement, mm^2
A_{tr}	=	total cross-sectional area of all transverse reinforcement that is within the spacing s and that crosses the potential plane of splitting through the reinforcement being developed, mm^2
A_v	=	area of shear reinforcement within a distance s, mm^2
A_w	=	area of an individual wire to be developed or spliced, mm^2
b_w	=	web width, or diameter of circular section, mm
c	=	spacing or cover dimension, mm. See 12.2.4
d	=	distance from extreme compression fiber to centroid of tension reinforcement, mm
d_b	=	nominal diameter of bar, wire, or prestressing strand, mm
f_c'	=	specified compressive strength of concrete, MPa
$\sqrt{f_c'}$	=	square root of specified compressive strength of concrete, MPa
f_{ct}	=	average splitting tensile strength of lightweight aggregate concrete, MPa
f_{ps}	=	stress in prestressed reinforcement at nominal strength, MPa
f_{se}	=	effective stress in prestressed reinforcement (after allowance for all prestress losses), MPa
f_y	=	specified yield strength of nonprestressed reinforcement, MPa
f_{yt}	=	specified yield strength of transverse reinforcement, MPa
h	=	overall thickness of member, mm
k_{tr}	=	transverse reinforcement index
	=	$\dfrac{A_{tr}\, f_{yt}}{10sn}$ (constant 10 carries the unit MPa)
ℓ_a	=	additional embedment length at support or at point of inflection, mm
ℓ_d	=	development length of deformed bars and deformed wire in tension, mm
ℓ_{dc}	=	development length of deformed bars and deformed wire in compression, mm
ℓ_{dh}	=	development length of standard hook in tension measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus radius of bend and one bar diameter), mm
ℓ_{hb}	=	basic development length of standard hook in tension, mm
M_n	=	nominal moment strength at section, N-mm
	=	$A_s f_y (d - a/2)$

n	=	number of bars or wires being spliced or developed along the plane of splitting
s	=	maximum center-to-center spacing of transverse reinforcement within ℓ_d, mm
s_w	=	spacing of wire to be developed or spliced, mm
V_u	=	factored shear force at section, N
α	=	reinforcement location factor. See 12.2.4
β	=	coating factor. See 12.2.4
β_b	=	ratio of area of reinforcement cut off to total area of tension reinforcement at section
γ	=	reinforcement size factor. See 12.2.4
λ	=	lightweight aggregate concrete factor. See 12.2.4

SECTION 12.1 DEVELOPMENT OF REINFORCEMENT GENERAL

12.1.1 Calculated tension or compression in reinforcement at each section of structural concrete members shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination thereof. Hooks shall not be used to develop bars in compression.

12.1.2 The values of $\sqrt{f_c'}$ used in this chapter shall not exceed 25/3 MPa.

SECTION 12.2 DEVELOPMENT OF DEFORMED BARS AND DEFORMED WIRE IN TENSION

12.2.1 Development length ℓ_d in mm, for deformed bars and deformed wire in tension shall be determined from either Section 12.2.2 or 12.2.3, but ℓ_d shall not be less than 300 mm.

12.2.2 For deformed bars or deformed wire, ℓ_d shall be as follows:

	Dia 20 mm and smaller bars and deformed wires	Dia 22 mm and larger bars
Clear spacing of bars being developed or spliced not less than d_b, clear cover not less than d_b, and stirrups or ties throughout ℓ_d not less than the code minimum or Clear spacing of bars being developed or spliced not less than $2d_b$ and clear cover not less than d_b	$\left(\dfrac{12 f_y \alpha\beta\lambda}{25\sqrt{f_c'}}\right) d_b$	$\left(\dfrac{3 f_y \alpha\beta\lambda}{5\sqrt{f_c'}}\right) d_b$
Other cases	$\left(\dfrac{18 f_y \alpha\beta\lambda}{25\sqrt{f_c'}}\right) d_b$	$\left(\dfrac{9 f_y \alpha\beta\lambda}{10\sqrt{f_c'}}\right) d_b$

12.2.3 For deformed bars or deformed wire, ℓ_d shall be:

$$\ell_d = \left(\frac{9}{10} \frac{f_y}{\sqrt{f_c'}} \frac{\alpha\beta\gamma\lambda}{\left(\frac{c + k_{tr}}{d_b} \right)} \right) d_b \qquad (12\text{-}1)$$

In which the term $\left(c + k_{tr}\right)/ d_b$ shall not be taken greater than 2.5.

12.2.4 The factors for use in the expressions for development of deformed bars and deformed wires in tension in Chapter 12 are as follows:

Horizontal reinforcement so placed that more than 300 mm of fresh concrete is cast in the member below the development length or splice	α =1.3
Other reinforcement	α =1.0
Epoxy-coated bars or wires with cover less than $3d_b$ or clear spacing less than $6d_b$	β =1.5
All other epoxy-coated bars or wires	β =1.2
Uncoated reinforcement	β =1.0
However, the product $\alpha\beta$ need not be taken greater than 1.7.	
Dia 20 mm and smaller bars and deformed wires	γ = 0.8
Dia 22 mm and larger bars	γ = 1.0
When lightweight aggregate concrete is used	λ = 1.3
However, when f_{ct} is specified, λ shall be permitted to be taken as, $\sqrt{f_c'}/1.8 f_{ct}$ but not less than 1.0	
When normalweight concrete is used	λ = 1.0

c = spacing or cover dimension, mm

Use the smaller of either the distance from the center of the bar or wire to the nearest concrete surface or one-half the center-to-center spacing of the bars or wires being developed.

k_{tr} = transverse reinforcement index = $\frac{A_{tr} f_{yt}}{10 sn}$

It shall be permitted to use $k_{tr} = 0$ as a design simplification even if transverse reinforcement is present.

12.2.5 **Excess reinforcement.** Reduction in development length shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for f_y is specifically required or the reinforcement is designed under provisions of Section 21.2.1.4 (A_s required)/(A_s provided).

SECTION 12.3
DEVELOPMENT OF DEFORMED BARS AND DEFORMED WIRE IN COMPRESSION

12.3.1 Development length ℓ_{dc}, in mm, for deformed bars and deformed wire in compression shall be determined from 12.3.2 and applicable modification factors of Section 12.3.3, but ℓ_{dc} shall not be less than 200 mm.

12.3.2 For deformed bars and deformed wire, ℓ_{dc} shall be taken as the larger of $\left(0.24 f_y / \sqrt{f_c^{'}}\right) d_b$ and $\left(0.043 f_y\right) d_b$, where the constant 0.043 carries the unit of mm^2/N.

12.3.3 The length ℓ_{dc} in 12.3.2 shall be permitted to be multiplied by the applicable factors for:

a)	Reinforcement in excess of that required by analysis	$\dfrac{As_{required}}{As_{provided}}$
b)	Reinforcement enclosed within spiral reinforcement not less than 6 mm diameter and not more than 100 mm pitch or within Dia 12 mm ties in conformance with 7.10.5 and spaced at not more than 100 mm on center	0.75

SECTION 12.4
DEVELOPMENT OF BUNDLED BARS

12.4.1 Development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased 20 percent for three-bar bundle, and 33 percent for four-bar bundle.

12.4.2 For determining the appropriate factors in Section 12.2, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

SECTION 12.5
DEVELOPMENT OF STANDARD HOOKS IN TENSION

12.5.1 Development length ℓ_{dh}, in millimeters, for deformed bars in tension terminating in a standard hook (see 7.1) shall be determined from Section 12.5.2 and the applicable modification factors of 12.5.3, but ℓ_{dh} shall not be less than $8d_b$ nor less than 150 mm.

12.5.2 For deformed bars, ℓ_{dh} shall be $\left(0.24 \beta \lambda f_y / \sqrt{f_c^{'}}\right) d_b$ with β taken as 1.2 for epoxy-coated reinforcement, and λ taken as 1.3 for lightweight aggregate concrete. For other cases, β and λ shall be taken as 1.0.

12.5.3 The length ℓ_{dh} in 12.5.2 shall be permitted to be multiplied by the following applicable factors:

a)	For Dia 36 mm bar and smaller hooks with side cover (normal to plane of hook) not less than 60 mm, and for 90 deg hook with cover on bar extension beyond hook not less than 50 mm	0.7
b)	For 90 deg hooks of Dia 36 mm and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length ℓ_{dh} of the hook; or enclosed within ties or stirrups parallel to the bar being developed, spaced not greater than $3d_b$ along the length of the tail extension of the hook plus bend	0.8
c)	For 180 deg hooks of Dia 36 mm and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length ℓ_{dh}, of the hook	0.8
d)	Where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis	$\dfrac{As_{required}}{As_{provided}}$

In 12.5.3(b) and 12.5.3(c), d_b is the diameter of the hooked bar, and the first tie or stirrup shall enclose the bent portion of the hook, within $2d_b$ of the outside of the bend.

12.5.4 For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 60 mm, the hooked bar shall be enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length ℓ_{dh} of the hook. The first tie or stirrup shall enclose the bent portion of the hook, within $2d_b$ of the outside of the bend, where d_b is the diameter of the hooked bar. For this case, the factors of Section 12.5.3 (b) and (c) shall not apply.

12.5.5 Hooks shall not be considered effective in developing bars in compression.

SECTION 12.6
MECHANICAL ANCHORAGE

12.6.1 Any mechanical device capable of developing the strength of reinforcement without damage to concrete is allowed as anchorage.

12.6.2 Test results showing adequacy of such mechanical devices shall be presented to the building official.

12.6.3 Development of reinforcement shall be permitted to consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between the point of maximum bar stress and the mechanical anchorage.

SECTION 12.7
DEVELOPMENT OF WELDED DEFORMED WIRE FABRIC IN TENSION

12.7.1 Development length ℓ_d in mm, of welded deformed wire fabric measured from the point of critical section to the end of wire shall be computed as the product of the development length ℓ_d, from 12.2.2 or 12.2.3 times a wire fabric factor from Section 12.7.2 or 12.7.3. It shall be permitted to reduce the development length in accordance with Section 12.2.5 when applicable, but ℓ_d shall not be less than 200 mm except in computation of lap splices by 12.18. When using the wire fabric factor from Section 12.7.2, it shall be permitted to use an epoxy-coating factor β of 1.0 for epoxy-coated welded wire fabric in Section 12.2.2 and 12.2.3.

12.7.2 For welded deformed wire fabric with at least one cross wire within the development length and not less than 50 mm from the point of the critical section, the wire fabric factor shall be the greater of:

$$\left(\frac{f_y - 240}{f_y}\right)$$

or

$$\left(\frac{5d_b}{s_w}\right)$$

but need not be greater than 1.

12.7.3 For welded deformed wire fabric with no cross wires within the development length or with a single cross wire less than 50 mm from the point of the critical section, the wire fabric factor shall be taken as 1, and the development length shall be determined as for deformed wire.

12.7.4 When any plain wires are present in the deformed wire fabric in the direction of the development length, the fabric shall be developed in accordance with 12.8.

SECTION 12.8
DEVELOPMENT OF WELDED PLAIN WIRE FABRIC IN TENSION

Yield strength of welded plain wire fabric shall be considered developed by embedment of two cross wires with the closer cross wire not less than 50 mm from the point of the critical section. However, the development length ℓ_d in mm, measured from the point of the critical section to the outermost cross wire shall not be less than

$$3.3\frac{A_w}{s_w}\left(\frac{f_y}{\sqrt{f_c'}}\right)\lambda$$

except that when reinforcement provided is in excess of that required, this length may be reduced in accordance with Section 12.2.5. ℓ_d shall not be less than 150 mm except in computation of lap splices by 12.19.

SECTION 12.9
DEVELOPMENT OF PRESTRESSING STRAND

12.9.1 Except as provided in 12.9.1.1, seven-wire strand shall be bonded beyond the critical section for a development length ℓ_d, in mm, not less than

$$\ell_d = \left(\frac{f_{se}}{3}\right)\frac{d_b}{7} + \left(f_{ps} - f_{se}\right)\frac{d_b}{7} \qquad (12\text{-}2)$$

where d_b is strand diameter in mm, and f_{ps} and f_{se} are expressed in MPa. The expressions in parenthesis are used as a constant without units.

12.9.1.1 Embedment less than the development length shall be permitted at a section of a member provided the design strand stress at that section does not exceed values obtained from the bilinear relationship defined by Eq. (12-2).

12.9.2 Limiting the investigation to cross sections nearest each end of the member that are required to develop full design strength under specified factored loads shall be permitted except where bonding of one or more strands does not extend to the end of the member, or where concentrated loads are applied within the strand development length.

12.9.3 Where bonding of a strand does not extend to end of member, and design includes tension at service load in pre-compressed tensile zone as permitted by 18.4.2, development length specified in 12.9.1 shall be doubled.

SECTION 12.10
DEVELOPMENT OF FLEXURAL REINFORCEMENT
GENERAL

12.10.1 Development of tension reinforcement by bending across the web to be anchored or made continuous with reinforcement on the opposite face of member shall be permitted.

12.10.2 Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent. Provisions of Section 12.11.3 must be satisfied.

12.10.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of member or $12d_b$, whichever is greater, except at supports of simple spans and at free end of cantilevers.

12.10.4 Continuing reinforcement shall have an embedment length not less than the development length ℓ_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

12.10.5 Flexural reinforcement shall not be terminated in a tension zone unless Section 12.10.5.1, 12.10.5.2, or 12.10.5.3 is satisfied.

12.10.5.1 Factored shear at the cutoff point does not exceed two-thirds of the design shear strength, ϕV_n.

12.10.5.2 Stirrup area in excess of that required for shear and torsion is provided along each terminated bar or wire over a distance from the termination point equal to three-fourths the effective depth of member. Excess stirrup area A_v shall be not less than $0.4 b_w s / f_y$. Spacing s shall not exceed $d / 8\beta_b$.

12.10.5.3 For Dia 36 mm bars and smaller, continuing reinforcement provides double the area required for flexure at the cutoff point and factored shear does not exceed three-fourths the design shear strength, ϕV_n.

12.10.6 Adequate anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets; deep flexural members; or members in which tension reinforcement is not parallel to compression face. See Section 12.11.4 and 12.12.4 for deep flexural members.

SECTION 12.11
DEVELOPMENT OF POSITIVE MOMENT REINFORCEMENT

12.11.1 At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 150 mm.

12.11.2 When a flexural member is part of a primary lateral load resisting system, positive moment reinforcement required to be extended into the support by Section 12.11.1 shall be anchored to develop the specified yield strength f_y in tension at the face of support.

12.11.3 At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that ℓ_d computed for f_y by 12.2 satisfies Eq. 12-3; except, Eq. 12-3 need not be satisfied for reinforcement terminating beyond centerline of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook.

$$\ell_d \le \frac{M_n}{V_u} + \ell_a \qquad (12\text{-}3)$$

where:

M_n is nominal moment strength assuming all reinforcement at the section to be stressed to the specified yield strength f_y;

V_u is factored shear force at the section;

ℓ_a at a support shall be the embedment length beyond center of support or ℓ_a at a point of inflection shall be limited to the effective depth of member or $12 d_b$, whichever is greater.

An increase of 30 percent in the value of M_n / V_u shall be permitted when the ends of reinforcement are confined by a compressive reaction.

12.11.4 At simple supports of deep beams, positive moment tension reinforcement shall be anchored to develop its specified yield strength, f_y, in tension at the face of the support except that if design is carried out using Appendix A, the positive moment tension reinforcement shall be anchored in accordance with A.4.3. At interior supports of deep beams, positive moment tension reinforcement shall be continuous or be spliced with that of the adjacent spans.

SECTION 12.12
DEVELOPMENT OF NEGATIVE MOMENT REINFORCEMENT

12.12.1 Negative moment reinforcement in a continuous, restrained, or cantilever member, or in any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks, or mechanical anchorage.

12.12.2 Negative moment reinforcement shall have an embedment length into the span as required by Section 12.1 and 12.10.3.

12.12.3 At least one-third the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than effective depth of member, $12d_b$, or one-sixteenth the clear span, whichever is greater.

12.12.4 At interior supports of deep flexural members, negative moment tension reinforcement shall be continuous with that of the adjacent spans.

SECTION 12.13
DEVELOPMENT OF WEB REINFORCEMENT

12.13.1 Web reinforcement shall be as close to the compression and tension surfaces of the member as cover requirements and proximity of other reinforcement permits.

12.13.2 Ends of single leg, simple U-, or multiple U-stirrups shall be anchored as required by Section 12.13.2.1 through 12.13.2.5.

12.13.2.1 For Dia 16 mm bar and WD 12.0 wire, and smaller, and for Dia 20, Dia 22, and Dia 25 mm bars with f_y of 300 MPa or less, a standard hook around longitudinal reinforcement.

12.13.2.2 For Dia 20, Dia 22, and Dia 25 mm stirrups with f_y greater than 300 MPa, a standard stirrup hook around a longitudinal bar plus an embedment between midheight of the member and the outside end of the hook equal to or greater than $0.17 d_b f_y / \sqrt{f_c'}$

12.13.2.3 For each leg of welded plain wire fabric forming simple U-stirrups, either:

(a) Two longitudinal wires spaced at a 50 mm spacing along the member at the top of the U; or

(b) One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 50 mm from the first wire. The second wire shall be permitted to be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than $8d_b$.

12.13.2.4 For each end of a single leg stirrup of welded plain or deformed wire fabric, two longitudinal wires at a minimum spacing of 50 mm and with the inner wire at least the greater of $d/4$ or 50 mm from $d/2$. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

12.13.2.5 In joist construction as defined in 8.11, for Dia 12 mm bar and WD 12.0 wire and smaller, a standard hook.

12.13.3 Between anchored ends, each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar.

12.13.4 Longitudinal bars bent to act as shear reinforcement, if extended into a region of tension, shall be continuous with longitudinal reinforcement and, if extended into a region of compression, shall be anchored beyond mid-depth $d/2$ as specified for development length in 12.2 for that part of f_y required to satisfy Eq. (11-17).

12.13.5 Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when length of laps are $1.3\ell_d$ in members at least 450 mm deep, such splices with $A_b f_y$ not more than 40 kN per leg shall be considered adequate if stirrup legs extend the full available depth of member.

SECTION 12.14
SPLICES OF REINFORCEMENT - GENERAL

12.14.1 Splices of reinforcement shall be made only as required or permitted on design drawings, or in specifications, or as authorized by the engineer.

12.14.2 Lap splices

12.14.2.1 Lap splices shall not be used for bars larger than Dia 36 except as provided in 12.16.2 and 15.8.2.3.

12.14.2.2 Lap splices of bars in a bundle shall be based on the lap splice length required for individual bars within the bundle, increased in accordance with 12.4. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced.

12.14.2.3 Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than one-fifth the required lap splice length, nor 150 mm.

12.14.3 Mechanical and welded splices

12.14.3.1 Mechanical and welded splices shall be permitted. For welded splices, see Section 3.5.2.

12.14.3.2 A full mechanical splice shall develop in tension or compression, as required, at least 125 percent of specified yield strength f_y of the bar.

12.14.3.3 Except as provided in SBC 304, all welding shall conform to “Structural Welding Code – Reinforcing Steel” (ANSI/AWS D1.4).

12.14.3.4 A full welded splice shall develop at least 125 percent of the specified yield strength f_y of the bar.

12.14.3.5 Mechanical or welded splices not meeting requirements of Section 12.14.3.2 or 12.14.3.4 shall be permitted only for Dia 16 mm bars and smaller and in accordance with Section 12.15.4.

SECTION 12.15
SPLICES OF DEFORMED BARS AND DEFORMED WIRE IN TENSION

12.15.1 Minimum length of lap for tension lap splices shall be as required for Class A or B splice, but not less than 300 mm, where:

Class A splice	$1.0\ell_d$
Class B splice	$1.3\ell_d$

where ℓ_d is the tensile development length for the specified yield strength f_y in accordance with 12.2 without the modification factor of 12.2.5.

12.15.2 Lap splices of deformed bars and deformed wire in tension shall be Class B splices except that Class A splices are allowed when:

(a) the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice; and

(b) one-half or less of the total reinforcement is spliced within the required lap length.

12.15.3 Mechanical or welded splices used where area of reinforcement provided is less than twice that required by analysis shall meet requirements of Section 12.14.3.2 or 12.14.3.4.

12.15.4 Mechanical or welded splices not meeting the requirements of Section 12.14.3.2 or 12.14.3.4 shall be permitted for Dia 16 mm bars and smaller if the requirements of Section 12.15.4.1 through 12.15.4.3 are met:

12.15.4.1 Splices shall be staggered at least 600 mm.

12.15.4.2 In computing the tensile forces that can be developed at each section, the spliced reinforcement stress shall be taken as the specified splice strength, but not greater than f_y. The stress in the unspliced reinforcement shall be taken as f_y times the ratio of the shortest length embedded beyond the section to ℓ_d, but not greater than f_y.

12.15.4.3 The total tensile force that can be developed at each section must be at least twice that required by analysis, and at least 140 MPa times the total area of reinforcement provided.

12.15.5 Splices in tension tie members shall be made with a full mechanical or full welded splice in accordance with Section 12.14.3.2 or 12.14.3.4 and splices in adjacent bars shall be staggered at least 750 mm.

SECTION 12.16
SPLICES OF DEFORMED BARS IN COMPRESSION

12.16.1 Compression lap splice length shall be $0.07 f_y d_b$, for f_y of 420 MPa or less, or $\left(0.13 f_y - 24\right) d_b$ for f_y greater than 420 MPa, but not less than 300 mm.

12.16.2 When bars of different size are lap spliced in compression, splice length shall be the larger of either development length of larger bar, or splice length of smaller bar. Lap splices of Dia 40 mm and larger bars to Dia 36 mm and smaller bars shall be permitted.

12.16.3 Mechanical or welded splices used in compression shall meet requirements of Section 12.14.3.2 or 12.14.3.4.

12.16.4 End-bearing splices

12.16.4.1 In bars required for compression only, transmission of compressive stress by bearing of square cut ends held in concentric contact by a suitable device shall be permitted.

12.16.4.2 Bar ends shall terminate in flat surfaces within 1.5 deg of a right angle to the axis of the bars and shall be fitted within 3 deg of full bearing after assembly.

12.16.4.3 End-bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

SECTION 12.17
SPECIAL SPLICE REQUIREMENTS FOR COLUMNS

12.17.1 Lap splices, mechanical splices, butt-welded splices, and end-bearing splices shall be used with the limitations of Section 12.17.2 through 12.17.4. A splice shall satisfy requirements for all load combinations for the column.

12.17.2 Lap splices in columns

12.17.2.1 Where the bar stress due to factored loads is compressive, lap splices shall conform to Section 12.16.1, 12.16.2, and, where applicable, to Section 12.17.2.4 or 12.17.2.5.

12.17.2.2 Where the bar stress due to factored loads is tensile and does not exceed $0.5 f_y$ in tension, lap splices shall be Class B tension lap splices if more than one-half of the bars are spliced at any section, or Class A tension lap splices if half or fewer of the bars are spliced at any section and alternate lap splices are staggered by ℓ_d.

12.17.2.3 Where the bar stress due to factored loads is greater than $0.5 f_y$ in tension, lap splices shall be Class B tension lap splices.

12.17.2.4 In tied reinforced compression members, where ties throughout the lap splice length have an effective area not less than $0.0015 hs$, lap splice length shall be permitted to be multiplied by 0.83, but lap length shall not be less than 300 mm. Tie legs perpendicular to dimension h shall be used in determining effective area.

12.17.2.5 In spirally reinforced compression members, lap splice length of bars within a spiral shall be permitted to be multiplied by 0.75, but lap length shall not be less than 300 mm.

12.17.3 **Mechanical or welded splices in columns**

Mechanical or welded splices in columns shall meet the requirements of Section 12.14.3.2 or 12.14.3.4.

12.17.4 **End-bearing splices in columns**

End-bearing splices complying with Section 12.16.4 shall be permitted to be used for column bars stressed in compression provided the splices are staggered or additional bars are provided at splice locations. The continuing bars in each face of the column shall have a tensile strength, based on the specified yield strength f_y not less than $0.25 f_y$ times the area of the vertical reinforcement in that face.

SECTION 12.18
SPLICES OF WELDED DEFORMED WIRE FABRIC IN TENSION

12.18.1 Minimum length of lap for lap splices of welded deformed wire fabric measured between the ends of each fabric sheet shall be not less than $1.3\ell_d$ nor 200 mm, and the overlap measured between outermost cross wires of each fabric sheet shall be not less than 50 mm. ℓ_d shall be the development length for the specified yield strength f_y in accordance with 12.7.

12.18.2 Lap splices of welded deformed wire fabric, with no cross wires within the lap splice length, shall be determined as for deformed wire.

12.18.3 When any plain wires are present in the deformed wire fabric in the direction of the lap splice or when deformed wire fabric is lap spliced to plain wire fabric, the fabric shall be lap spliced in accordance with 12.19.

SECTION 12.19
SPLICES OF WELDED PLAIN WIRE FABRIC IN TENSION

Minimum length of lap for lap splices of welded plain wire fabric shall be in accordance with Section 12.19.1 and 12.19.2.

12.19.1 When area of reinforcement provided is less than twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each fabric sheet shall be not less than one spacing of cross wires plus 50 mm, nor less than $1.5\ell_d$, nor 150 mm. The development length ℓ_d for the specified yield strength f_y shall be in accordance with Section 12.8.

12.19.2 When area of reinforcement provided is at least twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each fabric sheet shall not be less than $1.5\ell_d$, nor 50 mm. ℓ_d shall be the development length for the specified yield strength f_y in accordance with 12.8.

CHAPTER 13
TWO-WAY SLAB SYSTEM

SECTION 13.0
NOTATION

b_1 = width of the critical section defined in Section 11.12.1.2 measured in the direction of the span for which moments are determined, mm

b_2 = width of the critical section defined in Section 11.12.1.2 measured in the direction perpendicular to b_1, mm

c_1 = size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm

c_2 = size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which moments are being determined, mm

C = cross-sectional constant to define torsional properties

= $\sum (1 - 0.63 \frac{x}{y}) \frac{x^3 y}{3}$

The constant C for T- or L-sections shall be permitted to be evaluated by dividing the section into separate rectangular parts and summing the values of C for each part

E_{cb} = modulus of elasticity of beam concrete, MPa

E_{cs} = modulus of elasticity of slab concrete, MPa

h = overall thickness of member, mm

I_b = moment of inertia about centroidal axis of gross section of beam as defined in 13.2.4, mm^4

I_s = moment of inertia about centroidal axis of gross section of slab, mm^4

= $h^3/12$ times width of slab defined in notations α and β_t

k_t = torsional stiffness of torsional member; moment per unit rotation. See R13.7.5.

ℓ_n = length of clear span in direction that moments are being determined, measured face-to-face of supports, mm

ℓ_1 = length of span in direction that moments are being determined, measured center-to-center of supports, mm

ℓ_2 = length of span transverse to ℓ_1 measured center-to-center of supports, mm. See also 13.6.2.3 and 13.6.2.4.

M_o = total factored static moment, N-mm

M_u = factored moment at section, N-mm

V_c = nominal shear strength provided by concrete, N. See 11.12.2.1

V_u = factored shear force at section, N

w_d = factored dead load per unit area

w_ℓ = factored live load per unit area

w_u = factored load per unit area

x = shorter overall dimension of rectangular part rectangular part of cross section, mm

y = overall dimension of rectangular part of cross section, mm

α = ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of the beam

= $\dfrac{E_{cb} I_b}{E_{cs} I_s}$

α_1 = α in direction of ℓ_1

α_2 = α in direction of ℓ_2

β_t = ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of beam, center-to-center of supports

= $\dfrac{E_{cb} C}{2E_{cs} I_s}$

γ_f = fraction of unbalanced moment transferred by flexure at slab-column connections. See 13.5.3.2

γ_v = fraction of unbalanced moment transferred by eccentricity of shear at slab-column connections

= $1-\gamma_f$

ρ = ratio of nonprestressed tension reinforcement

ρ_b = reinforcement ratio producing balanced strain conditions

ϕ = strength reduction factor

SECTION 13.1
SCOPE

13.1.1 Provisions of Chapter 13 shall apply for design of slab systems reinforced for flexure in more than one direction, with or without beams between supports.

13.1.2 For a slab system supported by columns or walls, the dimensions c_1 and c_2 and the clear span ℓ_n shall be based on an effective support area defined by the intersection of the bottom surface of the slab, or of the drop panel if there is one, with the largest right circular cone, right pyramid, or tapered wedge whose surfaces are located within the column and the capital or bracket and are oriented no greater than 45 deg to the axis of the column.

13.1.3 Solid slabs and slabs with recesses or pockets made by permanent or removable fillers between ribs or joists in two directions are included within the scope of Chapter 13.

13.1.4 Minimum thickness of slabs designed in accordance with Chapter 13 shall be as required by Section 9.5.3.

SECTION 13.2
DEFINITIONS

13.2.1 Column strip is a design strip with a width on each side of a column centerline equal to $0.25\ell_2$ or $0.25\ell_1$, whichever is less. Column strip includes beams, if any.

13.2.2 Middle strip is a design strip bounded by two column strips.

13.2.3 A panel is bounded by column, beam, or wall centerlines on all sides.

13.2.4 For monolithic or fully composite construction, a beam includes that portion of slab on each side of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness.

SECTION 13.3
SLAB REINFORCEMENT

13.3.1 Area of reinforcement in each direction for two-way slab systems shall be determined from moments at critical sections, but shall not be less than required by 7.12.

13.3.2 Spacing of reinforcement at critical sections shall not exceed two times the slab thickness, except for portions of slab area of cellular or ribbed construction. In the slab over cellular spaces, reinforcement shall be provided as required by 7.12.

13.3.3 Positive moment reinforcement perpendicular to a discontinuous edge shall extend to the edge of slab and have embedment, straight or hooked, at least 150 mm in spandrel beams, columns, or walls.

13.3.4 Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored in spandrel beams, columns, or walls, and shall be developed at face of support according to provisions of Chapter 12.

13.3.5 Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, anchorage of reinforcement shall be permitted within the slab.

13.3.6 In slabs with beams between supports with a value of α greater than 1.0, special top and bottom slab reinforcement shall be provided at exterior corners in accordance with Sections 13.3.6.1 through 13.3.6.4.

13.3.6.1 The special reinforcement in both top and bottom of slab shall be sufficient to resist a moment per meter of width equal to the maximum positive moment in the slab.

13.3.6.2 The moment shall be assumed to be about an axis perpendicular to the diagonal from the corner in the top of the slab and about an axis parallel to the diagonal from the comer in the bottom of the slab.

13.3.6.3 The special reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth the longer span.

13.3.6.4 The special reinforcement shall be placed in a band parallel to the diagonal in the top of the slab and a band perpendicular to the diagonal in the bottom of the slab. Alternatively, the special reinforcement shall be placed in two layers parallel to the sides of the slab in both the top and bottom of the slab.

13.3.7 Where a drop panel is used to reduce amount of negative moment reinforcement over the column of a flat slab, size of drop panel shall be in accordance with the Section 13.3.7.1, 13.3.7.2, and 13.3.7.3.

13.3.7.1 Drop panel shall extend in each direction from centerline of support a distance not less than one-sixth the span length measured from center-to-center of supports in that direction.

13.3.7.2 Projection of drop panel below the slab shall be at least one-quarter the slab thickness beyond the drop.

13.3.7.3 In computing required slab reinforcement, the thickness of the drop panel below the slab shall not be assumed greater than one-quarter the distance from edge of drop panel to edge of column or column capital.

13.3.8 Details of reinforcement in slabs without beams

13.3.8.1 In addition to the other requirements of 13.3, reinforcement in slabs without beams shall have minimum extensions as prescribed in Fig. 13.3.8.

13.3.8.2 Where adjacent spans are unequal, extensions of negative moment reinforcement beyond the face of support as prescribed in Fig. 13.3.8 shall be based on requirements of the longer span.

13.3.8.3 Bent bars shall be permitted only when depth-span ratio permits use of bends of 45 deg or less.

13.3.8.4 In frames where two-way slabs act as primary members resisting lateral loads, lengths of reinforcement shall be determined by analysis but shall not be less than those prescribed in Fig. 13.3.8.

13.3.8.5 All bottom bars or wires within the column strip, in each direction, shall be continuous or spliced with Class A tension splices or with mechanical or welded splices satisfying 12.14.3. Splices shall be located as shown in Fig: 13.3.8. At least two of the column strip bottom bars or wires in each direction shall pass within the column core and shall be anchored at exterior support.

13.3.8.6 In slabs with shearheads and in lift-slab construction where it is not practical to pass the bottom bars required by Section 13.3.8.5 through the column, at least two bonded bottom bars or wires in each direction shall pass through the shearhead or lifting collar as close to the column as practicable and be continuous or spliced with a Class A splice. At exterior columns, the reinforcement shall be anchored at the shearhead or lifting collar.

SECTION 13.4
OPENINGS IN SLAB SYSTEMS

13.4.1 Openings of any size shall be permitted in slab systems if shown by analysis that the design strength is at least equal to the required strength set forth in 9.2 and 9.3, and that all serviceability conditions, including the limits on deflections, are met.

13.4.2 As an alternate to special analysis as required by 13.4.1, openings shall be permitted in slab systems without beams only in accordance with Sections 13.4.2.1 through 13.4.2.4.

13.4.2.1 Openings of any size shall be permitted in the area common to intersecting middle strips, provided total amount of reinforcement required for the panel without the opening is maintained.

13.4.2.2 In the area common to intersecting column strips, not more than one-eighth the width of column strip in either span shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening.

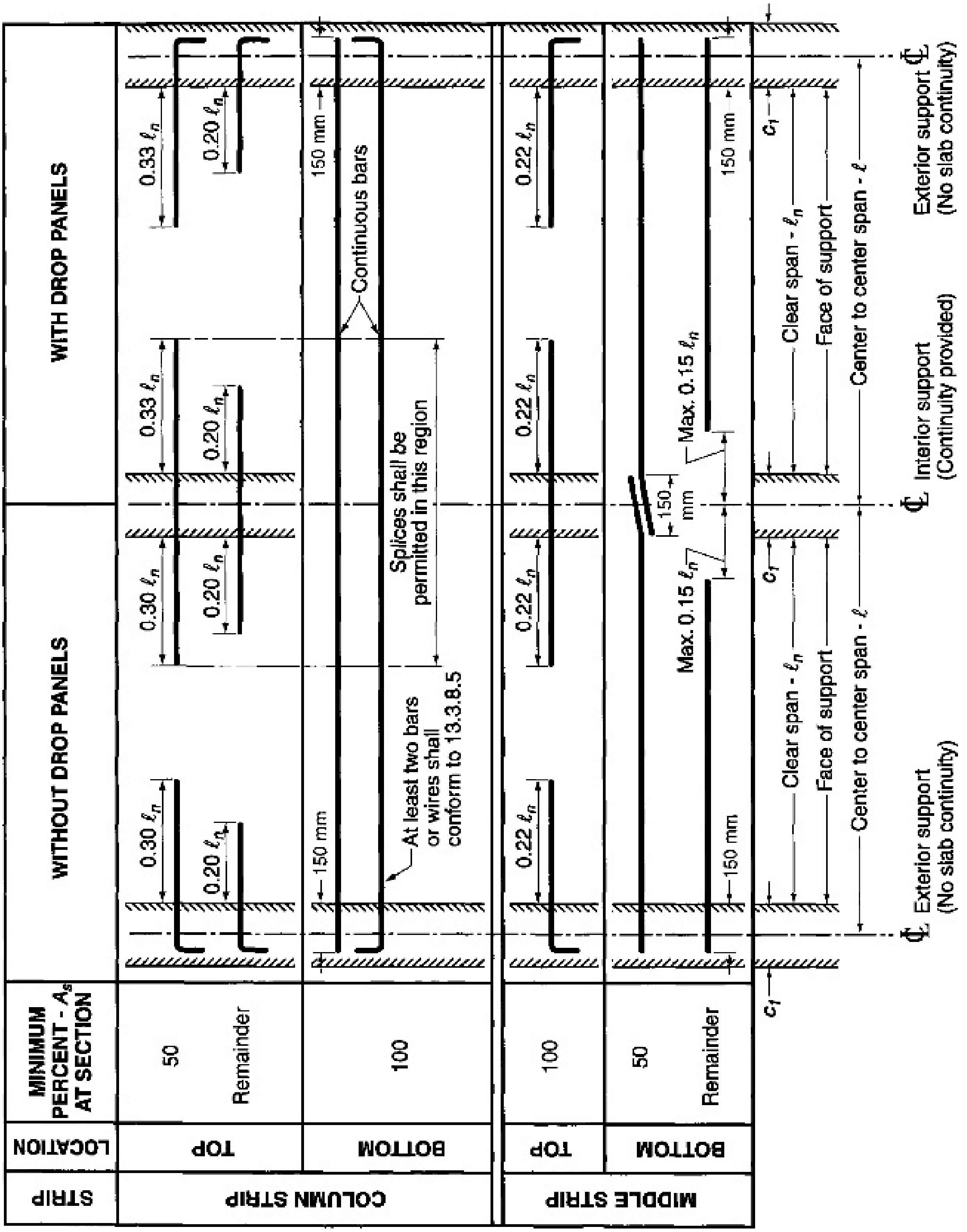


Fig. 13.3.8 – Minimum extensions for reinforcement in slabs without beams. (See 12.11.1 for reinforcement extension into supports)

13.4.2.3 In the area common to one column strip and one middle strip, not more than one-quarter of the reinforcement in either strip shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening.

13.4.2.4 Shear requirements of Section 11.12.5 shall be satisfied.

SECTION 13.5
DESIGN PROCEDURES

13.5.1 A slab system shall be designed by any procedure satisfying conditions of equilibrium and geometric compatibility, if shown that the design strength at every section is at least equal to the required strength set forth in Section 9.2 and 9.3, and that all serviceability conditions, including limits on deflections, are met.

13.5.1.1 Design of a slab system for gravity loads, including the slab and beams (if any) between supports and supporting columns or walls forming orthogonal frames, by either the Direct Design Method of Section 13.6 or the Equivalent Frame Method of 13.7 shall be permitted.

13.5.1.2 For lateral loads, analysis of frames shall take into account effects of cracking and reinforcement on stiffness of frame members.

13.5.1.3 Combining the results of the gravity load analysis with the results of the lateral load analysis shall be permitted.

13.5.1.4 Slabs supported on stiff edges that do not satisfy the limitations of Section 13.6.1 may be designed by the methods in Appendix C. The rigidity requirement of the supporting beam or girder may be considered satisfactory if the beam or girder is supported by columns or walls and has a total depth not less than three times the slab thickness.

13.5.2 The slab and beams (if any) between supports shall be proportioned for factored moments prevailing at every section.

13.5.3 When gravity load, wind, earthquake, or other lateral forces cause transfer of moment between slab and column, a fraction of the unbalanced moment shall be transferred by flexure in accordance with Section 13.5.3.2 and 13.5.3.3.

13.5.3.1 The fraction of unbalanced moment not transferred by flexure shall be transferred by eccentricity of shear in accordance with 11.12.6.

13.5.3.2 A fraction of the unbalanced moment given by $\gamma_f M_u$ shall be considered to be transferred by flexure within an effective slab width between lines that are one and one-half slab or drop panel thicknesses $(1.5h)$ outside opposite faces of the column or capital, where M_u is the moment to be transferred and

$$\gamma_f = \frac{1}{1+(2/3)\sqrt{b_1/b_2}} \tag{13-1}$$

13.5.3.3 For unbalanced moments about an axis parallel to the edge at exterior supports, the value of γ_f by Eq. (13-1) shall be permitted to be increased up to 1.0 provided that V_u at an edge support does not exceed $0.75\phi V_c$ or at a corner support does not exceed $0.5\phi V_c$. For unbalanced moments at interior supports, and for unbalanced moments about an axis transverse to the edge at exterior supports, the value of γ_f in Eq. (13-1) shall be permitted to be increased by up to 25 percent provided that V_u at the support does not exceed $0.4\phi V_c$. The reinforcement ratio ρ, within the effective slab width defined in Section 13.5.3.2, shall not exceed $0.375\rho_b$. No adjustments to γ_f shall be permitted for prestressed slab systems.

13.5.3.4 Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist moment on the effective slab width defined in Section 13.5.3.2.

13.5.4 Design for transfer of load from slabs to supporting columns or walls through shear and torsion shall be in accordance with Chapter 11.

SECTION 13.6
DIRECT DESIGN METHOD

13.6.1 **Limitations**

Design of slab systems within the limitations of 13.6.1.1 through 13.6.1.8 by the direct design method shall be permitted.

13.6.1.1 There shall be a minimum of three continuous spans in each direction.

13.6.1.2 Panels shall be rectangular, with a ratio of longer to shorter span center-to-center of supports within a panel not greater than 2.

13.6.1.3 Successive span lengths center-to-center of supports in each direction shall not differ by more than one-third the longer span.

13.6.1.4 Offset of columns by a maximum of 10 percent of the span (in direction of offset) from either axis between centerlines of successive columns shall be permitted.

13.6.1.5 All loads shall be due to gravity only and uniformly distributed over an entire panel. Live load shall not exceed two times dead load.

13.6.1.6 For a panel with beams between supports on all sides, the relative stiffness of beams in two perpendicular directions

$$\frac{\alpha_1 l_2^2}{\alpha_2 l_1^2} \tag{13-2}$$

shall not be less than 0.2 nor greater than 5.0.

13.6.1.7 Moment redistribution as permitted by 8.4 shall not be applied for slab systems designed by the Direct Design Method. See 13.6.7.

13.6.1.8 Variations from the limitations of 13.6.1 shall be permitted if demonstrated by analysis that requirements of 13.5.1 are satisfied.

13.6.2 **Total factored static moment for a span**

13.6.2.1 Total factored static moment for a span shall be determined in a strip bounded laterally by centerline of panel on each side of centerline of supports.

13.6.2.2 Absolute sum of positive and average negative factored moments in each direction shall not be less than

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} \tag{13-3}$$

13.6.2.3 Where the transverse span of panels on either side of the centerline of supports varies, ℓ_2 in Eq. (13-3) shall be taken as the average of adjacent transverse spans.

13.6.2.4 When the span adjacent and parallel to an edge is being considered, the distance from edge to panel centerline shall be substituted for ℓ_2 in Eq. (13-3).

13.6.2.5 Clear span ℓ_n shall extend from face to face of columns, capitals, brackets, or walls. Value of ℓ_1 used in Eq. (13-3) shall not be less than $0.65\ell_1$. Circular or regular polygon shaped supports shall be treated as square supports with the same area.

13.6.3 Negative and positive factored moments

13.6.3.1 Negative factored moments shall be located at face of rectangular supports. Circular or regular polygon shaped supports shall be treated as square supports with the same area.

13.6.3.2 In an interior span, total static moment M_o shall be distributed as follows:

Negative factored moment ………………………………..…….. 0.65
Positive factored moment ……………………………………….. 0.35

13.6.3.3 In an end span, total factored static moment M_o shall be distributed as follows:

	(1)	(2)	(3)	(4)	(5)
	Exterior edge unrestrained	**Slab with beams between all supports**	**Slab without beams between interior supports**		**Exterior edge fully restrained**
			Without edge beam	**With edge beam**	
Interior negative factored moment	0.75	0.70	0.70	0.70	0.65
Positive factored moment	0.63	0.57	0.52	0.50	0.35
Exterior negative factored moment	0	0.16	0.26	0.30	0.65

13.6.3.4 Negative moment sections shall be designed to resist the larger of the two interior negative factored moments determined for spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with stiffnesses of adjoining elements.

13.6.3.5 Edge beams or edges of slab shall be proportioned to resist in torsion their share of exterior negative factored moments.

13.6.3.6 The gravity load moment to be transferred between slab and edge column in accordance with 13.5.3.1 shall be 0.3 M_o.

13.6.4 Factored moments in column strips

13.6.4.1 Column strips shall be proportioned to resist the following portions in percent of interior negative factored moments:

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1)=0$	75	75	75
$(\alpha_1\ell_2/\ell_1)\geq 1.0$	90	75	45

Linear interpolations shall be made between values shown.

13.6.4.2 Column strips shall be proportioned to resist the following portions in percent of exterior negative factored moments:

ℓ_2/ℓ_1		0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1)=0$	$\beta_t=0$	100	100	100
	$\beta_t\geq 2.5$	75	75	75
$(\alpha_1\ell_2/\ell_1)\geq 1.0$	$\beta_t=0$	100	100	100
	$\beta_t\geq 2.5$	90	75	45

Linear interpolations shall be made between values shown.

13.6.4.3 Where supports consist of columns or walls extending for a distance equal to or greater than three-quarters the span length ℓ_2 used to compute M_o negative moments shall be considered to be uniformly distributed across ℓ_2.

13.6.4.4 Column strips shall be proportioned to resist the following portions in percent of positive factored moments:

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1)=0$	60	60	60
$(\alpha_1\ell_2/\ell_1)\geq 1.0$	90	75	45

Linear interpolations shall be made between values shown.

13.6.4.5 For slabs with beams between supports, the slab portion of column strips shall be proportioned to resist that portion of column strip moments not resisted by beams.

13.6.5 Factored moments in beams

13.6.5.1 Beams between supports shall be proportioned to resist 85 percent of column strip moments if $(\alpha_1\ell_2/\ell_1)$ is equal to or greater than 1.0.

13.6.5.2 For values of $(\alpha_1\ell_2/\ell_1)$ between 1.0 and zero, proportion of column strip moments resisted by beams shall be obtained by linear interpolation between 85 and zero percent.

13.6.5.3 In addition to moments calculated for uniform loads according to Sections 13.6.2.2, 13.6.5.1, and 13.6.5.2, beams shall be proportioned to resist all moments caused by concentrated or linear loads applied directly to beams, including weight of projecting beam stem above or below the slab.

13.6.6 Factored moments in middle strips

13.6.6.1 That portion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to corresponding half middle strips.

13.6.6.2 Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.

13.6.6.3 A middle strip adjacent to and parallel with a wall-supported edge shall be proportioned to resist twice the moment assigned to the half middle strip corresponding to the first row of interior supports.

13.6.7 **Modification of factored moments**

Modification of negative and positive factored moments by 10 percent shall be permitted provided the total static moment for a panel in the direction considered is not less than that required by Eq. (13-3).

13.6.8 **Factored shear in slab systems with beams**

13.6.8.1 Beams with $(\alpha_1 \ell_2 / \ell_1)$ equal to or greater than 1.0 shall be proportioned to resist shear caused by factored loads on tributary areas which are bounded by 45 deg lines drawn from the corners of the panels and the centerlines of the adjacent panels parallel to the long sides.

13.6.8.2 In proportioning of beams with $(\alpha_1 \ell_2 / \ell_1)$ less than 1.0 to resist shear, linear interpolation, assuming beams carry no load at $\alpha_1 = 0$, shall be permitted.

13.6.8.3 In addition to shears calculated according to Section 13.6.8.1 and 13.6.8.2, beams shall be proportioned to resist shears caused by factored loads applied directly on beams.

13.6.8.4 Computation of slab shear strength on the assumption that load is distributed to supporting beams in accordance with Section 13.6.8.1 or 13.6.8.2 shall be permitted. Resistance to total shear occurring on a panel shall be provided.

13.6.8.5 Shear strength shall satisfy the requirements of Chapter 11.

13.6.9 **Factored moments in columns and walls**

13.6.9.1 Columns and walls built integrally with a slab system shall resist moments caused by factored loads on the slab system.

13.6.9.2 At an interior support, supporting elements above and below the slab shall resist the moment specified by Eq. (13-4) in direct proportion to their stiffnesses unless a general analysis is made.

$$M = 0.07[(w_d + 0.5w_l)l_2 l_n^2 - w'_d l'_2 (l'_n)^2] \qquad \text{(13-4)}$$

where $w_d^{'}, l_2^{'}$, and $l_n^{'}$ refer to shorter span.

SECTION 13.7
EQUIVALENT FRAME METHOD

13.7.1 Design of slab systems by the equivalent frame method shall be based on assumptions given in Sections 13.7.2 through 13.7.6 and all sections of slabs and supporting members shall be proportioned for moments and shears thus obtained.

13.7.1.1 Where metal column capitals are used, it shall be permitted to take account of their contributions to stiffness and resistance to moment and to shear.

13.7.1.2 It shall be permitted to neglect the change in length of columns and slabs due to direct stress, and deflections due to shear.

13.7.2 Equivalent frame

13.7.2.1 The structure shall be considered to be made up of equivalent frames on column lines taken longitudinally and transversely through the building.

13.7.2.2 Each frame shall consist of a row of columns or supports and slab-beam strips, bounded laterally by the centerline of panel on each side of the centerline of columns or supports.

13.7.2.3 Columns or supports shall be assumed to be attached to slab-beam strips by torsional members (see 13.7.5) transverse to the direction of the span for which moments are being determined and extending to bounding lateral panel centerlines on each side of a column.

13.7.2.4 Frames adjacent and parallel to an edge shall be bounded by that edge and the centerline of adjacent panel.

13.7.2.5 Analysis of each equivalent frame in its entirety shall be permitted. Alternatively, for gravity loading, a separate analysis of each floor or roof with far ends of columns considered fixed shall be permitted.

13.7.2.6 Where slab-beams are analyzed separately, determination of moment at a given support assuming that the slab-beam is fixed at any support two panels distant therefrom, shall be permitted, provided the slab continues beyond that point.

13.7.3 Slab-beams

13.7.3.1 Determination of the moment of inertia of slab-beams at any cross section outside of joints or column capitals using the gross area of concrete shall be permitted.

13.7.3.2 Variation in moment of inertia along axis of slab-beams shall be taken into account.

13.7.3.3 Moment of inertia of slab-beams from center of column to face of column, bracket, or capital shall be assumed equal to the moment of inertia of the slab-beam at face of column, bracket, or capital divided by the quantity $(1 - c_2 / \ell_2)^2$ where c_2 and ℓ_2 are measured transverse to the direction of the span for which moments are being determined.

13.7.4 Columns

13.7.4.1 Determination of the moment of inertia of columns at any cross section outside of joints or column capitals using the gross area of concrete shall be permitted.

13.7.4.2 Variation in moment of inertia along axis of columns shall be taken into account.

13.7.4.3 Moment of inertia of columns from top to bottom of the slab-beam at a joint shall be assumed to be infinite.

13.7.5 Torsional members

13.7.5.1 Torsional members in Section 13.7.2.3 shall be assumed to have a constant cross section throughout their length consisting of the largest of (a), (b) and (c).

(a) A portion of slab having a width equal to that of the column, bracket, or capital in the direction of the span for which moments are being determined;

(b) For monolithic or fully composite construction, the portion of slab specified in (a) plus that part of the transverse beam above and below the slab;

(c) The transverse beam as defined in Section 13.2.4.

13.7.5.2 Where beams frame into columns in the direction of the span for which moments are being determined, the torsional stiffness shall be multiplied by the ratio of the moment of inertia of the slab with such a beam to the moment of inertia of the slab without such a beam.

13.7.6 Arrangement of live load

13.7.6.1 When the loading pattern is known, the equivalent frame shall be analyzed for that load.

13.7.6.2 When live load is variable but does not exceed three-quarters of the dead load, or the nature of live load is such that all panels will be loaded simultaneously, it shall be permitted to assume that maximum factored moments occur at all sections with full factored live load on entire slab system.

13.7.6.3 For loading conditions other than those defined in Section 13.7.6.2, it shall be permitted to assume that maximum positive factored moment near midspan of a panel occurs with three-quarters of the full factored live load on the panel and on alternate panels; and it shall be permitted to assume that maximum negative factored moment in the slab at a support occurs with three-quarters of the full live load on adjacent panels only.

13.7.6.4 Factored moments shall be taken not less than those occurring with full factored live load on all panels.

13.7.7 Factored moments

13.7.7.1 At interior supports, the critical section for negative factored moment (in both column and middle strips) shall be taken at face of rectilinear supports, but not farther away than $0.175\ell_1$ from the center of a column.

13.7.7.2 At exterior supports with brackets or capitals, the critical section for negative factored moment in the span perpendicular to an edge shall be taken at a distance from face of supporting element not greater than one-half the projection of bracket or capital beyond face of supporting element.

13.7.7.3 Circular or regular polygon shaped supports shall be treated as square supports with the same area for location of critical section for negative design moment.

13.7.7.4 When slab systems within limitations of 13.6.1 are analyzed by the equivalent frame method, it shall be permitted to reduce the resulting computed moments in such proportion that the absolute sum of the positive and average negative moments used in design need not exceed the value obtained from Eq. (13-3).

13.7.7.5 Distribution of moments at critical sections across the slab-beam strip of each frame to column strips, beams, and middle strips as provided in Section 13.6.4, 13.6.5, and 13.6.6 shall be permitted if the requirement of Section 13.6.1.6 is satisfied.

CHAPTER 14
WALLS

SECTION 14.0
NOTATION

A_g = gross area of section, mm²

A_s = area of longitudinal tension reinforcement in wall segment, mm²

A_{se} = area of effective longitudinal tension reinforcement in wall segment, mm², as calculated by Eq. (14-8)

c = distance from extreme compression fiber to neutral axis, mm

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm

E_c = modulus of elasticity of concrete, MPa

f_c' = specified compressive strength of concrete, MPa

f_y = specified yield strength of nonprestressed reinforcement, MPa

h = overall thickness of member, mm

I_{cr} = moment of inertia of cracked section transformed to concrete, m^4

I_e = effective moment of inertia for computation of deflection, m^4

k = effective length factor

ℓ_c = vertical distance between supports, mm

ℓ_w = horizontal length of wall, mm

M = maximum unfactored moment due to service loads, including $P\Delta$ effects, N-mm

M_a = maximum moment in member at stage deflection is computed, N-mm

M_{cr} = moment causing flexural cracking due to applied lateral and vertical loads, N-mm

M_n = nominal moment strength at section, N-mm

M_{sa} = maximum unfactored applied moment due to service loads, not including $P\Delta$ effects, N-mm

M_u = factored moment at section including $P\Delta$ effects, N-mm

M_{ua} = moment at the midheight section of the wall due to factored lateral and eccentric vertical loads, N-mm

n = modular ratio of elasticity, but not less than 6

$= \dfrac{E_e}{E_c}$

P_{nw} = nominal axial load strength of wall designed by the empirical method, N (see 14.5)

P_s = unfactored axial load at the design (midheight) section including effects of self-weight, N

P_u = factored axial load, N

Δ_s = maximum deflection at or near midheight due to service loads, mm

Δ_u = deflection at midheight of wall due to factored loads, mm

ϕ = strength reduction factor. See 9.3

ρ = ratio of tension reinforcement

$= A_s/(\ell_w d)$

ρ_b = reinforcement ratio producing balanced strain conditions

SECTION 14.1
SCOPE

14.1.1 Provisions of Chapter 14 shall apply for design of walls subjected to axial load, with or without flexure.

14.1.2 Cantilever retaining walls are designed according to flexural design provisions of Chapter 10 with minimum horizontal reinforcement according to Section 14.3.3.

SECTION 14.2
GENERAL

14.2.1 Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.

14.2.2 Walls subject to axial loads shall be designed in accordance with Section 14.2, 14.3, and either 14.4, 14.5, or 14.8.

14.2.3 Design for shear shall be in accordance with 11.10.

14.2.4 Unless otherwise demonstrated by an analysis, the horizontal length of wall considered as effective for each concentrated load shall not exceed center-to-center distance between loads, nor the bearing width plus four times the wall thickness.

14.2.5 Compression members built integrally with walls shall conform to 10.8.2.

14.2.6 Walls shall be anchored to intersecting elements, such as floors and roofs; or to columns, pilasters, buttresses, of intersecting walls; and to footings.

14.2.7 Quantity of reinforcement and limits of thickness required by 14.3 and 14.5 shall be permitted to be waived where structural analysis shows adequate strength and stability.

14.2.8 Transfer of force to footing at base of wall shall be in accordance with 15.8.

SECTION 14.3
MINIMUM REINFORCEMENT

14.3.1 Minimum vertical and horizontal reinforcement shall be in accordance with 14.3.2 and 14.3.3 unless a greater amount is required for shear by Section 11.10.8 and 11.10.9.

14.3.2 Minimum ratio of vertical reinforcement area to gross concrete area shall be:

(a) 0.0012 for deformed bars not larger than Dia 16 mm with a specified yield strength not less than 420 MPa; or

(b) 0.0015 for other deformed bars; or

(c) 0.0012 for welded wire fabric (plain or deformed) not larger than WD 12.

14.3.3 Minimum ratio of horizontal reinforcement area to gross concrete area shall be:

- **(a)** 0.0020 for deformed bars not larger than Dia 16 mm with a specified yield strength not less than 420 MPa; or
- **(b)** 0.0025 for other deformed bars; or
- **(c)** 0.0020 for welded wire fabric (plain or deformed) not larger than WD 12.

14.3.4 Walls more than 250 mm thick, except basement walls, shall have reinforcement for each direction placed in two layers parallel with faces of wall in accordance with the following:

- **(a)** One layer consisting of not less than one-half and not more than two-thirds of total reinforcement required for each direction shall be placed not less than 50 mm nor more than one-third the thickness of wall from the exterior surface;
- **(b)** The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than 20 mm nor more than one-third the thickness of wall from the interior surface.

14.3.5 Vertical and horizontal reinforcement shall not be spaced farther apart than two times the wall thickness, nor farther apart than 300 mm.

14.3.6 Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

14.3.7 In addition to the minimum reinforcement required by Section 14.3.1, not less than two Dia 16 mm bars shall be provided around all window and door openings. Such bars shall be extended to develop the bar beyond the corners of the openings but not less than 600 mm.

SECTION 14.4
WALLS DESIGNED AS COMPRESSION MEMBERS

14.4.1 Except as provided in 14.5, walls subject to axial load or combined flexure and axial load shall be designed as compression members in accordance with provisions of Section 10.2, 10.3, 10.10, 10.11, 10.12, 10.13, 10.14, 10.17, 14.2, and 14.3.

SECTION 14.5
EMPIRICAL DESIGN METHOD

14.5.1 Walls of solid rectangular cross section shall be permitted to be designed by the empirical provisions of Section 14.5 if the resultant of all factored loads is located within the middle third of the overall thickness of the wall and all limits of 14.2, 14.3, and 14.5 are satisfied.

14.5.2 Design axial load strength ϕP_{nw} of a wall satisfying limitations of 14.5.1 shall be computed by Eq. (14-1) unless designed in accordance with 14.4.

$$\phi P_{nw} = 0.55\,\phi\, f_c'\, A_g \left[1 - \left(\frac{k\ell_c}{32h} \right)^2 \right] \qquad (14\text{-}1)$$

where $\phi = 0.70$ and effective length factor k shall be: For walls braced at top and bottom against lateral translation and

(a) Restrained against rotation at one or both ends (top, bottom, or both) ….0.8

(b) Unrestrained against rotation at both ends……………………………..…..1.0

For walls not braced against lateral translation …………………………….…2.0

14.5.3 Minimum thickness of walls designed by empirical design method

14.5.3.1 Thickness of bearing walls shall not be less than 1/25 the supported height or length, whichever is shorter, nor less than 100 mm.

14.5.3.2 Thickness of exterior basement walls and foundation walls shall not be less than 200 mm.

SECTION 14.6
NONBEARING WALLS

14.6.1 Thickness of nonbearing walls shall not be less than 100 mm, nor less than 1/30 the least distance between members that provide lateral support.

SECTION 14.7
WALLS AS GRADE BEAMS

14.7.1 Walls designed as grade beams shall have top and bottom reinforcement as required for moment in accordance with provisions of Sections 10.2 through 10.7. Design for shear shall be in accordance with provisions of Chapter 11.

14.7.2 Portions of grade beam walls exposed above grade shall also meet requirements of 14.3.

SECTION 14.8
ALTERNATIVE DESIGN OF SLENDER WALLS

14.8.1 When flexural tension controls the design of a wall, the requirements of 14.8 are considered to satisfy 10.10.

14.8.2 Walls designed by the provisions of 14.8 shall satisfy 14.8.2.1 through 14.8.2.6.

14.8.2.1 The wall panel shall be designed as a simply supported, axially loaded member subjected to an out-of-plane uniform lateral load, with maximum moments and deflections occurring at midspan.

14.8.2.2 The cross section shall be constant over the height of the panel.

14.8.2.3 The reinforcement ratio ρ shall not exceed $0.6\rho_b$.

14.8.2.4 Reinforcement shall provide a design strength

$$\phi M_n \geq M_{cr} \qquad (14.2)$$

where M_{cr} shall be obtained using the modulus of rupture given by Eq. (9-10).

14.8.2.5 Concentrated gravity loads applied to the wall above the design flexural section shall be assumed to be distributed over a width:

(a) Equal to the bearing width, plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design section; but

(b) Not greater than the spacing of the concentrated loads; and

(c) Not extending beyond the edges of the wall panel.

14.8.2.6 Vertical stress P_u / A_g at the midheight section shall not exceed $0.06 f_c^{'}$.

14.8.3 The design moment strength ϕM_n for combined flexure and axial loads at the midheight cross section shall be

$$\phi M_n \geq M_u \qquad (14\text{-}3)$$

where:

$$M_u = M_{ua} + P_u \Delta_u \qquad (14\text{-}4)$$

M_{ua} is the moment at the midheight section of the wall due to factored loads, and Δ_u is:

$$\Delta_u = \frac{5 M_u \ell_c^2}{(0.75) 48 E_c I_{cr}} \qquad (14\text{-}5)$$

M_u shall be obtained by iteration of deflections, or by direct calculation using Eq. (14-6).

$$M_u = \frac{M_{ua}}{1 - \dfrac{5 P_u \ell_c^2}{(0.75) 48 E_c I_{cr}}} \qquad (14\text{-}6)$$

where:

$$I_{cr} = n A_{se} (d - c)^2 + \frac{\ell_w c^3}{3} \qquad (14\text{-}7)$$

and

$$A_{se} = \frac{P_u + A_s f_y}{f_y} \qquad (14\text{-}8)$$

14.8.4 The maximum deflection Δ_s due to service loads, including $P\Delta$ effects, shall not exceed $\ell_c / 150$. The midheight deflection Δ_s shall be determined by:

$$\Delta_s = \frac{(5M) \ell_c^2}{48 E_c I_e} \qquad (14\text{-}9)$$

$$M = \frac{M_{sa}}{1 - \dfrac{5 P_s \ell_c^2}{48 E_c I_e}} \qquad (14\text{-}10)$$

I_e shall be calculated using the procedure of 9.5.2.3, substituting M for $M_a I_{cr}$ shall be evaluated using Eq. (14-7).

CHAPTER 15
FOOTINGS

SECTION 15.0
NOTATION

A_g = gross area of section, mm^2

d_p = diameter of pile at footing base, mm

β = ratio of long side to short side of footing

SECTION 15.1
SCOPE

15.1.1 Provisions of Chapter 15 shall apply for design of isolated footings and, where applicable, to combined footings and mats.

15.1.2 Additional requirements for design of combined footings and mats are given in 15.10.

SECTION 15.2
LOADS AND REACTIONS

15.2.1 Footings shall be proportioned to resist the factored loads and induced reactions, in accordance with the appropriate design requirements of SBC 304 and as provided in Chapter 15.

15.2.2 Base area of footing or number and arrangement of piles shall be determined from unfactored forces and moments transmitted by footing to soil or piles and permissible soil pressure or permissible pile capacity determined through principles of soil mechanics.

15.2.3 For footings on piles, computations for moments and shears shall be permitted to be based on the assumption that the reaction from any pile is concentrated at pile center.

SECTION 15.3
FOOTINGS SUPPORTING CIRCULAR OR REGULAR POLYGON SHAPED COLUMNS OR PEDESTALS

For location of critical sections for moment, shear, and development of reinforcement in footings, it shall be permitted to treat circular or regular polygon shaped concrete columns or pedestals as square members with the same area.

SECTION 15.4
MOMENT IN FOOTINGS

15.4.1 External moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over entire area of footing on one side of that vertical plane.

15.4.2 Maximum factored moment for an isolated footing shall be computed as prescribed in Section 15.4.1 at critical sections located as follows:

(a) At face of column, pedestal, or wall, for footings supporting a concrete column, pedestal, or wall;

(b) Halfway between middle and edge of wall, for footings supporting a masonry wall;

(c) Halfway between face of column and edge of steel base plate, for footings supporting a column with steel base plate.

15.4.3 In one-way footings and two-way square footings, reinforcement shall be distributed uniformly across entire width of footing.

15.4.4 In two-way rectangular footings, reinforcement shall be distributed in accordance with Section 15.4.4.1 and 15.4.4.2.

15.4.4.1 Reinforcement in long direction shall be distributed uniformly across entire width of footing.

15.4.4.2 For reinforcement in short direction, a portion of the total reinforcement given by Eq. (15-1) shall be distributed uniformly over a band width (centered on centerline of column or pedestal) equal to the length of short side of footing. Remainder of reinforcement required in short direction shall be distributed uniformly outside center band width of footing.

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta+1)} \qquad (15\text{-}1)$$

SECTION 15.5
SHEAR IN FOOTINGS

15.5.1 Shear strength of footings supported on soil or rock shall be in accordance with 11.12.

15.5.2 Location of critical section for shear in accordance with Chapter 11 shall be measured from face of column, pedestal, or wall, for footings supporting a column, pedestal, or wall. For footings supporting a column or pedestal with steel base plates, the critical section shall be measured from location defined in 15.4.2(c).

15.5.3 Where the distance between the axis of any pile to the axis of the column is more than two times the distance between the top of the pile cap and the top of the pile, the pile cap shall satisfy 11.12 and 15.5.4. Other pile caps shall satisfy one of 11.12, 15.5.4, or Appendix A. If Appendix A is used, the effective concrete compression strength of the struts, f_{cu}, shall be determined using A.3.2.2(b).

15.5.4 Computation of shear on any section through a footing supported on piles shall be in accordance with 15.5.4.1, 15.5.4.2, and 15.5.4.3.

15.5.4.1 Entire reaction from any pile whose center is located $d_p/2$ or more outside the section shall be considered as producing shear on that section.

15.5.4.2 Reaction from any pile whose center is located $d_p/2$ or more inside the section shall be considered as producing no shear on that section.

15.5.4.3 For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the section shall be based *on* straight-line interpolation between full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section.

SECTION 15.6
DEVELOPMENT OF REINFORCEMENT IN FOOTINGS

15.6.1 Development of reinforcement in footings shall be in accordance with Chapter 12.

15.6.2 Calculated tension or compression in reinforcement at each section shall be developed on each side of that section by embedment length, hook (tension only) or mechanical device, or a combination thereof.

15.6.3 Critical sections for development of reinforcement shall be assumed at the same locations as defined in 15.4.2 for maximum factored moment, and at all other vertical planes where changes of section or reinforcement occur. See also 12.10.6.

SECTION 15.7
MINIMUM FOOTING DEPTH

15.7.1 Depth of footing above bottom reinforcement shall not be less than 150 mm for footings on soil, nor less than 300 mm for footings on piles.

SECTION 15.8
TRANSFER OF FORCE AT BASE OF COLUMN, WALL OR REINFORCED PEDESTAL

15.8.1 Forces and moments at base of column, wall, or pedestal shall be transferred to supporting pedestal or footing by bearing on concrete and by reinforcement, dowels, and mechanical connectors.

15.8.1.1 Bearing stress on concrete at contact surface between supported and supporting member shall not exceed concrete bearing strength for either surface as given by 10.17.

15.8.1.2 Reinforcement, dowels, or mechanical connectors between supported and supporting members shall be adequate to transfer:

(a) All compressive force that exceeds concrete bearing strength of either member;

(b) Any computed tensile force across interface.

In addition, reinforcement, dowels, or mechanical connectors shall satisfy 15.8.2 or 15.8.3.

15.8.1.3 If calculated moments are transferred to supporting pedestal or footing, then reinforcement, dowels, or mechanical connectors shall be adequate to satisfy 12.17.

15.8.1.4 Lateral forces shall be transferred to supporting pedestal or footing in accordance with shear-friction provisions of 11.7, or by other appropriate means.

15.8.2 In cast-in-place construction, reinforcement required to satisfy 15.8.1 shall be provided either by extending longitudinal bars into supporting pedestal or footing, or by dowels.

15.8.2.1 For cast-in-place columns and pedestals, area of reinforcement across interface shall be not less than 0.005 times gross area of supported member.

15.8.2.2 For cast-in-place walls, area of reinforcement across interface shall be not less than minimum vertical reinforcement given in 14.3.2.

15.8.2.3 At footings, it shall be permitted to lap splice Dia 40 mm and larger longitudinal bars, in compression only, with dowels to provide reinforcement required to satisfy 15.8.1. Dowels shall not be larger than Dia 36 mm bar and shall extend into supported member a distance not less than the development length needed for bars with larger diameters than 40 mm or the splice length of the dowels, whichever is greater, and into the footing a distance not less than the development length of the dowels.

15.8.2.4 If a pinned or rocker connection is provided in cast-in-place construction, connection shall conform to 15.8.1 and 15.8.3.

15.8.3 In precast construction, anchor bolts or suitable mechanical connectors shall be permitted for satisfying 15.8.1. Anchor bolts shall be designed in accordance with Appendix D.

15.8.3.1 Connection between precast columns or pedestals and supporting members shall meet the requirements of 16.5.1.3(a).

15.8.3.2 Connection between precast walls and supporting members shall meet the requirements of 16.5.1.3(b) and (c).

15.8.3.3 Anchor bolts and mechanical connections shall be designed to reach their design strength before anchorage failure or failure of surrounding concrete. Anchor bolts shall be designed in accordance with Appendix D.

SECTION 15.9
SLOPED OR STEPPED FOOTINGS

15.9.1 In sloped or stepped footings, angle of slope or depth and location of steps shall be such that design requirements are satisfied at every section. (See also 12.10.6.)

15.9.2 Sloped or stepped footings designed as a unit shall be constructed to ensure action as a unit.

SECTION 15.10
COMBINED FOOTINGS AND MATS

15.10.1 Footings supporting more than one column, pedestal, or wall (combined footings or mats) shall be proportioned to resist the factored loads and induced reactions, in accordance with appropriate design requirements of the code.

15.10.2 The Direct Design Method of Chapter 13 shall not be used for design of combined footings and mats.

15.10.3 Distribution of soil pressure under combined footings and mats shall be consistent with properties of the soil and the structure and with established principles of soil mechanics.

CHAPTER 16
PRECAST CONCRETE

SECTION 16.0
NOTATION

A_g = gross area of column, mm²

ℓ = clear span, mm

SECTION 16.1
SCOPE

16.1.1 All provisions of SBC 304, not specifically excluded and not in conflict with the provisions of Chapter 16, shall apply to structures incorporating precast concrete structural members.

SECTION 16.2
GENERAL

16.2.1 Design of precast members and connections shall include loading and restraint conditions from initial fabrication to end use in the structure, including form removal, storage, transportation, and erection.

16.2.2 When precast members are incorporated into a structural system, the forces and deformations occurring in and adjacent to connections shall be included in the design.

16.2.3 Tolerances for both precast members and interfacing members shall be specified. Design of precast members and connections shall include the effects of these tolerances.

16.2.4 In addition to the requirements for drawings and specifications in 1.2, (a) and (b) shall be included in either the contract documents or shop drawings:

(a) Details of reinforcement, inserts and lifting devices required to resist temporary loads from handling, storage, transportation, and erection;

(b) Required concrete strength at stated ages or stages of construction.

SECTION 16.3
DISTRIBUTION OF FORCES AMONG MEMBERS

16.3.1 Distribution of forces that are perpendicular to the plane of members shall be established by analysis or by test.

16.3.2 Where the system behavior requires in-plane forces to be transferred between the members of a precast floor or wall system, Section 16.3.2.1 and 16.3.2.2 shall apply.

16.3.2.1 In-plane force paths shall be continuous through both connections and members.

16.3.2.2 Where tension forces occur, a continuous path of steel or steel reinforcement shall be provided.

SECTION 16.4
MEMBER DESIGN

16.4.1 In one-way precast floor and roof slabs and in one-way precast, prestressed wall panels, all not wider than 4 m, and where members are not mechanically connected to cause restraint in the transverse direction, the shrinkage and temperature reinforcement requirements of 7.12 in the direction normal to the flexural reinforcement shall be permitted to be waived. This waiver shall not apply to members that require reinforcement to resist transverse flexural stresses.

16.4.2 For precast, nonprestressed walls the reinforcement shall be designed in accordance with the provisions of Chapters 10 or 14, except that the area of horizontal and vertical reinforcement each shall be not less than 0.001 times the gross cross-sectional area of the wall panel. Spacing of reinforcement shall not exceed 3 times the wall thickness nor 500 mm for interior walls nor 300 mm for exterior walls.

SECTION 16.5
STRUCTURAL INTEGRITY

16.5.1 Except where the provisions of Section 16.5.2 govern, the minimum provisions of 16.5.1.1 through 16.5.1.4 for structural integrity shall apply to all precast concrete structures.

16.5.1.1 Longitudinal and transverse ties required by Section 7.13.3 shall connect members to a lateral load resisting system.

16.5.1.2 Where precast elements form floor or roof diaphragms, the connections between diaphragm and those members being laterally supported shall have a nominal tensile strength capable of resisting not less than 4.5 kN/m.

16.5.1.3 Vertical tension tie requirements of Section 7.13.3 shall apply to all vertical structural members, except cladding, and shall be achieved by providing connections at horizontal joints in accordance with (a) through (c):

(a) Precast columns shall have a nominal strength in tension not less than $1.5A_g$, in kN. For columns with a larger cross section than required by consideration of loading, a reduced effective area A_g based on cross section required but not less than one-half the total area, shall be permitted;

(b) Precast wall panels shall have a minimum of two ties per panel, with a nominal tensile strength not less than 45 kN per tie;

(c) When design forces result in no tension at the base, the ties required by 16.5.1.3(b) shall be permitted to be anchored into an appropriately reinforced concrete floor slab on grade.

16.5.1.4 Connection details that rely solely on friction caused by gravity loads shall not be used.

16.5.2 For precast concrete bearing wall structures three or more stories in height, the minimum provisions of Section 16.5.2.1 through 16.5.2.5 shall apply.

16.5.2.1 Longitudinal and transverse ties shall be provided in floor and roof systems to provide a nominal strength of 20 kN/m of width or length. Ties shall be provided over interior wall supports and between members and exterior walls. Ties shall be positioned in or within 0.6 m of the plane of the floor or roof system.

16.5.2.2 Longitudinal ties parallel to floor or roof slab spans shall be spaced not more than 3.0 m on centers. Provisions shall be made to transfer forces around openings.

16.5.2.3 Transverse ties perpendicular to floor or roof slab spans shall be spaced not greater than the bearing wall spacing.

16.5.2.4 Ties around the perimeter of each floor and roof, within 1.2 m of the edge, shall provide a nominal strength in tension not less than 70 kN.

16.5.2.5 Vertical tension ties shall be provided in all walls and shall be continuous over the height of the building. They shall provide a nominal tensile strength not less than 40 kN per horizontal meter of wall. Not less than two ties shall be provided for each precast panel.

SECTION 16.6
CONNECTION AND BEARING DESIGN

16.6.1 Forces shall be permitted to be transferred between members by grouted joints, shear keys, mechanical connectors, reinforcing steel connections, reinforced topping, or a combination of these means.

16.6.1.1 The adequacy of connections to transfer forces between members shall be determined by analysis or by test. Where shear is the primary result of imposed loading, it shall be permitted to use the provisions of 11.7 as applicable.

16.6.1.2 When designing a connection using materials with different structural properties, their relative stiffnesses, strengths, and ductilities shall be considered.

16.6.2 Bearing for precast floor and roof members on simple supports shall satisfy Section 16.6.2.1 and 16.6.2.2.

16.6.2.1 The allowable bearing stress at the contact surface between supported and supporting members and between any intermediate bearing elements shall not exceed the bearing strength for either surface and the bearing element. Concrete bearing strength shall be as given in 10.17.

16.6.2.2 Unless shown by test or analysis that performance will not be impaired, (a) and (b) shall be met:

(a) Each member and its supporting system shall have design dimensions selected so that, after consideration of tolerances, the distance from the edge of the support to the end of the precast member in the direction of the span is at least 1/180 of the clear span ℓ, but not less than:

For solid or hollow-core slabs ....50 mm

For beams or stemmed members ....75 mm

(b) Bearing pads at unarmored edges shall be set back a minimum of 15 mm from the face of the support, or at least the chamfer dimension at chamfered edges.

16.6.2.3 The requirements of Section 12.11.1 shall not apply to the positive bending moment reinforcement for statically determined precast members, but at least one-third of such reinforcement shall extend to the center of the bearing length, taking into account permitted tolerance in Section 7.5.2.2 and 16.2.3.

SECTION 16.7
ITEMS EMBEDDED AFTER CONCRETE PLACEMENT

16.7.1 When approved by the registered design professional, embedded items (such as dowels or inserts) that either protrude from the concrete or remain exposed for inspection shall be permitted to be embedded while the concrete is in a plastic state provided that Section 16.7.1.1, 16.7.1.2, and 16.7.1.3 are met.

16.7.1.1 Embedded items are not required to be hooked or tied to reinforcement within the concrete.

16.7.1.2 Embedded items are maintained in the correct position while the concrete remains plastic.

16.7.1.3 The concrete is properly consolidated around the embedded item.

SECTION 16.8
MARKING AND IDENTIFICATION

16.8.1 Each precast member shall be marked to indicate its location and orientation in the structure and date of manufacture.

16.8.2 Identification marks shall correspond to placing drawings.

SECTION 16.9
HANDLING

16.9.1 Member design shall consider forces and distortions during curing, stripping, storage, transportation, and erection so that precast members are not overstressed or otherwise damaged.

16.9.2 During erection, precast members and structures shall be adequately supported and braced to ensure proper alignment and structural integrity until permanent connections are completed.

SECTION 16.10
STRENGTH EVALUATION OF PRECAST CONSTRUCTION

16.10.1 A precast element to be made composite with cast-in-place concrete shall be permitted to be tested in flexure as a precast element alone in accordance with Section 16.10.1.1 and 16.10.1.2.

16.10.1.1 Test loads shall be applied only when calculations indicate the isolated precast element will not be critical in compression or buckling.

16.10.1.2 The test load shall be that load which, when applied to the precast member alone, induces the same total force in the tension reinforcement as would be induced by loading the composite member with the test load required by Section 20.3.2.

16.10.2 The provisions of 20.5 shall be the basis for acceptance or rejection of the precast element.

CHAPTER 17
COMPOSITE CONCRETE FLEXURAL MEMBERS

SECTION 17.0
NOTATION

A_c = area of contact surface being investigated for horizontal shear, mm²

A_v = area of ties within a distance s , mm²

b_v = width of cross section at contact surface being investigated for horizontal shear, mm

d = distance from extreme compression fiber to centroid of tension reinforcement for entire composite section, mm

h = overall thickness of composite member, m

s = spacing of ties measured along the longitudinal axis of the member, m

V_{nh} = nominal horizontal shear strength, N

V_u = factored shear force at section, N

λ = correction factor related to unit weight of concrete

ρ_v = ratio of tie reinforcement area to area of contact surface
= $A_v / b_v s$

ϕ = strength reduction factor. See 9.3

SECTION 17.1
SCOPE

17.1.1 Provisions of Chapter 17 shall apply for design of composite concrete flexural members defined as precast concrete, cast-in-place concrete elements, or both, constructed in separate placements but so interconnected that all elements respond to loads as a unit.

17.1.2 All provisions of the code shall apply to composite concrete flexural members, except as specifically modified in Chapter 17.

SECTION 17.2
GENERAL

17.2.1 The use of an entire composite member or portions thereof for resisting shear and moment shall be permitted.

17.2.2 Individual elements shall be investigated for all critical stages of loading.

17.2.3 If the specified strength, unit weight, or other properties of the various elements are different, properties of the individual elements or the most critical values shall be used in design.

17.2.4 In strength computations of composite members, no distinction shall be made between shored and unshored members.

17.2.5 All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.

17.2.6 Reinforcement shall be provided as required to minimize cracking and to prevent separation of individual elements of composite members.

17.2.7 Composite members shall meet requirements for control of deflections in accordance with Section 9.5.5.

SECTION 17.3
SHORING

When used, shoring shall not be removed until supported elements have developed design properties required to support all loads and limit deflections and cracking at time of shoring removal.

SECTION 17.4
VERTICAL SHEAR STRENGTH

17.4.1 When an entire composite member is assumed to resist vertical shear, design shall be in accordance with requirements of Chapter 11 as for a monolithically cast member of the same cross-sectional shape.

17.4.2 Shear reinforcement shall be fully anchored into interconnected elements in accordance with Section 12.13.

17.4.3 Extended and anchored shear reinforcement shall be permitted to be included as ties for horizontal shear.

SECTION 17.5
HORIZONTAL SHEAR STRENGTH

17.5.1 In a composite member, full transfer of horizontal shear forces shall be ensured at contact surfaces of interconnected elements.

17.5.2 Unless calculated in accordance with Section 17.5.3, design of cross sections subject to horizontal shear shall be based on

$$V_u \le \phi V_{nh} \qquad (17\text{-}1)$$

where V_u is factored shear force at the section considered and V_{nh} is nominal horizontal shear strength in accordance with Section 17.5.2.1 through 17.5.2.5.

17.5.2.1 When contact surfaces are clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not be taken greater than $0.6 b_v d$, in N.

17.5.2.2 When minimum ties are provided in accordance with 17.6, and contact surfaces are clean and free of laitance, but not intentionally roughened, shear strength V_{nh} shall not be taken greater than $0.6 b_v d$, in N.

17.5.2.3 When ties are provided in accordance with 17.6, and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 5 mm, shear strength V_{nh} shall be taken equal to $(1.8 + 0.6 \rho_v f_y) \lambda b_v d$, in N, but not greater than $3.5 b_v d$, in N. Values for λ in 11.7.4.3 shall apply.

17.5.2.4 When factored shear force V_u at section considered exceeds $\phi(3.5b_v d)$, design for horizontal shear shall be in accordance with Section 11.7.4.

17.5.2.5 When determining nominal horizontal shear strength over prestressed concrete elements, d shall be as defined or $0.8h$, whichever is greater.

17.5.3 As an alternative to 17.5.2, horizontal shear shall be permitted to be determined by computing the actual change in compressive or tensile force in any segment, and provisions shall be made to transfer that force as horizontal shear to the supporting element. The factored horizontal shear force shall not exceed horizontal shear strength ϕV_{nh} as given in Section 17.5.2.1 through 17.5.2.4, where area of contact surface A_c shall be substituted for $b_v d$.

17.5.3.1 When ties provided to resist horizontal shear are designed to satisfy Section 17.5.3, the tie area to tie spacing ratio along the member shall approximately reflect the distribution of shear forces in the member.

17.5.4 When tension exists across any contact surface between interconnected elements, shear transfer by contact shall be permitted only when minimum ties are provided in accordance with 17.6.

SECTION 17.6
TIES FOR HORIZONTAL SHEAR

17.6.1 When ties are provided to transfer horizontal shear, tie area shall not be less than that required by 11.5.5.3, and tie spacing shall not exceed four times the least dimension of supported element, nor exceed 600 mm.

17.6.2 Ties for horizontal shear shall consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric (plain or deformed).

17.6.3 All ties shall be fully anchored into interconnected elements in accordance with 12.13.

CHAPTER 18
PRESTRESSED CONCRETE

SECTION 18.0
NOTATION

A = area of that part of cross section between flexural tension face and center of gravity of gross section, mm²

A_{cf} = larger gross cross-sectional area of the slab-beam strips of the two orthogonal equivalent frames intersecting at a column of a two-way slab, mm²

A_{ps} = area of prestressed reinforcement in tension zone, mm²

A_s = area of nonprestressed tension reinforcement, mm²

$A_s^{'}$ = area of compression reinforcement, mm²

b = width of compression face of member, mm

c_c = clear cover from the nearest surface in tension to the surface of the flexural tension steel, mm

d = distance from extreme compression fiber to centroid of nonprestressed tension reinforcement, mm

$d^{'}$ = distance from extreme compression fiber to centroid of compression reinforcement, mm

d_p = distance from extreme compression fiber to centroid of prestressed reinforcement, mm

D = dead loads, or related internal moments and forces

e = base of Napierian logarithms

$f_c^{'}$ = specified compressive strength of concrete, MPa

$\sqrt{f_c^{'}}$ = square root of specified compressive strength of concrete, MPa

$f_{ci}^{'}$ = compressive strength of concrete at time of initial prestress, MPa

$\sqrt{f_{ci}^{'}}$ = square root of compressive strength of concrete at time of initial prestress, MPa

f_{dc} = decompression stress. Stress in the prestressing steel when stress is zero in the concrete at the same level as the centroid of the tendons, MPa

f_{pc} = average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses), MPa

f_{ps} = stress in prestressed reinforcement at nominal strength, MPa

f_{pu} = specified tensile strength of prestressing steel, MPa

f_{py} = specified yield strength of prestressing steel, MPa

f_r = modulus of rupture of concrete, MPa

f_{se} = effective stress in prestressed reinforcement (after allowance for all prestress losses), MPa

f_t = extreme fiber stress in tension in the precompressed tensile zone, computed using gross section properties, MPa

f_y = specified yield strength of nonprestressed reinforcement, MPa

h = overall thickness of member, mm

K = wobble friction coefficient per meter of tendon

ℓ_x = length of prestressing steel element from jacking end to any point x, m. See Eq. (18-1) and (18-2)

L	=	live loads, or related internal moments and forces
n	=	number of monostrand anchorage devices in a group
N_c	=	tensile force in concrete due to unfactored dead load plus live load $(D+L)$, N
P_s	=	prestressing force at jacking end, N
P_{su}	=	factored prestressing force at the anchorage device, N
P_x	=	prestressing force at any point x, N
s	=	center-to-center spacing of flexural tension steel near the extreme tension face, mm. Where there is only one bar or tendon near the extreme tension face, s is the width of extreme tension face
α	=	total angular change of tendon profile in radians from tendon jacking end to any point x
β_1	=	factor defined in 10.2.7.3
Δf_{ps}	=	stress in prestressing steel at service loads less decompression stress, MPa
γ_p	=	factor for type of prestressing steel
	=	0.55 for f_{py}/f_{pu} not less than 0.80
	=	0.40 for f_{py}/f_{pu} not less than 0.85
	=	0.28 for f_{py}/f_{pu} not less than 0.90
λ	=	correction factor related to unit weight of concrete (See 11.7.4.3)
μ	=	curvature friction coefficient
ρ	=	ratio of nonprestressed tension reinforcement
	=	A_s/bd
ρ'	=	ratio of compression reinforcement
	=	A_s'/bd
ρ_p	=	ratio of prestressed reinforcement
	=	A_{ps}/bd_p
ϕ	=	strength reduction factor. See 9.3
ω	=	$\rho f_y/f_c'$
ω'	=	$\rho' f_y/f_c'$
ω_p	=	$\rho_p f_{ps}/f_c'$
$\omega_w, \omega_{pw}, \omega_w'$	=	reinforcement indices for flanged sections. computed as for ω, ω_p and ω' except that b shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only

SECTION 18.1
SCOPE

18.1.1 Provisions of Chapter 18 shall apply to members prestressed with wire, strands, or bars conforming to provisions for prestressing steel in 3.5.5.

18.1.2 All provisions of SBC 304 not specifically excluded, and not in conflict with provisions of Chapter 18, shall apply to prestressed concrete.

18.1.3 The following provisions of SBC 304 shall not apply to prestressed concrete, except as specifically noted: Sections 7.6.5, 8.10.2, 8.10.3, 8.10.4, 8.11, 10.5, 10.6, 10.9.1, and 10.9.2; Chapter 13; and Sections 14.3, 14.5, and 14.6, except that certain sections of 10.6 apply as noted in 18.4.4.

SECTION 18.2
GENERAL

18.2.1 Prestressed members shall meet the strength requirements of SBC 304.

18.2.2 Design of prestressed members shall be based on strength and on behavior at service conditions at all stages that will be critical during the life of the structure from the time prestress is first applied.

18.2.3 Stress concentrations due to prestressing shall be considered in design.

18.2.4 Provisions shall be made for effects on adjoining construction of elastic and plastic deformations, deflections, changes in length, and rotations due to prestressing. Effects of temperature and shrinkage shall also be included.

18.2.5 The possibility of buckling in a member between points where there is intermittent contact between the prestressing steel and an oversize duct, and buckling in thin webs and flanges shall be considered.

18.2.6 In computing section properties before bonding of prestressing steel, effect of loss of area due to open ducts shall be considered.

SECTION 18.3
DESIGN ASSUMPTIONS

18.3.1 Strength design of prestressed members for flexure and axial loads shall be based on assumptions given in 10.2, except that 10.2.4 shall apply only to reinforcement conforming to 3.5.3.

18.3.2 For investigation of stresses at transfer of prestress, at service loads, and at cracking loads, elastic theory shall be used with the assumptions of Section 18.3.2.1 and 18.3.2.2.

18.3.2.1 Strains vary linearly with depth through the entire load range.

18.3.2.2 At cracked sections, concrete resists no tension.

18.3.3 Prestressed flexural members shall be classified as Class U, Class T, or Class C based on the computed extreme fiber stress f_t at service loads in the precompressed tensile zone, as follows:

(a) Class U: $f_t \leq 0.7\sqrt{f_c^{'}}$;

(b) Class T: $0.7\sqrt{f_c^{'}} < f_t \leq \sqrt{f_c^{'}}$;

(c) Class C: $f_t > \sqrt{f_c^{'}}$;

Prestressed two-way slab systems shall be designed as Class U.

18.3.4 For Class U and Class T flexural members, stresses at service loads shall be permitted to be calculated using the uncracked section. For Class C flexural members, stresses at service loads shall be calculated using the cracked transformed section.

18.3.5 Deflections of prestressed flexural members shall be calculated in accordance with 9.5.4

SECTION 18.4
SERVICEABILITY REQUIREMENTS - FLEXURAL MEMBERS

18.4.1 Stresses in concrete immediately after prestress transfer (before time-dependent prestress losses) shall not exceed the following:

(a) Extreme fiber stress in compression……………………… $0.60 f'_{ci}$

(b) Extreme fiber stress in tension except as permitted in (c) ....…………………………………………………….…… $(1/4)\sqrt{f'_{ci}}$

(c) Extreme fiber stress in tension at ends of simply supported members…………………………………………………$(1/2)\sqrt{f'_{ci}}$

Where computed tensile stresses exceed these values, bonded additional reinforcement (nonprestressed or prestressed) shall be provided in the tensile zone to resist the total tensile force in concrete computed with the assumption of an uncracked section.

18.4.2 For Class U and Class T prestressed flexural members, stresses in concrete at service loads (based on uncracked section properties, and after allowance for all prestress losses) shall not exceed the following:

(a) Extreme fiber stress in compression due to prestress plus sustained load ……..………………..……………………………… $0.45 f'_c$

(b) Extreme fiber stress in compression due to prestress plus total load……………..………………………………………... $0.60 f'_c$

18.4.3 Permissible stresses in 18.4.1 and 18.4.2 shall be permitted to be exceeded if shown by test or analysis that performance will not be impaired.

18.4.4 For Class C prestressed flexural members not subject to fatigue or to aggressive exposure, the spacing of bonded reinforcement nearest the extreme tension face shall not exceed that given by 10.6.4.

For structures subject to fatigue or exposed to corrosive environments, special investigations and precautions are required.

18.4.4.1 The spacing requirements shall be met by nonprestressed reinforcement and bonded tendons. The spacing of bonded tendons shall not exceed 2/3 of the maximum spacing permitted for nonprestressed reinforcement.

Where both reinforcement and bonded tendons are used to meet the spacing requirement, the spacing between a bar and a tendon shall not exceed 5/6 of that permitted by Section 10.6.4. See also 18.4.4.3.

18.4.4.2 In applying Eq. (10-4) to prestressing tendons, Δf_{ps} shall be substituted for f_s , where Δf_{ps} shall be taken as the difference between the stress computed in the prestressing tendons at service loads based on a cracked section analysis and the decompression stress f_{dc} in the prestressing tendons. It shall be permitted to take f_{dc} equal to the effective prestress f_{se}. See also 18.4.4.3.

18.4.4.3 The magnitude of Δf_{ps} shall not exceed 250 MPa. When Δf_{ps} is less than or equal to 140 MPa, the spacing requirements of Section 18.4.4.1 and 18.4.4.2 shall not apply.

18.4.4.4 If the effective depth of a beam exceeds 1 m, the area of skin reinforcement consisting of reinforcement or bonded tendons shall be provided as required by 10.6.7.

SECTION 18.5
PERMISSIBLE STRESSES IN PRESTRESSING STEEL

18.5.1 Tensile stress in prestressing steel shall not exceed the following:

(a) Due to prestressing steel jacking force $0.94 f_{py}$ but not greater than the lesser of $0.80 f_{pu}$ and the maximum value recommended by the manufacturer of prestressing steel or anchorage devices.

(b) Immediately after prestress transfer…………………….. $0.82 f_{py}$
but not greater than $0.74 f_{pu}$

(c) Post-tensioning tendons, at anchorage devices and couplers, immediately after force transfer ……..………………... $0.70 f_{pu}$

SECTION 18.6
LOSS OF PRESTRESS

18.6.1 To determine effective prestress f_{se}, allowance for the following sources of loss of prestress shall be considered:

(a) Prestressing steel seating at transfer;

(b) Elastic shortening of concrete;

(c) Creep of concrete;

(d) Shrinkage of concrete;

(e) Relaxation of prestressing steel stress;

(f) Friction loss due to intended or unintended curvature in post-tensioning tendons.

18.6.2 Friction loss in post-tensioning tendons

18.6.2.1 Effect of friction loss in post-tensioning tendons shall be computed by

$$P_s = P_x e^{(K\ell_x + \mu\alpha)} \qquad (18\text{-}1)$$

When $(Kl_x + \mu\alpha)$ is not greater than 0.3, effect of friction loss shall be permitted to be computed by

$$P_s = P_x(1 + K\ell_x + \mu\alpha) \tag{18-2}$$

18.6.2.2 Friction loss shall be based on experimentally determined wobble K and curvature μ friction coefficients, and shall be verified during tendon stressing operations.

18.6.2.3 Values of wobble and curvature friction coefficients used in design shall be shown on design drawings.

18.6.3 Where loss of prestress in a member occurs due to connection of the member to adjoining construction, such loss of prestress shall be allowed for in design.

SECTION 18.7 FLEXURAL STRENGTH

18.7.1 Design moment strength of flexural members shall be computed by the strength design methods of the code. For prestressing steel, f_{ps} shall be substituted for f_y in strength computations.

18.7.2 As an alternative to a more accurate determination of f_{ps} based on strain compatibility, the following approximate values of f_{ps} shall be permitted to be used if f_{ps} is not less than $0.5f_{pu}$

(a) For members with bonded tendons:

$$f_{ps} = f_{pu}\left\{1 - \frac{\gamma_p}{\beta_1}\left[\rho_p \frac{f_{pu}}{f_c^{'}} + \frac{d}{d_p}(\omega - \omega')\right]\right\} \tag{18.3}$$

If any compression reinforcement is taken into account when calculating f_{ps} by Eq. (18-3), the term

$$\left[\rho_p \frac{f_{pu}}{f_c^{'}} + \frac{d}{d_p}(\omega - \omega')\right]$$

shall be taken not less than 0.17 and $d^{'}$ shall be no greater than $0.15d_p$

(b) For members with unbonded tendons and with a span-to-depth ratio of 35 or less:

$$f_{ps} = f_{se} + 70 + \frac{f_c'}{100\rho_p} \tag{18-4}$$

but f_{ps} in Eq. (18-4) shall not be taken greater than f_{py} nor greater than $(f_{se} + 420)$.

(c) For members with unbonded tendons and with a span-to-depth ratio greater than 35:

$$f_{ps} = f_{se} + 70 + \frac{f_c'}{300\rho_p} \tag{18-5}$$

but f_{ps} in Eq. (18-5) shall not be taken greater than f_{py} nor greater than $(f_{se} + 200)$.

18.7.3 Nonprestressed reinforcement conforming to 3.5.3, if used with prestressing steel, shall be permitted to be considered to contribute to the tensile force and to be included in moment strength computations at a stress equal to the specified yield strength f_y. Other nonprestressed reinforcement shall be permitted to be included in strength computations only if a strain compatibility analysis is performed to determine stresses in such reinforcement.

SECTION 18.8
LIMITS FOR REINFORCEMENT OF FLEXURAL MEMBERS

18.8.1 Prestressed concrete sections shall be classified as either tension-controlled, transition, or compression-controlled sections, in accordance with Section 10.3.3 and 10.3.4. The appropriate ϕ-factors from 9.3.2 shall apply.

18.8.2 Total amount of prestressed and nonprestressed reinforcement shall be adequate to develop a factored load at least 1.2 times the cracking load computed on the basis of the modulus of rupture f_r specified in Section 9.5.2.3. This provision shall be permitted to be waived for:

(a) Two-way, unbonded post-tensioned slabs; and

(b) Flexural members with shear and flexural strength at least twice that required by 9.2.

18.8.3 Part or all of the bonded reinforcement consisting of bars or tendons shall be provided as close as practicable to the extreme tension fiber in all prestressed flexural members, except that in members prestressed with unbonded tendons, the minimum bonded reinforcement consisting of bars or tendons shall be as required by Section 18.9.

SECTION 18.9
MINIMUM BONDED REINFORCEMENT

18.9.1 A minimum area of bonded reinforcement shall be provided in all flexural members with unbonded tendons as required by Section 18.9.2 and 18.9.3.

18.9.2 Except as provided in 18.9.3, minimum area of bonded reinforcement shall be computed by

$$A_s = 0.004A \qquad (18\text{-}6)$$

18.9.2.1 Bonded reinforcement required by Eq. (18-6) shall be uniformly distributed over pre-compressed tensile zone as close as practicable to extreme tension fiber.

18.9.2.2 Bonded reinforcement shall be required regardless of service load stress conditions.

18.9.3 For two-way flat slab systems, minimum area and distribution of bonded reinforcement shall be as required in 18.9.3.1, 18.9.3.2, and 18.9.3.3.

18.9.3.1 Bonded reinforcement shall not be required in positive moment areas where computed concrete tensile stress at service load (after allowance for all prestress losses) does not exceed $(1/6)\sqrt{f_c'}$.

18.9.3.2 In positive moment areas where computed tensile stress in concrete at service load exceeds $(1/6)\sqrt{f_c'}$, minimum area of bonded reinforcement shall be computed by

$$A_s = \frac{N_c}{0.5 f_y} \qquad (18\text{-}7)$$

where design yield strength f_y shall not exceed 420 MPa. Bonded reinforcement shall be uniformly distributed over precompressed tensile zone as close as practicable to the extreme tension fiber.

18.9.3.3 In negative moment areas at column supports, the minimum area of bonded reinforcement A_s in the top of the slab in each direction shall be computed by

$$A_s = 0.00075 A_{cf} \qquad (18\text{-}8)$$

Bonded reinforcement required by Eq.(18-8) shall be distributed between lines that are $1.5h$ outside opposite faces of the column support. At least four bars or wires shall be provided in each direction. Spacing of bonded reinforcement shall not exceed 300 mm.

18.9.4 Minimum length of bonded reinforcement required by 18.9.2 and 18.9.3 shall be as required in 18.9.4.1, 18.9.4.2, and 18.9.4.3.

18.9.4.1 In positive moment areas, minimum length of bonded reinforcement shall be one-third the clear span length and centered in positive moment area.

18.9.4.2 In negative moment areas, bonded reinforcement shall extend one-sixth the clear span on each side of support.

18.9.4.3 Where bonded reinforcement is provided for design moment strength in accordance with Section 18.7.3, or for tensile stress conditions in accordance with 18.9.3.2, minimum length also shall conform to provisions of Chapter 12.

SECTION 18.10
STATICALLY INDETERMINATE STRUCTURES

18.10.1 Frames and continuous construction of prestressed concrete shall be designed for satisfactory performance at service load conditions and for adequate strength.

18.10.2 Performance at service load conditions shall be determined by elastic analysis, considering reactions, moments, shears, and axial forces induced by prestressing, creep, shrinkage, temperature change, axial deformation, restraint of attached structural elements, and foundation settlement.

18.10.3 Moments used to compute required strength shall be the sum of the moments due to reactions induced by prestressing (with a load factor of 1.0) and the moments due to factored loads. Adjustment of the sum of these moments shall be permitted as allowed in 18.10.4.

18.10.4 **Redistribution of negative moments in continuous prestressed flexural members**

18.10.4.1 Where bonded reinforcement is provided at supports in accordance with 18.9, it shall be permitted to increase or decrease negative moments calculated by elastic theory for any assumed loading, in accordance with 8.4.

18.10.4.2 The modified negative moments shall be used for calculating moments at sections within spans for the same loading arrangement.

SECTION 18.11
COMPRESSION MEMBERS-COMBINED FLEXURE AND AXIAL LOADS

18.11.1 Prestressed concrete members subject to combined flexure and axial load, with or without nonprestressed reinforcement, shall be proportioned by the strength design methods of SBC 304. Effects of prestress, creep, shrinkage, and temperature change shall be included.

18.1 1.2 Limits for reinforcement of prestressed compression members

18.11.2.1 Members with average prestress f_{pc} less than 1.5 MPa shall have minimum reinforcement in accordance with Section 7.10, 10.9.1 and 10.9.2 for columns, or 14.3 for walls.

18.11.2.2 Except for walls, members with average prestress f_{pc} equal to or greater than 1.5 MPa shall have all tendons enclosed by spirals or lateral ties in accordance with (a) through (d):

- **(a)** Spirals shall conform to 7.10.4;
- **(b)** Lateral ties shall be at least Dia 10 mm in size or welded wire fabric of equivalent area, and shall be spaced vertically not to exceed 48 tie bar or wire diameters, or the least dimension of the compression member;
- **(c)** Ties shall be located vertically not more than half a tie spacing above top of footing or slab in any story, and not more than half a tie spacing below the lowest horizontal reinforcement in members supported above;
- **(d)** Where beams or brackets frame into all sides of a column, ties shall be terminated not more than 75 mm below lowest reinforcement in such beams or brackets.

18.11.2.3 For walls with average prestress f_{pc} equal to or greater than 1.5 MPa, minimum reinforcement required by 14.3 shall not apply where structural analysis shows adequate strength and stability.

SECTION 18.12
SLAB SYSTEMS

18.12.1 Factored moments and shears in prestressed slab systems reinforced for flexure in more than one direction shall be determined in accordance with provisions of 13.7 (excluding 13.7.7.4 and 13.7.7.5), or by more detailed design procedures.

18.12.2 Design moment strength of prestressed slabs required by 9.3 at every section shall be equal to or exceed the required strength considering Section 9.2, 18.10.3, and 18.10.4. Design shear strength of prestressed slabs at columns required by 9.3 shall be equal to or exceed the required strength considering Section 9.2, 11.1, 11.12.2, and 11.12.6.2.

18.12.3 At service load conditions, all serviceability limitations, including limits on deflections, shall be met, with appropriate consideration of the factors listed in 18.10.2.

18.12.4 For normal live loads and loads uniformly distributed, spacing of tendons or groups of tendons in one direction shall not exceed eight times the slab thickness, nor 1.5 m. Spacing of tendons also shall provide a minimum average prestress (after allowance for all prestress losses) of 0.9 MPa on the slab section tributary to the tendon or tendon group. A minimum of two tendons shall be provided in each direction through the critical shear section over columns. Special consideration of tendon spacing shall be provided for slabs with concentrated loads.

18.12.5 In slabs with unbonded tendons, bonded reinforcement shall be provided in accordance with 18.9.3 and 18.9.4.

18.12.6 In lift slabs, bonded bottom reinforcement shall be detailed in accordance with Section 13.3.8.6.

SECTION 18.13
POST-TENSIONED TENDON ANCHORAGE ZONES

18.13.1 Anchorage zone

The anchorage zone shall be considered as composed of two zones:

(a) The local zone is the rectangular prism (or equivalent rectangular prism for circular or oval anchorages) of concrete immediately surrounding the anchorage device and any confining reinforcement;

(b) The general zone is the anchorage zone as defined in 2.1 and includes the local zone.

18.13.2 Local zone

18.13.2.1 Design of local zones shall be based upon the factored prestressing force, P_{su} and the requirements of Section 9.2.5 and 9.3.2.5.

18.13.2.2 Local-zone reinforcement shall be provided where required for proper functioning of the anchorage device.

18.13.2.3 Local-zone requirements of Section 18.13.2.2 are satisfied by Section 18.14.1 or 18.15.1 and 18.15.2.

18.13.3 General zone

18.13.3.1 Design of general zones shall be based upon the factored prestressing force, P_{su} and the requirements of 9.2.5 and 9.3.2.5.

18.13.3.2 General-zone reinforcement shall be provided where required to resist bursting, spalling, and longitudinal edge tension forces induced by anchorage devices. Effects of abrupt change in section shall be considered.

18.13.3.3 The general-zone requirements of Section 18.13.3.2 are satisfied by Section 18.13.4, 18.13.5, 18.13.6 and whichever one of Section 18.14.2 or 18.14.3 or 18.15.3 is applicable.

18.13.4 Nominal material strengths

18.13.4.1 Nominal tensile strength of bonded reinforcement is limited to f_y for nonprestressed reinforcement and to f_{py} for prestressed reinforcement. Nominal tensile stress of unbonded prestressed reinforcement for resisting tensile forces in the anchorage zone shall be limited to $f_{ps} = f_{se} + 70$.

18.13.4.2 Except for concrete confined within spirals or hoops providing confinement equivalent to that corresponding to Eq. (10-5), nominal compressive strength of concrete in the general zone shall be limited to $0.7\lambda f_{ci}^{'}$.

18.13.4.3 Compressive strength of concrete at time of post-tensioning shall be specified on the design drawings. Unless oversize anchorage devices sized to compensate for the lower compressive strength are used or the prestressing steel is stressed to no more than 50 percent of the final prestressing force, prestressing steel shall not be stressed until $f_{ci}^{'}$ as indicated by tests consistent with the curing of the member, is at least 28 MPa for multistrand tendons or at least 20 MPa for single-strand or bar tendons.

18.13.5 Design methods

18.13.5.1 The following methods shall be permitted for the design of general zones provided that the specific procedures used result in prediction of strength in substantial agreement with results of comprehensive tests:

(a) Equilibrium based plasticity models (strut-and-tie models);

(b) Linear stress analysis (including finite element analysis or equivalent); or

(c) Simplified equations where applicable.

18.13.5.2 Simplified equations shall not be used where member cross sections are nonrectangular, where discontinuities in or near the general zone cause deviations in the force flow path, where minimum edge distance is less than 1-1/2 times the anchorage device lateral dimension in that direction, or where multiple anchorage devices are used in other than one closely spaced group.

18.13.5.3 The stressing sequence shall be specified on the design drawings and considered in the design.

18.13.5.4 Three-dimensional effects shall be considered in design and analyzed using three-dimensional procedures or approximated by considering the summation of effects for two orthogonal planes.

18.13.5.5 For anchorage devices located away from the end of the member, bonded reinforcement shall be provided to transfer at least $0.35P_{su}$ into the concrete section behind the anchor. Such reinforcement shall be placed symmetrically around the anchorage devices and shall be fully developed both behind and ahead of the anchorage devices.

18.13.5.6 Where tendons are curved in the general zone, except for monostrand tendons in slabs or where analysis shows reinforcement is not required, bonded reinforcement shall be provided to resist radial and splitting forces.

18.13.5.7 Except for monostrand tendons in slabs or where analysis shows reinforcement is not required, minimum reinforcement with a nominal tensile strength equal to 2 percent of each factored prestressing force shall be provided in orthogonal directions parallel to the back face of all anchorage zones to limit spalling.

18.13.5.8 Tensile strength of concrete shall be neglected in calculations of reinforcement requirements.

18.13.6 Detailing requirements

Selection of reinforcement sizes, spacings, cover, and other details for anchorage zones shall make allowances for tolerances on the bending, fabrication, and placement of reinforcement, for the size of aggregate, and for adequate placement and consolidation of the concrete.

SECTION 18.14
DESIGN OF ANCHORAGE ZONES FOR MONOSTRAND OR SINGLE 16 MM DIAMETER BAR TENDONS

18.14.1 Local zone design

Monostrand or single 16 mm or smaller diameter bar anchorage devices and local zone reinforcement shall meet the requirements of ACI 423.6 or the special anchorage device requirements of 18.15.2.

18.14.2 General-zone design for slab tendons

18.14.2.1 For anchorage devices for 12.5 mm or smaller diameter strands in normalweight concrete slabs, minimum reinforcement meeting the requirements of 18.14.2.2 and 18.14.2.3 shall be provided unless a detailed analysis satisfying 18.13.5 shows such reinforcement is not required.

18.14.2.2 Two horizontal bars at least Dia 14 mm in size shall be provided parallel to the slab edge. They shall be permitted to be in contact with the front face of the anchorage device and shall be within a distance of $(1/2)h$ ahead of each device. Those bars shall extend at least 150 mm either side of the outer edges of each device.

18.14.2.3 If the center-to-center spacing of anchorage devices is 300 mm or less, the anchorage devices shall be considered as a group. For each group of six or more anchorage devices, $n+1$ hairpin bars or closed stirrups at least Dia 10 mm in size shall be provided, where n is the number of anchorage devices. One hairpin bar or stirrup shall be placed between each anchorage device and one on each side of the group. The hairpin bars or stirrups shall be placed with the legs extending into the slab perpendicular to the edge. The center portion of the hairpin bars or stirrups shall be placed perpendicular to the plane of the slab from $3h/8$ to $h/2$ ahead of the anchorage devices.

18.14.2.4 For anchorage devices not conforming to Section 18.14.2.1, minimum reinforcement shall be based upon a detailed analysis satisfying 18.13.5.

18.14.3 General-zone design for groups of monostrand tendons in beams and girders

Design of general zones for groups of monostrand tendons in beams and girders shall meet the requirements of Section 18.13.3 through 18.13.5.

SECTION 18.15
DESIGN OF ANCHORAGE ZONES FOR MULTISTRAND TENDONS

18.15.1 Local zone design

Basic multistrand anchorage devices and local zone reinforcement shall meet the requirements of AASHTO "Standard Specification for Highway Bridges," Divisions, Articles 9.21.7.2.2 through 9.21.7.2.4.
Special anchorage devices shall satisfy the tests required in AASHTO "Standard Specification for Highway Bridges," Division I, Article 9.21.7.3 and described in AASHTO "Standard Specification for Highway Bridges," Division II, Article 10.3.2.3.

18.15.2 Use of special anchorage devices

Where special anchorage devices are to be used, supplemental skin reinforcement shall be furnished in the corresponding regions of the anchorage zone, in addition to the confining reinforcement specified for the anchorage device. This supplemental reinforcement shall be similar in configuration and at least equivalent in volumetric ratio to any supplementary skin reinforcement used in the qualifying acceptance tests of the anchorage device.

18.15.3 General-zone design

Design for general zones for multistrand tendons shall meet the requirements of Section 18.13.3 through 18.13.5.

SECTION 18.16
CORROSION PROTECTION FOR UNBONDED TENDONS

18.16.1 Unbonded prestressing steel shall be encased with sheathing. The prestressing steel shall be completely coated and the sheathing around the prestressing steel filled with suitable material to inhibit corrosion.

18.16.2 Sheathing shall be watertight and continuous over entire length to be unbonded.

18.16.3 For applications in corrosive environments, the sheathing shall be connected to all stressing, intermediate and fixed anchorages in a watertight fashion.

18.16.4 Unbonded single strand tendons shall be protected against corrosion in accordance with ACI's "Specification for Unbonded Single Strand Tendons (ACI 423.6)".

SECTION 18.17
POST-TENSIONING DUCTS

18.17.1 Ducts for grouted tendons shall be mortar-tight and nonreactive with concrete, prestressing steel, grout, and corrosion inhibitor.

18.17.2 Ducts for grouted single wire, single strand, or single bar tendons shall have an inside diameter at least 5 mm larger than the prestressing steel diameter.

18.17.3 Ducts for grouted multiple wire, multiple strand, or multiple bar tendons shall have an inside cross-sectional area at least two times the cross-sectional area of the prestressing steel.

18.17.4 Ducts shall be maintained free of ponded water if members to be grouted are exposed to temperatures below freezing prior to grouting.

SECTION 18.18
GROUT FOR BONDED TENDONS

18.18.1 Grout shall consist of Portland cement and water; or Portland cement, sand, and water.

18.18.2 Materials for grout shall conform to Section 18.18.2.1 through 18.18.2.4.

18.18.2.1 Portland cement shall conform to 3.2.

18.18.2.2 Water shall conform to 3.4.

18.18.2.3 Sand, if used, shall conform to "Standard Specification for Aggregate for Masonry Mortar" (ASTM C 144) except that gradation shall be permitted to be modified as necessary to obtain satisfactory workability.

18.18.2.4 Admixtures conforming to 3.6 and known to have no injurious effects on grout, steel, or concrete shall be permitted. Calcium chloride shall not be used.

18.18.3 Selection of grout proportions

18.18.3.1 Proportions of materials for grout shall be based on either (a) or (b):

- **(a)** Results of tests on fresh and hardened grout prior to beginning grouting operations; or
- **(b)** Prior documented experience with similar materials and equipment and under comparable field conditions.

18.18.3.2 Cement used in the work shall correspond to that on which selection of grout proportions was based.

18.18.3.3 Water content shall be minimum necessary for proper pumping of grout; however, water-cement ratio shall not exceed 0.45 by weight.

18.18.3.4 Water shall not be added to increase grout flowability that has been decreased by delayed use of the grout.

18.18.4 Mixing and pumping grout

18.18.4.1 Grout shall be mixed in equipment capable of continuous mechanical mixing and agitation that will produce uniform distribution of materials, passed through screens, and pumped in a manner that will completely fill the ducts.

18.18.4.2 Temperature of members at time of grouting shall be above 2°C and shall be maintained above 2°C until field-cured 50 mm cubes of grout reach a minimum compressive strength of 6 MPa.

18.18.4.3 Grout temperatures shall not be above 30°C during mixing and pumping.

SECTION 18.19
PROTECTION FOR PRESTRESSING STEEL

18.19.1 Burning or welding operations in the vicinity of prestressing steel shall be performed so that prestressing steel is not subject to excessive temperatures, welding sparks, or ground currents.

SECTION 18.20
APPLICATION AND MEASUREMENT OF PRESTRESSING FORCE

18.20.1 Prestressing force shall be determined by both of (a) and (b):

(a) Measurement of steel elongation. Required elongation shall be determined from average load-elongation curves for the prestressing steel used;

(b) Observation of jacking force on a calibrated gage or load cell or by use of a calibrated dynamometer.

Cause of any difference in force determination between (a) and (b) that exceeds 5 percent for pretensioned elements or 7 percent for post-tensioned construction shall be ascertained and corrected.

18.20.2 Where the transfer of force from the bulk-heads of pre-tensioning bed to the concrete is accomplished by flame cutting prestressing steel, cutting points and cutting sequence shall be predetermined to avoid undesired temporary stresses.

18.20.3 Long lengths of exposed pretensioned strand shall be cut near the member to minimize shock to concrete.

18.20.4 Total loss of prestress due to unreplaced broken prestressing steel shall not exceed 2 percent of total prestress.

SECTION 18.21
POST-TENSIONING ANCHORAGES AND COUPLERS

18.21.1 Anchorages and couplers for bonded and unbonded tendons shall develop at least 95 percent of the specified breaking strength of the prestressing steel, when tested in an unbonded condition, without exceeding anticipated set. For bonded tendons, anchorages and couplers shall be located so that 100 percent of the specified breaking strength of the prestressing steel shall be developed at critical sections after the prestressing steel is bonded in the member.

18.21.2 Couplers shall be placed in areas approved by the engineer and enclosed in housing long enough to permit necessary movements.

18.21.3 In unbonded construction subject to repetitive loads, special attention shall be given to the possibility of fatigue in anchorages and couplers.

18.21.4 Anchorages, couplers, and end fittings shall be permanently protected against corrosion.

SECTION 18.22
EXTERNAL POST-TENSIONING

18.22.1 Post-tensioning tendons shall be permitted to be external to any concrete section of a member. The strength and serviceability design methods of SBC 304 shall be used in evaluating the effects of external tendon forces on the concrete structure.

18.22.2 External tendons shall be considered as unbonded tendons when computing flexural strength unless provisions are made to effectively bond the external tendons to the concrete section along its entire length.

18.22.3 External tendons shall be attached to the concrete member in a manner that maintains the desired eccentricity between the tendons and the concrete centroid throughout the full range of anticipated member deflection.

18.22.4 External tendons and tendon anchorage regions shall be protected against corrosion, and the details of the protection method shall be indicated on the drawings or in the project specifications.

CHAPTER 19
SHELLS AND FOLDED PLATE MEMBERS

SECTION 19.0
NOTATION

E_c = modulus of elasticity of concrete, MPa. See 8.5.1

f_c' = specified compressive strength of concrete, MPa

$\sqrt{f_c'}$ = square root of specified compressive strength of concrete, MPa

f_y = specified yield strength of nonprestressed reinforcement, MPa

h = thickness of shell or folded plate, mm

ℓ_d = development length, mm

ϕ = strength reduction factor. See 9.3

SECTION 19.1
SCOPE AND DEFINITIONS

19.1.1 Provisions of Chapter 19 shall apply to thin shell and folded plate concrete structures, including ribs and edge members.

19.1.2 All provisions of SBC 304 not specifically excluded, and not in conflict with provisions of Chapter 19, shall apply to thin-shell structures.

Thin shells. Three-dimensional spatial structures made up of one or more curved slabs or folded plates whose thicknesses are small compared to their other dimensions. Thin shells are characterized by their three-dimensional load-carrying behavior, which is determined by the geometry of their forms, by the manner in which they are supported, and by the nature of the applied load.

Folded plates. A special class of shell structure formed by joining flat, thin slabs along their edges to create a three-dimensional spatial structure.

Ribbed shells. Spatial structures with material placed primarily along certain preferred rib lines, with the area between the ribs filled with thin slabs or left open.

Auxiliary members. Ribs or edge beams that serve to strengthen, stiffen, or support the shell; usually, auxiliary members act jointly with the shell.

Elastic analysis. An analysis of deformations and internal forces based on equilibrium, compatibility of strains, and assumed elastic behavior, and representing to a suitable approximation the three-dimensional action of the shell together with its auxiliary members.

Inelastic analysis. An analysis of deformations and internal forces based on equilibrium, non-linear stress-strain relations for concrete and reinforcement, consideration of cracking and time-dependent effects, and compatibility of strains. The analysis shall represent to a suitable approximation three-dimensional action of the shell together with its auxiliary members.

Experimental analysis. An analysis procedure based on the measurement of deformations or strains, or both, of the structure or its model; experimental analysis is based on either elastic or inelastic behavior.

SECTION 19.2
ANALYSIS AND DESIGN

19.2.1 Elastic behavior shall be an accepted basis for determining internal forces and displacements of thin shells. This behavior shall be permitted to be established by computations based on an analysis of the uncracked concrete structure in which the material is assumed linearly elastic, homogeneous, and isotropic. Poisson's ratio of concrete shall be permitted to be taken equal to zero.

19.2.2 Inelastic analyses shall be permitted to be used where it can be shown that such methods provide a safe basis for design.

19.2.3 Equilibrium checks of internal resistances and external loads shall be made to ensure consistency of results.

19.2.4 Experimental or numerical analysis procedures shall be permitted where it can be shown that such procedures provide a safe basis for design.

19.2.5 Approximate methods of analysis shall be permitted where it can be shown that such methods provide a safe basis for design.

19.2.6 In prestressed shells, the analysis shall also consider behavior under loads induced during prestressing, at cracking load, and at factored load. Where tendons are draped within a shell, design shall take into account force components on the shell resulting from the tendon profile not lying in one plane.

19.2.7 The thickness of a shell and its reinforcement shall be proportioned for the required strength and serviceability, using either the strength design method of Section 8.1.1 or the design method of 8.1.2.

19.2.8 Shell instability shall be investigated and shown by design to be precluded.

19.2.9 Auxiliary members shall be designed according to the applicable provisions of the code. It shall be permitted to assume that a portion of the shell equal to the flange width, as specified in 8.10, acts with the auxiliary member. In such portions of the shell, the reinforcement perpendicular to the auxiliary member shall be at least equal to that required for the flange of a T-beam by 8.10.5.

19.2.10 Strength design of shell slabs for membrane and bending forces shall be based on the distribution of stresses and strains as determined from either elastic or an inelastic analysis.

19.2.11 In a region where membrane cracking is predicted, the nominal compressive strength parallel to the cracks shall be taken as $0.4 f_c^{'}$.

SECTION 19.3
DESIGN STRENGTH OF MATERIALS

19.3.1 Specified compressive strength of concrete f_c' at 28 days shall not be less than 20 MPa.

19.3.2 Specified yield strength of nonprestressed reinforcement f_y shall not exceed 420 MPa.

SECTION 19.4
SHELL REINFORCEMENT

19.4.1 Shell reinforcement shall be provided to resist tensile stresses from internal membrane forces, to resist tension from bending and twisting moments, to limit shrinkage and temperature crack width and spacing, and as special reinforcement at shell boundaries, load attachments, and shell openings.

19.4.2 Tensile reinforcement shall be provided in two or more directions and shall be proportioned such that its resistance in any direction equals or exceeds the component of internal forces in that direction.
Alternatively, reinforcement for the membrane forces in the slab shall be calculated as the reinforcement required to resist axial tensile forces plus the tensile force due to shear-friction required to transfer shear across any cross section of the membrane. The assumed coefficient of friction shall not exceed 1.0λ where λ = 1.0 for normal weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for all-lightweight concrete. Linear interpolation shall be permitted when partial sand replacement is used.

19.4.3 The area of shell reinforcement at any section as measured in two orthogonal directions shall not be less than the slab shrinkage or temperature reinforcement required by 7.12.

19.4.4 Reinforcement for shear and bending moments about axes in the plane of the shell slab shall be calculated in accordance with Chapters 10, 11, and 13.

19.4.5 The area of shell tension reinforcement shall be limited so that the reinforcement will yield before either crushing of concrete in compression or shell buckling can take place.

19.4.6 In regions of high tension, membrane reinforcement shall, if practical, be placed in the general directions of the principal tensile membrane forces. Where this is not practical, it shall be permitted to place membrane reinforcement in two or more component directions.

19.4.7 If the direction of reinforcement varies more than 10 deg from the direction of principal tensile membrane force, the amount of reinforcement shall be reviewed in relation to cracking at service loads.

19.4.8 Where the magnitude of the principal tensile membrane stress within the shell varies greatly over the area of the shell surface, reinforcement resisting the total tension shall be permitted to be concentrated in the regions of largest tensile stress where it can be shown that this provides a safe basis for design. However, the ratio of shell reinforcement in any portion of the tensile zone shall be not less than 0.0035 based on the overall thickness of the shell.

19.4.9 Reinforcement required to resist shell bending moments shall be proportioned with due regard to the simultaneous action of membrane axial forces at the same location. Where shell reinforcement is required in only one face to resist bending moments, equal amounts shall be placed near both surfaces of the shell even though a reversal of bending moments is not indicated by the analysis.

19.4.10 Shell reinforcement in any direction shall not be spaced farther apart than 300 mm nor farther apart than three times the shell thickness.

19.4.11 Shell reinforcement at the junction of the shell and supporting members or edge members shall be anchored in or extended through such members in accordance with the requirements of Chapter 12, except that the minimum development length shall be $1.2\ell_d$ but not less than 450 mm.

19.4.12 Splice lengths of shell reinforcement shall be governed by the provisions of Chapter 12, except that the minimum splice length of tension bars shall be 1.2 times the value required by Chapter 12 but not less than 450 mm. The number of splices in principal tensile reinforcement shall be kept to a practical minimum. Where splices are necessary they shall be staggered at least ℓ_d with not more than one-third of the reinforcement spliced at any section.

SECTION 19.4
CONSTRUCTION

19.5.1 When removal of formwork is based on a specific modulus of elasticity of concrete because of stability or deflection considerations, the value of the modulus of elasticity E_c shall be determined from flexural tests of field-cured beam specimens. The number of test specimens, the dimensions of test beam specimens, and test procedures shall be specified by the structural engineer.

19.5.2 The structural engineer shall specify the tolerances for the shape of the shell. If construction results in deviations from the shape greater than the specified tolerances, an analysis of the effect of the deviations shall be made and any required remedial actions shall be taken to ensure safe behavior.

CHAPTER 20
STRENGTH EVALUATION OF EXISTING STRUCTURES

SECTION 20.0
NOTATION

D	=	dead loads or related internal moments and forces
f_c'	=	specified compressive strength of concrete, MPa
h	=	overall thickness of member, mm
L	=	live loads or related internal moments and forces
ℓ_t	=	span of member under load test, mm (The shorter span for two-way slab systems.) Span is the smaller of (a) distance between centers of supports, and (b) clear distance between supports plus thickness h of member. In Eq. (20-1), span for a cantilever shall be taken as twice the distance from support to cantilever end
Δ_{max}	=	measured maximum deflection, mm. See Eq. (20-1)
$\Delta_{r\,max}$	=	measured residual deflection, mm. See Eq.(20-2) and (20-3)
$\Delta_{f\,max}$	=	maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test, mm. See Eq. (20-3)

SECTION 20.1
STRENGTH EVALUATION – GENERAL

20.1.1 If there is doubt that a part or all of a structure meets the safety requirements of SBC 304, a strength evaluation shall be carried out as required by the structural engineer.

20.1.2 If the effect of the strength deficiency is well understood and if it is feasible to measure the dimensions and material properties required for analysis, analytical evaluations of strength based on those measurements shall suffice. Required data shall be determined in accordance with 20.2.

20.1.3 If the effect of the strength deficiency is not well understood or if it is not feasible to establish the required dimensions and material properties by measurement, a load test shall be required if the structure is to remain in service.

20.1.4 If the doubt about safety of a part or all of a structure involves deterioration, and if the observed response during the load test satisfies the acceptance criteria (see 20.5), the structure or part of the structure shall be permitted to remain in service for a specified time period. If deemed necessary by the structural engineer, periodic reevaluations shall be conducted.

SECTION 20.2
DETERMINATION OF REQUIRED DIMENSIONS AND MATERIAL PROPERTIES

20.2.1 Dimensions of the structural elements shall be established at critical sections.

20.2.2 Locations and sizes of the reinforcing bars, welded wire fabric, or tendons shall be determined by measurement. It shall be permitted to base reinforcement locations on available drawings if spot checks are made confirming the information on the drawings.

20.2.3 If required, concrete strength shall be based on results of cylinder tests or tests of cores removed from the part of the structure where the strength is in doubt. Concrete strengths shall be determined as specified in 5.6.5.

20.2.4 If required, reinforcement or prestressing steel strength shall be based on tensile tests of representative samples of the material in the structure in question.

20.2.5 If the required dimensions and material properties are determined through measurements and testing, and if calculations can be made in accordance with 20.1.2, it shall be permitted to increase the strength reduction factor in 9.3, but the strength reduction factor shall not be more than:

Tension-controlled sections, as defined in 10.3.4 ……………1.0

Compression-controlled sections, as defined in 10.3.3

Members with spiral reinforcement conforming to 10.9.3 …..0.85

Other reinforced members … ………………………………...0.8

Shear and/or torsion ……………………………………..……0.8

Bearing on concrete ………………………………………..0.8

SECTION 20.3
LOAD TEST PROCEDURE

20.3.1 **Load arrangement**

The number and arrangement of spans or panels loaded shall be selected to maximize the deflection and stresses in the critical regions of the structural elements of which strength is in doubt. More than one test load arrangement shall be used if a single arrangement will not simultaneously result in maximum values of the effects (such as deflection, rotation, or stress) necessary to demonstrate the adequacy of the structure.

20.3.2 **Load intensity**

The total test load (including dead load already in place) shall not be less than $0.85(1.4D + 1.7L)$. It shall be permitted to reduce L in accordance with the requirements of SBC 301.

20.3.3 A load test shall not be made until that portion of the structure to be subjected to load is at least 56 days old. If the owner of the structure, the contractor, and all involved parties agree, it shall be permitted to make the test at an earlier age.

SECTION 20.4
LOADING CRITERIA

20.4.1 The initial value for all applicable response measurements (such as deflection, rotation, strain, slip, crack widths) shall be obtained not more than 1 hour before application of the first load increment. Measurements shall be made at locations where maximum response is expected. Additional measurements shall be made if required.

20.4.2 Test load shall be applied in not less than four approximately equal increments.

20.4.3 Uniform test load shall be applied in a manner to ensure uniform distribution of the load transmitted to the structure or portion of the structure being tested. Arching of the applied load shall be avoided.

20.4.4 A set of response measurements shall be made after each load increment is applied and after the total load has been applied on the structure for at Least 24 hours.

20.4.5 Total test load shall be removed immediately after all response measurements defined in 20.4.4 are made.

20.4.6 A set of final response measurements shall be made 24 hours after the test load is removed.

SECTION 20.5
ACCEPTANCE CRITERIA

20.5.1 The portion of the structure tested shall show no evidence of failure. Spalling and crushing of compressed concrete shall be considered an indication of failure.

20.5.2 Measured maximum deflections shall satisfy one of the following conditions:

$$\Delta_{\max} \leq \frac{\ell_t^2}{20{,}000h} \tag{20-1}$$

$$\Delta_{r\max} \leq \frac{\Delta_{\max}}{4} \tag{20-2}$$

If the measured maximum and residual deflections do not satisfy Eq. (20-1) or (20-2), it shall be permitted to repeat the load test.
The repeat test shall be conducted not earlier than 72 hours after removal of the first test load. The portion of the structure tested in the repeat test shall be considered acceptable if deflection recovery satisfies the condition:

$$\Delta_{r\max} \leq \frac{\Delta_{f\max}}{5} \tag{20-3}$$

where $\Delta_{f\max}$ is the maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test.

20.5.3 Structural members tested shall not have cracks indicating the imminence of shear failure.

20.5.4 In regions of structural members without transverse reinforcement, appearance of structural cracks inclined to the longitudinal axis and having a horizontal projection longer than the depth of the member at midpoint of the crack shall be evaluated.

20.5.5 In regions of anchorage and lap splices, the appearance along the line of reinforcement of a series of short inclined cracks or horizontal cracks shall be evaluated.

SECTION 20.6
PROVISION FOR LOWER LOAD RATING

20.6.1 If the structure under investigation does not satisfy conditions or criteria of 20.1.2, 20.5.2, or 20.5.3, the structure shall be permitted for use at a lower load rating based on the results of the load test or analysis, if approved by the building official.

SECTION 20.7
SAFETY

20.7.1 Load tests shall be conducted in such a manner as to provide for safety of life and structure during the test.

20.7.2 No safety measures shall interfere with load test procedures or affect results.

CHAPTER 21
SPECIAL PROVISIONS FOR SEISMIC DESIGN

SECTION 21.0
NOTATION

A_{ch} = cross-sectional area of a structural member measured out-to-out of transverse reinforcement, mm²

A_{cp} = area of concrete section, resisting shear, of an individual pier or horizontal wall segment, mm²

A_{cv} = gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, mm²

A_g = gross area of section, mm²

A_j = effective cross-sectional area within a joint, see 21.5.3.1, in a plane parallel to plane of reinforcement generating shear in the joint, mm². The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of:
(a) beam width plus the joint depth
(b) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. See 21.5.3.1

A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension h_c, mm²

A_{vd} = total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam, mm²

b = effective compressive flange width of a structural member, mm

b_w = web width, or diameter of circular section, mm

c = distance from the extreme compression fiber to neutral axis, see 10.2.7, calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth, mm

c_1 = Size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm

c_t = dimension equal to the distance from the interior face of the column to the slab edge measured parallel to c_1, but not exceeding c_1, mm

d = effective depth of section, mm

d_b = bar diameter, mm

E = load effects of earthquake, or related internal moments and forces

$f_c^{'}$ = specified compressive strength of concrete, MPa

$\sqrt{f_c^{'}}$ = square root of specified compressive strength of concrete, MPa

f_y specified yield strength of reinforcement, MPa

f_{yh} = specified yield strength of transverse reinforcement, MPa

h = overall dimension of member in the direction of action considered, mm

h_c = cross-sectional dimension of column core measured center-to-center of confining reinforcement, mm

h_w = height of entire wall or of the segment of wall considered, mm

h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the column, mm

ℓ_d = development length for a straight bar, mm

ℓ_{dh} = development length for a bar with a standard hook as defined in Eq. (21-6), mm

ℓ_n = clear span measured face-to-face of supports, mm

ℓ_o = minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, mm

ℓ_w = length of entire wall or of segment of wall considered in direction of shear force, mm

M_c = moment at the face of the joint, corresponding to the nominal flexural strength of the column framing into that joint, calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength, N–mm. See 21.4.2.2

M_g = moment at the face of the joint, corresponding to the nominal flexural strength of the girder including slab where in tension, framing into that joint, N– mm. See 21.4.2.2

M_{pr} = probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least $1.45 f_y$ and a strength reduction factor ϕ of 1.0, N-mm.

M_s = portion of slab moment balanced by support moment, N-mm

M_u = factored moment at section, N-mm.

S_e = moment, shear, or axial force at connection corresponding with development of probable strength at intended yield locations, based on the governing mechanism of inelastic lateral deformation, considering both gravity and earthquake load effects

S_n = nominal flexural, shear, or axial strength of the connection

S_y = yield strength of connection, based on f_y for moment, shear, or axial force

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, mm

s_o = maximum spacing of transverse reinforcement, mm

s_x = longitudinal spacing of transverse reinforcement within the length ℓ_o, mm

V_c = nominal shear strength provided by concrete, N

V_e = design shear force determined from 21.3.4.1or 21.4.5.1, N

V_n = nominal shear strength, N

V_u = factored shear force at section, N

α = angle between the diagonal reinforcement and the longitudinal axis of a diagonally reinforced coupling beam

α_c = coefficient defining the relative contribution of concrete strength to wall strength. See Eq. (21-7)

δ_u = design displacement, mm

ρ = ratio of nonprestressed tension reinforcement

= A_s / bd

ρ_g = ratio of total reinforcement area to cross-sectional area of column

ρ_n = ratio of area of distributed reinforcement parallel to the plane of A_{cv} to gross concrete area perpendicular to that reinforcement

ρ_S = ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out)

ρ_v = ratio of area of distributed reinforcement perpendicular to the plane of A_{cv} to gross concrete area A_{cv}

ϕ = strength reduction factor

SECTION 21.1
DEFINITIONS

Base of structure. Level at which earthquake motions are assumed to be imparted to a building. This level does not necessarily coincide with the ground level.

Boundary elements. Portions along structural wall and structural diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms shall be provided with boundary elements as required by Section 21.7.6 or 21.9.5.3.

Collector elements. Elements that serve to transmit the inertial forces within structural diaphragms to members of the lateral-force-resisting systems.

Connection. A region that joins two or more members, of which one or more is precast.

Ductile connection. Connection that experiences yielding as a result of the design displacements.

Strong connection. Connection that remains elastic while adjoining members experience yielding as a result of the design displacements.

Crosstie. A continuous reinforcing bar having a seismic hook at one end and a hook not less than 90 deg with at least a six-diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90 deg hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.

Design displacement. Total lateral displacement expected for the design-basis earthquake, as specified by SBC 301.

Design load combinations. Combinations of factored loads and forces in 9.2.

Development length for a bar with a standard hook. The shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent to the outer edge of the 90 deg hook.

Factored loads and forces. Loads and forces multiplied by appropriate load factors in 9.2.

Hoop. A closed tie or continuously wound tie. A closed tie can be made up of several reinforcement elements each having seismic hooks at both ends. A continuously wound tie shall have a seismic hook at both ends.

Joint. Portion of structure common to intersecting members. The effective area of the joint for shear strength computations is defined in 21.0 (See A_j).

Lateral-force resisting system. That portion of the structure composed of members proportioned to resist forces related to earthquake effects.

Lightweight aggregate concrete. All-lightweight or sand-lightweight aggregate concrete made with lightweight aggregates conforming to 3.3.

Moment frame. Frame in which members and joints resist forces through flexure, shear, and axial force. Moment frames shall be categorized as follows:

Intermediate moment frame. A cast-in-place frame complying with the requirements of Section 21.2.2.3 and 21.12 in addition to the requirements for ordinary moment frames.

Ordinary moment frame. A cast-in-place or precast concrete frame complying with the requirements of Chapters 1 through 18.

Special moment frame. A cast-in-place frame complying with the requirements of Section 21.2 through 21.5 or a precast frame complying with the requirements of Section 21.2 through 21.6. In addition, the requirements for ordinary moment frames shall be satisfied.

Plastic hinge region. Length of frame element over which flexural yielding is intended to occur due to design displacements, extending not less than a distance h from the critical section where flexural yielding initiates.

Seismic hook. A hook on a stirrup, hoop, or crosstie having a bend not less than 135 deg, except that circular hoops shall have a bend not less than 90 deg. Hooks shall have a six-diameter (but not less than 75 mm) extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

Special boundary elements. Boundary elements required by Section 21.7.6.2 or 21.7.6.3.

Specified lateral forces. Lateral forces corresponding to the appropriate distribution of the design base shear force prescribed by the SBC 301 provisions for earthquake-resistant design.

Structural diaphragms. Structural members, such as floor and roof slabs, that transmit inertial forces to lateral-force resisting members.

Structural trusses. Assemblages of reinforced concrete members subjected primarily to axial forces.

Structural walls. Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shearwall is a structural wall. Structural walls shall be categorized as follows:

Intermediate precast structural wall. A wall complying with all applicable requirements of Chapters 1 through 18 in addition to Section 21.13.

Ordinary reinforced concrete structural wall. A wall complying with the requirements of Chapters 1 through 18.

Special precast structural wall. A precast wall complying with the requirements of 21.8. In addition, the requirements for ordinary reinforced concrete structural walls and the requirements of Section 21.2 shall be satisfied.

Special reinforced concrete structural wall. A cast-in-place wall complying with the requirements of Section 21.2 and 21.7 in addition to the requirements for ordinary reinforced concrete structural walls.

Story drift ratio. The design displacement over a story divided by the story height.

Strut. An element of a structural diaphragm used to provide continuity around an opening in the diaphragm.

Tie elements. Elements that serve to transmit inertia forces and prevent separation of building components such as footings and walls.

Wall pier. A wall segment with a horizontal length-to-thickness ratio of at least 2.5, but not exceeding six, whose clear height is at least two times its horizontal length.

SECTION 21.2
GENERAL REQUIREMENTS

21.2.1 Scope

21.2.1.1 Chapter 21 contains special requirements for design and construction of reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response. For applicable specified concrete strengths, see 1.1.1 and 21.2.4.1

21.2.1.2 For structures assigned to Seismic Design Category A or B, provisions of Chapters 1 through 18 and 22 shall apply except as modified by the provisions of this chapter. Where the seismic design loads are computed using provisions for intermediate or special concrete systems, the requirements of Chapter 21 for intermediate or special systems, as applicable, shall be satisfied.

21.2.1.3 For structures assigned to Seismic Design Category C, intermediate or special moment frames, or ordinary or special reinforced concrete structural walls shall be used to resist seismic forces induced by earthquake motions. Where the design seismic loads are computed using provisions for special concrete systems, the requirements of Chapter 21 for special systems, as applicable, shall be satisfied.

21.2.1.4 For structures assigned to Seismic Design Category D, special moment frames, special reinforced concrete structural walls, diaphragms and trusses and foundations complying with Sections 21.2 through 21.10 shall be used to resist forces induced by earthquake motions. Frame members not proportioned to resist earthquake forces shall comply with 21.11.

21.2.1.5 A reinforced concrete structural system not satisfying the requirements of this chapter shall be permitted if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this chapter.

21.2.2 Analysis and proportioning of structural members

21.2.2.1 The interaction of all structural and non-structural members that materially affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

21.2.2.2 Rigid members assumed not to be a part of the lateral-force resisting system shall be permitted provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of failure of structural and non-structural members, which are not a part of the lateral-force resisting system, shall also be considered.

21.2.2.3 Structural members below base of structure that are required to transmit to the foundation forces resulting from earthquake effects shall also comply with the requirements of Chapter 21.

21.2.2.4 All structural members assumed not to be part of the lateral-force resisting system shall conform to 21.11.

21.2.3 Strength reduction factors. Strength reduction factors shall be as given in Section 9.3.4.

21.2.4 Concrete in members resisting earthquake-induced forces

21.2.4.1 Compressive strength $f_c^{'}$ of the concrete shall be not less than 20 MPa.

21.2.4.2 Compressive strength of lightweight aggregate concrete used in design shall not exceed 35 MPa. Lightweight aggregate concrete with higher design compressive strength shall be permitted if demonstrated by experimental evidence that structural members made with that lightweight aggregate concrete provide strength and toughness equal to or exceeding those of comparable members made with normalweight aggregate concrete of the same strength.

21.2.5 Reinforcement in members resisting earthquake-induced forces
Reinforcement resisting earthquake-induced flexural and axial forces in frame members and in structural wall boundary elements shall comply with ASTM A 706M. ASTM A 615M Grades 300 and 420 reinforcement shall be permitted in these members if:

(a) The actual yield strength based on mill tests does not exceed the specified yield strength by more than 120 MPa (retests shall not exceed this value by more than an additional 20 MPa); and

(b) The ratio of the actual ultimate tensile strength to the actual tensile yield strength is not less than 1.25.

21.2.6 Mechanical splices

21.2.6.1 Mechanical splices shall be classified as either Type 1 or Type 2 mechanical splices, as follows:

(a) Type 1 mechanical splices shall conform to Section 12.14.3.2;

(b) Type 2 mechanical splices shall conform to Section 12.14.3.2 and shall develop the specified tensile strength of the spliced bar.

21.2.6.2 Type 1 mechanical splices shall not be used within a distance equal to twice the member depth from the column or beam face or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements. Type 2 mechanical splices shall be permitted to be used at any location.

21.2.7 Welded splices

21.2.7.1 Welded splices in reinforcement resisting earthquake-induced forces shall conform to Section 12.14.3.4 and 3.5.2 and shall not be used within a distance equal to twice the member depth from the column or beam face or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements.

21.2.7.2 Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement that is required by design shall not be permitted.

21.2.8 Anchoring to concrete

21.2.8.1 Anchors resisting earthquake-induced forces in structures in regions of moderate or high seismic risk, or assigned to intermediate or high seismic performance or design categories shall conform to the additional requirements of D.3.3 of Appendix D.

SECTION 21.3
FLEXURAL MEMBERS OF SPECIAL MOMENT FRAMES

21.3.1 **Scope**. Requirements of 21.3 apply to special moment frame members (a) resisting earthquake-induced forces and (b) proportioned primarily to resist flexure. These frame members shall also satisfy the conditions of Section 21.3.1.1 through 21.3.1.4.

21.3.1.1 Factored axial compressive force on the member shall not exceed $\left(A_g f_c^{'} / 10\right)$.

21.3.1.2 Clear span for the member shall not be less than four times its effective depth.

21.3.1.3 The width-to-depth ratio shall not be less than 0.3.

21.3.1.4 The width shall not be

(a) less than 250 mm; and

(b) more than the width of the supporting member (measured on a plane perpendicular to the longitudinal axis of the flexural member) plus distances on each side of the supporting member not exceeding three-fourths of the depth of the flexural member.

21.3.2 Longitudinal reinforcement

21.3.2.1 At any section of a flexural member, except as provided in 10.5.3, for top as well as for bottom reinforcement, the amount of reinforcement shall not be less than that given by Eq. (10-3) but not less than $1.4 b_w d / f_y$ and the reinforcement ratio ρ shall not exceed 0.020. At least two bars shall be provided continuously both top and bottom.

21.3.2.2 Positive moment strength at joint face shall be not less than one-half of the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along member length shall be less than one-fourth the maximum moment strength provided at face of either joint.

21.3.2.3 Lap splices of flexural reinforcement shall be permitted only if hoop or spiral reinforcement is provided over the lap length. Maximum spacing of the transverse reinforcement enclosing the lapped bars shall not exceed $d/4$ or 100 mm. Lap splices shall not be used

- **(a)** within the joints;
- **(b)** within a distance of twice the member depth from the face of the joint; and
- **(c)** at locations where analysis indicates flexural yielding caused by inelastic lateral displacements of the frame.

21.3.2.4 Mechanical splices shall conform to 21.2.6 and welded splices shall conform to 21.2.7.

21.3.3 Transverse reinforcement

21.3.3.1 Hoops shall be provided in the following regions of frame members:

- **(a)** Over a length equal to twice the member depth measured from the face of the supporting member toward midspan, at both ends of the flexural member;
- **(b)** Over lengths equal to twice the member depth on both sides of a section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame.

21.3.3.2 The first hoop shall be located not more than 50 mm from the face of a supporting member. Maximum spacing of the hoops shall not exceed (a), (b), (c) and (d):

- **(a)** $d/4$;
- **(b)** eight times the diameter of the smallest longitudinal bars;
- **(c)** 24 times the diameter of the hoop bars; and
- **(d)** 250 mm

21.3.3.3 Where hoops are required, longitudinal bars on the perimeter shall have lateral support conforming to 7.10.5.3.

21.3.3.4 Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than $d/2$ throughout the length of the member.

21.3.3.5 Stirrups or ties required to resist shear shall be hoops over lengths of members in 21.3.3, 21.4.4, and 21.5.2.

21.3.3.6 Hoops in flexural members shall be permitted to be made up of two pieces of reinforcement: a stirrup having seismic hooks at both ends and closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90 deg hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the crossties are confined by a slab on only one side of the flexural frame member, the 90 deg hooks of the crossties shall be placed on that side.

21.3.4 **Shear strength requirements**

21.3.4.1 **Design forces**

The design shear force V_e shall be determined from consideration of the statical forces on the portion of the member between faces of the joints. It shall be assumed that moments of opposite sign corresponding to probable flexural moment strength M_{pr} act at the joint faces and that the member is loaded with the factored tributary gravity load along its span.

21.3.4.2 **Transverse reinforcement**

Transverse reinforcement over the lengths identified in 21.3.3.1 shall be proportioned to resist shear assuming $V_c = 0$ when both of the following conditions occur:

(a) The earthquake-induced shear force calculated in accordance with 21.3.4.1 represents one-half or more of the maximum required shear strength within those lengths;

(b) The factored axial compressive force including earthquake effects is less than $A_g f_c^{'} / 20$.

SECTION 21.4
SPECIAL MOMENT FRAME MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

21.4.1 **Scope**. The requirements of this section apply to special moment frame members (a) resisting earthquake-induced forces and (b) having a factored axial force exceeding $\left(A_g f_c^{'} / 10\right)$. These frame members shall also satisfy the conditions of Section 21.4.1.1 and 21.4.1.2.

21.4.1.1 The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 300 mm.

21.4.1.2 The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

21.4.2 **Minimum flexural strength of columns**

21.4.2.1 Flexural strength of any column proportioned to resist a factored axial compressive force exceeding $\left(A_g f_c^{'} / 10\right)$ shall satisfy Section 21.4.2.2 or 21.4.2.3.

Lateral strength and stiffness of columns not satisfying 21.4.2.2 shall be ignored in determining the calculated strength and stiffness of the structure, but such columns shall conform to 21.11.

21.4.2.2 The flexural strengths of the columns shall satisfy Eq. (21-1)

$$\sum M_c \geq (6/5)\sum M_g \tag{21-1}$$

$\sum M_c =$ sum of moments at the faces of the joint corresponding to the nominal flexural strength of the columns framing into that joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.

$\sum M_g =$ sum of moments at the faces of the joint corresponding to the nominal flexural strengths of the girders framing into that joint. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width defined in 8.10 shall be assumed to contribute to flexural strength if the slab reinforcement is developed at the critical section for flexure.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Eq. (21-1) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

21.4.2.3 If 21.4.2.2 is not satisfied at a joint, columns supporting reactions from that joint shall be provided with transverse reinforcement as specified in Section 21.4.4.1 through 21.4.4.3 over their full height.

21.4.3 Longitudinal reinforcement

21.4.3.1 The reinforcement ratio ρ_g shall not be less than 0.01 and shall not exceed 0.06.

21.4.3.2 Mechanical splices shall conform to Section 21.2.6 and welded splices shall conform to 21.2.7. Lap splices shall be permitted only within the center half of the member length, shall be designed as tension lap splices, and shall be enclosed within transverse reinforcement conforming to Section 21.4.4.2 and 21.4.4.3.

21.4.4 Transverse reinforcement

21.4.4.1 Transverse reinforcement as required below in (a) through (e) shall be provided unless a larger amount is required by Section 21.4.3.2 or 21.4.5.

(a) The volumetric ratio of spiral or circular hoop reinforcement ρ_S shall not be less than that required by Eq. (21-2).

$$\rho_s = 0.12 f_c' / f_{yh} \tag{21-2}$$

and shall not be less than that required by Eq. (10-5).

(b) The total cross-sectional area of rectangular hoop reinforcement shall not be less than that required by Eq. (21-3) and (21-4).

$$A_{sh} = 0.3\left(s h_c f_c' / f_{yh}\right)\left(\left(A_g / A_{ch}\right) - 1\right) \tag{21-3}$$

$$A_{sh} = 0.09\, s h_c f_c' / f_{yh} \tag{21-4}$$

(c) Transverse reinforcement shall be provided by either single or overlapping hoops. Crossties of the same bar size and spacing as the hoops shall be permitted. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement.

(d) If the design strength of member core satisfies the requirement of the design loading combinations including earthquake effect, Eq. (21-3) and (10-5) need not be satisfied.

(e) If the thickness of the concrete outside the confining transverse reinforcement exceeds 100 mm, additional transverse reinforcement shall be provided at a spacing not exceeding 300 mm. Concrete cover on the additional reinforcement shall not exceed 100 mm.

21.4.4.2 Transverse reinforcement shall be spaced at a distance not exceeding (a) one-quarter of the minimum member dimension, (b) six times the diameter of the longitudinal reinforcement, and (c) s_x as defined by Eq. (21-5).

$$s_x = 100 + \left(\frac{350 - h_x}{3}\right) \tag{21-5}$$

The value of s_x shall not exceed 150 mm and need not be taken less than 100 mm.

21.4.4.3 Crossties or legs of overlapping hoops shall not be spaced more than 350 mm on center in the direction perpendicular to the longitudinal axis of the structural member.

21.4.4.4 Transverse reinforcement in amount specified in 21.4.4.1 through 21.4.4.3 shall be provided over a length ℓ_o from each joint face and on both sides of any section where flexural yielding is likely to occur as a result of inelastic lateral displacements of the frame. The length ℓ_o shall not be less than (a), (b), and (c):

(a) the depth of the member at the joint face or at the section where flexural yielding is likely to occur;

(b) one-sixth of the clear span of the member; and

(c) 500 mm.

21.4.4.5 Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 over their full height beneath the level at which the discontinuity occurs if the factored axial compressive force in these members, related to earthquake effect, exceeds $\left(A_g f_c^{'} / 10\right)$. Transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 shall extend into the discontinued member for at least the development length of the largest longitudinal reinforcement in the column in accordance with 21.5.4. If the lower end of the column terminates on a wall, transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 shall extend into the wall for at least the development length of the largest longitudinal bar in the column at the point of termination. If the column terminates on a footing or mat, transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 shall extend at least 300 mm into the footing or mat.

21.4.4.6 Where transverse reinforcement, as specified in Section 21.4.4.1 through 21.4.4.3, is not provided throughout the full length of the column, the remainder of the column length shall contain spiral or hoop reinforcement with center-to-center spacing not exceeding the smaller of six times the diameter of the longitudinal column bars or 150 mm.

21.4.5 **Shear strength requirements**

21.4.5.1 **Design forces**. The design shear force V_e shall be determined from consideration of the maximum forces that can be generated at the faces of the joints at each end of the member. These joint forces shall be determined using the maximum probable moment strengths M_{pr} of the member associated with the range of factored axial loads on the member. The member shears need not exceed those determined from joint strengths based on the probable moment strength M_{pr} of the transverse members framing into the joint. In no case shall V_e be less than the factored shear determined by analysis of the structure.

21.4.5.2 Transverse reinforcement over the lengths ℓ_o identified in 21.4.4.4, shall be proportioned to resist shear assuming $V_c = 0$ when both the following conditions occur:

(a) The earthquake-induced shear force, calculated in accordance with 21.4.5.1, represents one-half or more of the maximum required shear strength within those lengths;

(b) The factored axial compressive force including earthquake effects is less than $A_g f_c^{'} / 20$.

SECTION 21.5
JOINTS OF SPECIAL MOMENT FRAMES

21.5.1 **General requirements**

21.5.1.1 Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural tensile reinforcement is $1.45 f_y$.

21.5.1.2 Strength of joint shall be governed by the appropriate strength reduction factors in 9.3.

21.5.1.3 Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension according to 21.5.4 and in compression according to Chapter 12.

21-5.1.4 Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall not be less than 25 times the diameter of the largest longitudinal bar for normal weight concrete. For lightweight concrete, the dimension shall be not less than 30 times the bar diameter.

21.5.2 **Transverse reinforcement**

21.5.2.1 Transverse hoop reinforcement in 21.4.4 shall be provided within the joint, unless the joint is confined by structural members in 21.5.2.2.

21.5.2.2 Within the depth of the shallowest framing member, transverse reinforcement equal to at least one-half the amount required by 21.4.4.1 shall be provided where members frame into all four sides of the joint and where each member width is at least three-fourths the column width. At these locations, the spacing required in 21.4.4.2 shall be permitted to be increased to 150 mm.

21.5.2.3 Transverse reinforcement as required by 21.4.4 shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement is not provided by a beam framing into the joint.

21.5. 3 Shear strength

21.5.3.1 The nominal shear strength of the joint shall not be taken as greater than the values specified below for normalweight aggregate concrete.

For joints confined on all four faces………………………………..…… $1.7\sqrt{f_c'}A_j$

For joints confined on three faces or on two opposite faces ………... $1.25\sqrt{f_c'}A_j$

For others ………………………………………………………………... $1.0\sqrt{f_c'}A_j$

A member that frames into a face is considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. A joint is considered to be confined if such confining members frame into all faces of the joint.

215.3.2 For lightweight aggregate concrete, the nominal shear strength of the joint shall not exceed three-quarters of the limits given in Section 21.5.3.1.

21.5.4 Development length of bars in tension

21.5.4.1 The development length ℓ_{dh} for a bar with a standard 90 deg hook in normalweight aggregate concrete shall not be less than the largest of $8d_b$, 150 mm, and the length required by Eq. (21-6).

$$\ell_{dh} = f_y d_b / \left(4.5\sqrt{f_c'}\right) \qquad (21\text{-}6)$$

for bar sizes Dia 10 mm through Dia 36 mm.

For lightweight aggregate concrete, the development length for a bar with a standard 90 deg hook shall not be less than the largest of $10d_b$, 200 mm, and 1.25 times that required by Eq. (21-6).

The 90 deg hook shall be located within the confined core of a column or of a boundary element.

21.5.4.2 For bar sizes Dia 10 mm through Dia 36 mm, the development length for ℓ_d a straight bar shall not be less than (a) and (b):

(a) 2.5 times the length required by 21.5.4.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 300 mm; and

(b) 3.5 times the length required by 21.5.4.1 if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm.

21.5.4.3 Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary element. Any portion of the straight embedment length not within the confined core shall be increased by a factor of 1.6.

21.5.4.4 If epoxy-coated reinforcement is used, the development lengths in 21.5.4.1 through 21.5.4.3 shall be multiplied by the applicable factor in 12.2.4 or 12.5.2.

SECTION 21.6
SPECIAL MOMENT FRAMES CONSTRUCTED USING PRECAST CONCRETE

21.6.1 Special moment frames with ductile connections constructed using precast concrete shall satisfy the requirements of (a) and (b) and all requirements for special moment frames constructed with cast-in-place concrete:

(a) The nominal shear strength for connections, V_n computed according to Section 11.7.4 shall be greater than or equal to $2V_e$, where V_e is calculated according to Section 21.3.4.1 or 21.4.5.1; and

(b) Mechanical splices of beam reinforcement shall be located not closer than $h/2$ from the joint face and shall meet the requirements of Section 21.2.6.

21.6.2 Special moment frames with strong connections constructed using precast concrete shall satisfy all requirements for special moment frames constructed with cast-in-place concrete, as well as the requirements of (a), (b), (c), and (d).

(a) Provisions of Section 21.3.1.2 shall apply to segments between locations where flexural yielding is intended to occur due to design displacements;

(b) Design strength of the strong connection ϕS_n shall be not less than S_e;

(c) Primary longitudinal reinforcement shall be made continuous across connections and shall be developed outside both the strong connection and the plastic hinge region; and

(d) Column-to-column connections shall have design strength ϕS_n not less than $1.4S_e$. At column-to-column connections, the design flexural strength ϕM_n shall be not less than 0.4 times the maximum probable flexural strength M_{pr} for the column within the story height, and the design shear strength ϕV_n of the connection shall be not less than that determined by Section 21.4.5.1.

21.6.3 Special moment frames constructed using precast concrete and not satisfying the requirements of Section 21.6.1 or 21.6.2 shall satisfy the requirements of ACI T1.1, "Acceptance Criteria for Moment Frames Based on Structural Testing," and the requirements of (a) and (b):

(a) Details and materials used in the test specimens shall be representative of those used in the structure; and

(b) The design procedure used to proportion the test specimens shall define the mechanism by which the frame resists gravity and earthquake effects, and shall establish acceptance values for sustaining that mechanism. Portions of the mechanism that deviate from code requirements shall be contained in the test specimens and shall be tested to determine upper bounds for acceptance values.

SECTION 21.7 SPECIAL REINFORCED CONCRETE STRUCTURAL WALLS AND COUPLING BEAMS

21.7.1 **Scope**. The requirements of this section apply to special reinforced concrete structural walls and coupling beams serving as part of the earthquake force-resisting system.

21.7.2 **Reinforcement**

21.7.2.1 The distributed web reinforcement ratios, ρ_v and ρ_n for structural walls shall not be less than 0.0025, except if the design shear force does not exceed $(1/12)A_{cv}\sqrt{f_c^{'}}$, the minimum reinforcement for structural walls shall be permitted to be reduced to that required in 14.3. Reinforcement spacing each way in structural walls shall not exceed 300 mm. Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane.

21.7.2.2 At least two curtains of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $(1/6)A_{cv}\sqrt{f_c^{'}}$.

21.7.2.3 All continuous reinforcement in structural walls shall be anchored or spliced in accordance with the provisions for reinforcement in tension in 21.5.4.

21.7.3 **Design forces**

The design shear force V_u shall be obtained from the lateral load analysis in accordance with the factored load combinations.

21.7.4 **Shear strength**

21.7.4.1 Nominal shear strength V_n of structural walls shall not exceed

$$V_n = A_{cv}\left(\alpha_c\sqrt{f_c^{'}} + \rho_n f_y\right) \qquad (21\text{-}7)$$

where the coefficient α_c is 1/4 for $h_w/\ell_w \le 1.5$, is 1/6 for $h_w/\ell_w \ge 2.0$, and varies linearly between 1/4 and 1/6 for h_w/ℓ_w between 1.5 and 2.0.

21.7.4.2 In 21.7.4.1, the value of ratio h_w/ℓ_w used for determining V_n for segments of a wall shall be the larger of the ratios for the entire wall and the segment of wall considered.

21.7.4.3 Walls shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. If the ratio h_w/ℓ_w does not exceed 2.0, reinforcement ratio ρ_v shall not be less than reinforcement ratio ρ_n.

21.7.4.4 Nominal shear strength of all wall piers sharing a common lateral force shall not be assumed to exceed $(2/3)A_{cv}\sqrt{f_c^{'}}$ where A_{cv} is the total crosssectional area, and the nominal shear strength of any one of the individual wall piers shall not be assumed to exceed $(5/6)A_{cp}\sqrt{f_c'}$ where A_{cp} is the cross-sectional area of the pier considered.

21.7.4.5 Nominal shear strength of horizontal wall segments and coupling beams shall be assumed not to exceed $(5/6)A_{cp}\sqrt{f_c'}$ where A_{cp} is the cross-sectional area of a horizontal wall segment or coupling beam.

21.7.5 Design for flexure and axial loads

21.7.5.1 Structural walls and portions of such walls subject to combined flexural and axial loads shall be designed in accordance with 10.2 and 10.3 except that 10.3.6 and the nonlinear strain requirements of 10.2.2 shall not apply. Concrete and developed longitudinal reinforcement within effective flange widths, boundary elements, and the wall web shall be considered effective. The effects of openings shall be considered.

21.7.5.2 Unless a more detailed analysis is performed, effective flange widths of flanged sections shall extend from the face of the web a distance equal to the smaller of one-half the distance to an adjacent wall web and 25 percent of the total wall height.

21.7.6 Boundary elements of special reinforced concrete structural walls

21.7.6.1 The need for special boundary elements at the edges of structural walls shall be evaluated in accordance with 21.7.6.2 or 21.7.6.3. The requirements of 21.7.6.4 and 21.7.6.5 also shall be satisfied.

21.7.6.2 This section applies to walls or wall piers that are effectively continuous from the base of structure to top of wall and designed to have a single critical section for flexure and axial loads. Walls not satisfying these requirements shall be designed by 21.7.6.3.

(a) Compression zones shall be reinforced with special boundary elements where:

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)} \qquad \text{(21-8)}$$

The quantity δ_u / h_w in Eq. (21.8) shall not be taken less than 0.007.

(b) Where special boundary elements are required by 21.7.6.2(a), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of ℓ_w or $M_u/4V_u$.

21.7.6.3 Structural walls not designed to the provisions of 21.7.6.2 shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to factored forces including earthquake effect, exceeds $0.2f_c'$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f_c'$ Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in 21.7.5.2 shall be used.

21.7.6.4 Where special boundary elements are required by 21.7.6.2 or 21.7.6.3, (a) through (f) shall be satisfied:

(a) The boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of $(c - 0.1\ell_w)$ and $c/2$;

(b) In flanged sections, the boundary element shall include the effective flange width in compression and shall extend at least 300 mm into the web;

(c) Special boundary element transverse reinforcement shall satisfy the requirements of 21.4.4.1 through 21.4.4.3, except Eq. (21-3) need not be satisfied;

(d) Special boundary element transverse reinforcement at the wall base shall extend into the support at least the development length of the largest longitudinal reinforcement in the special boundary element unless the special boundary element terminates on a footing or mat, where special boundary element transverse reinforcement shall extend at least 300 mm into the footing or mat;

(e) Horizontal reinforcement in the wall web shall be anchored to develop the specified yield strength f_y within the confined core of the boundary element;

(f) Mechanical splices of longitudinal reinforcement of boundary elements shall conform to 21.2.6. Welded splices of longitudinal reinforcement of boundary elements shall conform to 21.2.7.

21.7.6.5 Where special boundary elements are not required by 21.7.6.2 or 21.7.6.3, (a) and (b) shall be satisfied:

(a) If the longitudinal reinforcement ratio at the wall boundary is greater than $2.8/f_y$ boundary transverse reinforcement shall satisfy 21.4.4.1(c), 21.4.4.3, and 21.7.6.4(a). The maximum longitudinal spacing of transverse reinforcement in the boundary shall not exceed 200 mm;

(b) Except when V_u in the plane of the wall is less than $(1/12)A_{cv}\sqrt{f_c'}$, horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

21.7.6.6 Mechanical and welded splices of longitudinal reinforcement of boundary elements shall conform to 21.2.6 and 21.2.7.

21.7.7 Coupling beams

21.7.7.1 Coupling beams with aspect ratio $\ell_n / h \geq 4$, shall satisfy the requirements of 21.3. The provisions of 21.3.1.3 and 21.3.1.4(a) shall not be required if it can be shown by analysis that the beam has adequate lateral stability.

21.7.7.2 Coupling beams with aspect ratio, $\ell_n / h < 4$, shall be permitted to be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan.

21.7.7.3 Coupling beams with aspect ratio, $\ell_n / h < 2$ and with factored shear force V_u exceeding $(1/3)\sqrt{f_c^{'}} A_{cp}$ shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load carrying capacity of the structure, or the egress from the structure, or the integrity of nonstructural components and their connections to the structure.

21.7.7.4 Coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan shall satisfy the following:

(a) Each group of diagonally placed bars shall consist of a minimum of four bars assembled in a core having sides measured to the outside of transverse reinforcement no smaller than $b_w / 2$ perpendicular to the plane of the beam and $b_w / 5$ in the plane of the beam and perpendicular to the diagonal bars;

(b) The nominal shear strength, V_n shall be determined by

$$V_n = 2 A_{vd} f_y \sin\alpha \le (5/6)\sqrt{f_c^{'}} A_{cp} \qquad (21\text{-}9)$$

(c) Each group of diagonally placed bars shall be enclosed in transverse reinforcement satisy-ing 21.4.4.1 through 21.4.4.3. For the purpose of computing A_g for use in Eq. (10-5) and (21-3), the minimum concrete cover as required in 7.7 shall be assumed on all four sides of each group of diagonally placed reinforcing bars;

(d) The diagonally placed bars shall be developed for tension in the wall;

(e) The diagonally placed bars shall be considered to contribute to nominal flexural strength of the coupling beam;

(f) Reinforcement parallel and transverse to the longitudinal axis shall be provided and, as a minimum, shall conform to Section 11.8.4 and 11.8.5.

21.7.8 Construction joints

All construction joints in structural walls shall conform to 6.4 and contact surfaces shall be roughened as in 11.7.9.

21.7.9 Discontinuous walls

Columns supporting discontinuous structural walls shall be reinforced in accordance with 21.4.4.5.

21.7.10 Wall piers and wall segments

21.7.10.1 Wall piers not designed as a part of a special moment frame shall have transverse reinforcement de-signed to satisfy the requirements in Section 21.7.10.2.

Exceptions:

a. Wall piers that satisfy Section 21.11.

b. Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers, and such segments have a total stiffness of at least six times the sum of the stiffness of all the wall piers.

21.7.10.2 Transverse reinforcement shall be designed to resist the shear forces determined from Sections 21.3.4.2 and 21.4.5.1.Where the axial compressive force, including earthquake effects, is less than $A_g f_c^{'} / 20$, transverse reinforcement in wall piers is permitted to have standard hooks at each end in lieu of hoops. Spacing of transverse reinforcement shall not exceed 150 mm. Transverse reinforcement shall be extended beyond the pier clear height for at least the development length of the largest longitudinal reinforcement in the wall pier.

21.7.10.3 Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

SECTION 21.8
SPECIAL STRUCTURAL WALLS CONSTRUCTED USING PRECAST CONCRETE

21.8.1 Special structural walls constructed using precast concrete shall satisfy all requirements of 21.7 for cast-in-place special structural walls in addition to 21.13.2 and 21.13.3.

SECTION 21.9
STRUCTURAL DIAPHRAGMS AND TRUSSES

21.9.1 **Scope.** Floor and roof slabs acting as structural diaphragms to transmit design actions induced by earthquake ground motions shall be designed in accordance with this section. This section also applies to struts, ties, chords, and collector elements that transmit forces induced by earthquakes, as well as trusses serving as parts of the earthquake force-resisting systems.

21.9.2 **Cast-in-place composite-topping slab diaphragms.** A composite-topping slab cast in place on a precast floor or roof shall be permitted to be used as a structural diaphragm provided the topping slab is reinforced and its connections are proportioned and detailed to provide for a complete transfer of forces to chords, collector elements, and the lateral-force-resisting system. The surface of the previously hardened concrete on which the topping slab is placed shall be clean, free of laitance, and intentionally roughened.

21.9.3 **Cast-in-place topping slab diaphragms.** A cast-in-place noncomposite topping on a precast floor or roof shall be permitted to serve as a structural diaphragm, provided the cast-in-place topping acting alone is proportioned and detailed to resist the design forces.

21.9.4 **Minimum thickness of diaphragms.** Concrete slabs and composite topping slabs serving as structural diaphragms used to transmit earthquake forces shall not be less than 50 mm thick. Topping slabs placed over precast floor or roof elements, acting as structural diaphragms and not relying on composite action with the precast elements to resist the design seismic forces, shall have thickness not less than 65 mm.

21.9.5 Reinforcement

21.9.5.1 The minimum reinforcement ratio for structural diaphragms shall be in conformance with 7.12. Reinforcement spacing each way in nonposttensioned floor or roof systems shall not exceed 250 mm. Where welded wire fabric is used as the distributed reinforcement to resist shear in topping slabs placed over precast floor and roof elements, the wires parallel to the span of the precast elements shall be spaced not less than 250 mm on center. Reinforcement provided for shear strength shall be continuous and shall be distributed uniformly across the shear plane.

21.9.5.2 Bonded tendons used as primary reinforcement in diaphragm chords or collectors shall be proportioned such that the stress due to design seismic forces does not exceed 420 MPa. Precompression from unbonded tendons shall be permitted to resist diaphragm design forces if a complete load path is provided.

21.9.5.3 Structural truss elements, struts, ties, diaphragm chords, and collector elements with compressive stresses exceeding $0.2f_c^{'}$ at any section shall have transverse reinforcement, as given in 21.4.4.1 through 21.4.4.3, over the length of the element. The special transverse reinforcement is allowed to be discontinued at a section where the calculated compressive strength is less than $0.15f_c^{'}$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross-section properties of the elements considered.

21.9.5.4 All continuous reinforcement in diaphragms, trusses, struts, ties, chords, and collector elements shall be anchored or spliced in accordance with the provisions for reinforcement tension as specified in 21.5.4.

21.9.5.5 Type 2 splices are required where mechanical splices are used to transfer forces between the diaphragm and the vertical components of the lateral-force-resisting system.

21.9.6 Design forces

The seismic design forces for structural diaphragms shall be obtained from the lateral load analysis in accordance with the design load combinations.

21.9.7 Shear strength

21.9.7.1 Nominal shear strength V_n of structural diaphragms shall not exceed

$$V_n = A_{cv}\left(\frac{\sqrt{f_c^{'}}}{6} + \rho_n f_y\right) \qquad (21\text{-}10)$$

21.9.7.2 Nominal shear strength V_n of cast-in-place composite-topping slab diaphragms and cast-in-place noncomposite topping slab diaphragms on a precast floor or roof shall not exceed the shear force

$$V_n = A_{cv}\rho_n f_y \qquad (21\text{-}11)$$

where A_{cv} is based on the thickness of the topping slab. The required web reinforcement shall be distributed uniformly in both directions.

21.9.7.3 Nominal shear strength shall not exceed $(2/3)A_{cv}\sqrt{f_c^{'}}$ where A_{cv} is the gross cross-sectional area of the diaphragm.

21.9.8 **Boundary elements of structural diaphragms**

21.9.8.1 Boundary elements of structural diaphragms shall be proportioned to resist the sum of the factored axial forces acting in the plane of the diaphragm and the force obtained from dividing the factored moment at the section by the distance between the boundary elements of the diaphragm at that section.

21.9.8.2 Splices of tensile reinforcement in the chords and collector elements of diaphragms shall develop the yield strength of the reinforcement. Mechanical and welded splices shall conform to Section 21.2.6 and 21.2.7, respectively.

21.9.8.3 Reinforcement for chords and collectors at splices and anchorage zones shall have either:

(a) A minimum center-to-center spacing of three longitudinal bar diameters, but not less than 40 mm, and a minimum concrete clear cover of two and one-half longitudinal bar diameters, but not less than 50 mm; or

(b) Transverse reinforcement as required by Section 11.5.5.3, except as required in 21.9.5.3.

21.9.9 **Construction joints**

All construction joints in diaphragms shall conform to 6.4 and contact surfaces shall be roughened as in 11.7.9.

SECTION 21.10
FOUNDATIONS

21.10.1 **Scope**

21.10.1.1 Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between a structure and the ground shall comply with the requirements of 21.10 and other applicable provisions of SBC 304.

21.10.1.2 The provisions in this section for piles, drilled piers, caissons, and slabs on grade shall supplement other SBC requirements design and construction criteria. See 1.1.5 and 1.1.6.

21.10.2 **Footings, foundation mats, and pile caps**

21.10.2.1 Longitudinal reinforcement of columns and structural walls resisting forces induced by earthquake effects shall extend into the footing, mat, or pile cap, and shall be fully developed for tension at the interface.

21.10.2.2 Columns designed assuming fixed-end conditions at the foundation shall comply with 21.10.2.1 and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90 deg hooks near the bottom of the foundation with the free end of the bars oriented towards the center of the column.

21.10.2.3 Columns or boundary elements of special reinforced concrete structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with 21.4.4 provided below the top of the footing. This reinforcement shall extend into the footing a distance no less than the smaller of the depth of the footing, mat, or pile cap, or the development length in tension of the longitudinal reinforcement.

21.10.2.4 Where earthquake effects create uplift forces in boundary elements of special reinforced concrete structural walls or columns, flexural reinforcement shall be provided in the top of the footing, mat or pile cap to resist the design load combinations, and shall not be less than required by 10.5.

21.10.3 Grade beams and slabs on grade

21.10.3.1 Grade beams designed to act as horizontal ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supported column or anchored within the pile cap or footing at all discontinuities.

21.10.3.2 Grade beams designed to act as horizontal ties between pile caps or footings shall be proportioned such that the smallest cross-sectional dimension shall be equal to or greater than the clear spacing between connected columns divided by 20, but need not be greater than 450 mm. Closed ties shall be provided at a spacing not to exceed the lesser of one-half the smallest orthogonal cross-sectional dimension or 300 mm.

21.10.3.3 Grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the Iateral-force-resisting system shall conform to 21.3.

21.10.3.4 Slabs on grade that resist seismic forces from walls or columns that are part of the lateral-force-resisting system shall be designed as structural diaphragms in accordance with 21.9. The design drawings shall clearly state that the slab on grade is a structural diaphragm and part of the lateral-force-resisting system.

21.10.4 Piles, piers, and caissons

21.10.4.1 Provisions of 21.10.4 shall apply to concrete piles, piers, and caissons supporting structures designed for earthquake resistance.

21.10.4.2 Piles, piers, or caissons resisting tension loads shall have continuous longitudinal reinforcement over the length resisting design tension forces. The longitudinal reinforcement shall be detailed to transfer tension forces within the pile cap to supported structural members.

21.10.4.3 Where tension forces induced by earthquake effects are transferred between pile cap or mat foundation and precast pile by reinforcing bars grouted or post-installed in the top of the pile, the grouting system shall have been demonstrated by test to develop at least 125 percent of the specified yield strength of the bar.

21.10.4.4 Piles, piers, or caissons shall have transverse reinforcement in accordance with 21.4.4 at locations (a) and (b):

- **(a)** At the top of the member for at least 5 times the member cross-sectional dimension, but not less than 2 m below the bottom of the pile cap;
- **(b)** For the portion of piles in soil that is not capable of providing lateral support, or in air and water, along the entire unsupported length plus the length required in 21.10.4.4(a).

21.10.4.5 For precast concrete driven piles, the length of transverse reinforcement provided shall be sufficient to account for potential variations in the elevation in pile tips.

21.10.4.6 Concrete piles, piers, or caissons in foundations supporting one- and two-story stud bearing wall construction are exempt from the transverse reinforcement requirements of Section 21.10.4.4 and 21.10.4.5.

21.10.4.7 Pile caps incorporating batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns. The slenderness effects of batter piles shall be considered for the portion of the piles in soil that is not capable of providing lateral support, or in air or water.

SECTION 21.11
FRAME MEMBERS NOT PROPORTIONED TO RESIST FORCES INDUCED BY EARTHQUAKE MOTIONS

21.11.1 Frame members assumed not to contribute to lateral resistance shall be detailed according to Section 21.11.2 or 21.11.3 depending on the magnitude of moments induced in those members if subjected to the design displacement. If effects of design displacements are not explicitly checked, it shall be permitted to apply the requirements of Section 21.11.3.

21.11.2 When the induced moments and shears under design displacements of 21.11.1 combined with the factored gravity moments and shears do not exceed the design moment and shear strength of the frame member, the conditions of Section 21.11.2.1, 21.11.2.2, and 21.11.2.3 shall be satisfied. The gravity load combinations of (1.2*D* + 1.0*L*) or 0.90*D*, whichever is critical, shall be used. The load factor on *L* shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where the live load, *L* is greater than 5 kN/m^2.

21.11.2.1 Members with factored gravity axial forces not exceeding $A_g f_c^{'}/10$ shall satisfy 21.3.2.1.

Stirrups shall be spaced not more than $d/2$ throughout the length of the member.

21.11.2.2 Members with factored gravity axial forces exceeding $A_g f_c^{'}/10$ shall satisfy Section 21.4.3, 21.4.4.1(c), 21.4.4.3, and 21.4.5. The maximum longitudinal spacing of ties shall be s_o for the full column height. The spacing s_o shall not be more than six diameters of the smallest longitudinal bar enclosed or 150 mm, whichever is smaller.

21.11.2.3 Members with factored gravity axial forces exceeding $0.35P_o$ shall satisfy Section 21.11.2.2 and the amount of transverse reinforcement provided shall be one-half of that required by 21.4.4.1 but shall not exceed a spacing s_o for the full height of the column.

21.11.3 If the induced moment or shear under design displacements of Section 21.11.1 exceeds the design moment or shear strength of the frame member, or if induced moments are not calculated, the conditions of Section 21.11.3.1, 21.11.3.2, and 21.11.3.3 shall be satisfied.

21.11.3.1 Materials shall satisfy Section 21.2.4 and 21.2.5. Mechanical splices shall satisfy 21.2.6 and welded splices shall satisfy 21.2.7.1.

21.11.3.2 Members with factored gravity axial forces not exceeding $A_g f_c^{'}/10$ shall satisfy Section 21.3.2.1 and 21.3.4. Stirrups shall be spaced at not more than $d/2$ throughout the length of the member.

21.11.3.3 Members with factored gravity axial forces exceeding $A_g f_c^{'}/10$ shall satisfy Section 21.4.3.1, 21.4.4, 21.4.5, and 21.5.2.1.

21.11.4 Precast concrete frame members assumed not to contribute to lateral resistance, including their connections, shall satisfy (a), (b), and (c), in addition to Sections 21.11.1 through 21.11.3:

(a) Ties specified in Section 21.11.2.2 shall be provided over the entire column height, including the depth of the beams;

(b) Structural integrity reinforcement, as specified in 16.5, shall be provided; and

(c) Bearing length at support of a beam shall be at least 50 mm longer than determined from calculations using bearing strength values from 10. 17.

SECTION 21.12
REQUIREMENTS FOR INTERMEDIATE MOMENT FRAMES

21.12.1 The requirements of this section apply to intermediate moment frames.

21.12.2 Reinforcement details in a frame member shall satisfy 21.12.4 if the factored compressive axial load for the member does not exceed $A_g f_c^{'}/10$. If the factored compressive axial load is larger, frame reinforcement details shall satisfy 21.12.5 unless the member has spiral reinforcement according to Eq. (10-5). If a two-way slab system without beams is treated as part of a frame resisting earthquake effect, reinforcement details in any span resisting moments caused by lateral force shall satisfy 21.12.6.

21.12.3 Design shear strength of beams, columns, and two-way slabs resisting earthquake effect shall not be less than either (a) or (b):

(a) The sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for factored gravity loads;

(b) The maximum shear obtained from design load combinations that include earthquake effect E, with E assumed to be twice that prescribed by the governing code for earthquake-resistant design.

21.12.4 Beams

21.12.4.1 The positive moment strength at the face of the joint shall be not less than one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the member shall be less than one-fifth the maximum moment strength provided at the face of either joint.

21.12.4.2 At both ends of the member, hoops shall be provided over lengths equal to twice the member depth measured from the face of the supporting member toward midspan. The first hoop shall be located at not more than 50 mm from the face of the supporting member. Maximum hoop spacing shall not exceed the smallest of (a), (b), (c), or (d):

- **(a)** $d/4$;
- **(b)** Eight times the diameter of the smallest longitudinal bar enclosed;
- **(c)** 24 times the diameter of the hoop bar;
- **(d)** 250 mm.

21.12.4.3 Stirrups shall be placed at not more than $d/2$ throughout the length of the member.

21.12.5 Columns

21.12.5.1 Columns shall be spirally reinforced in accordance with 7.10.4 or shall conform with 21.12.5.2 through 21.12.5.4.Section 21.12.5.5 shall apply to all columns.

21.12.5.2 At both ends of the member, hoops shall be provided at spacing s_o over a length ℓ_o measured from the joint face. Spacing s_o shall not exceed the smallest of (a), (b), (c), and (d):

- **(a)** Eight times the diameter of the smallest longitudinal bar enclosed:
- **(b)** 24 times the diameter of the hoop bar;
- **(c)** One-half of the smallest cross-sectional dimension of the frame member;
- **(d)** 250 mm.

Length ℓ_o shall not be less than the largest of (e), (f), and (g):

- **(e)** One-sixth of the clear span of the member;
- **(f)** Maximum cross-sectional dimension of the member;
- **(g)** 500 mm.

21.12.5.3 The first hoop shall be located at not more than $s_o/2$ from the joint face.

21.12.5.4 Outside the length ℓ_o spacing of transverse reinforcement shall conform to 7.10 and 11.5.4.1.

21.12.5.5 Joint transverse reinforcement shall conform to 11.11.2.

21.12.6 Two-way slabs without beams

21.12.6.1 Factored slab moment at support related to earthquake effect shall be determined for load combinations given in Eq. (9-5) and (9-7). All reinforcement provided to resist M_s the portion of slab moment balanced by support moment, shall be placed within the column strip defined in 13.2.1.

21.12.6.2 The fraction, defined by Eq. (13-1), of moment M_s shall be resisted by reinforcement placed within the effective width specified in 13.5.3.2. Effective slab width for exterior and corner connections shall not extend beyond the column face a distance greater than c_t measured perpendicular to the slab span.

21.12.6.3 Not less than one-half of the reinforcement in the column strip at support shall be placed within the effective slab width given in 13.5.3.2.

21.12.6.4 Not less than one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span

21.12.6.5 Continuous bottom reinforcement in the column strip shall be not less than one-third of the top reinforcement at the support in the column strip.

21.12.6.6 Not less than one-half of all bottom middle strip reinforcement and all bottom column strip reinforcement at midspan shall be continuous and shall develop its yield strength at face of support as defined in 13.6.2.5.

21.12.6.7 At discontinuous edges of the slab all top and bottom reinforcement at support shall be developed at the face of support as defined in 13.6.2.5.

21.12.6.8 At the critical sections for columns defined in 11.12.1.2, two-way shear caused by factored gravity loads shall not exceed $0.4\phi V_c$ where V_c shall be calculated as defined in 11.12.2.1 for nonprestressed slabs and in 11.12.2.2 for prestressed slabs. It shall be permitted to waive this requirement if the contribution of the earthquake-induced factored two- way shear stress transferred by eccentricity of shear in accordance with Section 11.12.6.1 and 11.12.6.2 at the point of maximum stress does not exceed one-half of the stress ϕV_n permitted by 11.12.6.2.

SECTION 21.13
INTERMEDIATE PRECAST STRUCTURAL WALLS

21.13.1 The requirements of this section apply to intermediate precast structural walls used to resist forces induced by earthquake motions.

21.13.2 In connections between wall panels, or between wail panels and the foundation, yielding shall be restricted to reinforcement.

21.13.3 Elements of the connection that are not designed to yield shall develop at least $1.5S_y$.

APPENDIX A
STRUT-AND-TIE MODELS

SECTION A.0
NOTATION

a = shear span, equal to the distance between a load and a support in a structure, mm

A_c = the effective cross-sectional area at one end of a strut in a strut-and-tie model, taken perpendicular to the axis of the strut, mm²

A_n = area of a face of a nodal zone or a section through a nodal zone, mm²

A_{ps} = area of prestressed reinforcement in a tie, mm²

A_{si} = area of surface reinforcement in the ith layer crossing a strut, mm²

A_{st} = area of nonprestressed reinforcement in a tie, mm²

$A_s^{'}$ = area of compression reinforcement in a strut, mm²

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm

$f_c^{'}$ = specified compressive strength of concrete, MPa

f_{cu} = effective compressive strength of the concrete in a strut or a nodal zone, MPa

$f_s^{'}$ = stress in compression reinforcement, MPa

f_{se} = effective stress after losses in prestressed reinforcement, MPa

f_y = specified yield strength of nonprestressed reinforcement, MPa

F_n = nominal strength of a strut, tie, or nodal zone, N

F_{nn} = nominal strength of a face of a nodal zone, N

F_{ns} = nominal strength of a strut, N

F_{nt} = nominal strength of a tie, N

F_u = factored force acting in a strut, tie, bearing area, or nodal zone in a strut-and-tie model, N

ℓ_n = clear span, mm

s_i = spacing of reinforcement in the i^{th} layer adjacent to the surface of the member, mm

w_s = effective width of strut, mm

w_t = effective width of tie. mm

β_s = factor to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete in a strut

β_n = factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone.

γ_i = angle between the axis of a strut and the bars in the i^{th} layer of reinforcement crossing that strut

Δf_p = increase in stress in prestressing tendons due to factored loads, MPa

ε_s = the strain in the longitudinal reinforcement in a compression zone or a longitudinally reinforced strut

θ = angle between the axis of a strut or compression field and the tension chord of the member
λ = correction factor related to the unit weight of concrete. See 11.7.4.3
ϕ = strength reduction factor

SECTION A.1
DEFINITIONS

B-region — A portion of a member in which the plane sections assumption of flexure theory from 10.2.2 can be applied.

Discontinuity — An abrupt change in geometry or loading.

D-region — The portion of a member within a distance equal to the member height h or depth d from a force discontinuity or a geometric discontinuity.

Deep beam — See 10.7.1 and 11.8.1.

Node — The point in a joint in a strut-and-tie model where the axes of the struts, ties, and concentrated forces acting on the joint intersect.

Nodal zone —The volume of concrete around a node that is assumed to transfer strut-and-tie forces through the node.

Strut — A compression member in a strut-and-tie model. A strut represents the resultant of a parallel or a fan-shaped compression field.

Bottle-shaped strut —A strut that is wider at mid-length than at its ends.

Strut-and-tie model — A truss model of a structural member, or of a D-region in such a member, made up of struts and ties connected at nodes, capable of transferring the factored loads to the supports or to adjacent B-regions.

Tie — A tension member in a strut-and-tie model.

SECTION A.2
STRUT-AND-TIE MODEL DESIGN PROCEDURE

A.2.1 It shall be permitted to design structural concrete members or D-regions in such members, by modeling the member or region as an idealized truss. The truss model shall contain struts, ties, and nodes as defined in A.1. The truss model shall be capable of transferring all factored loads to the supports or adjacent B-regions.

A.2.2 The strut-and-tie model shall be in equilibrium with the applied loads and the reactions.

A.2.3 In determining the geometry of the truss, the dimensions of the struts, ties, and nodal zones shall be taken into account.

A.2.4 Ties shall be permitted to cross struts. Struts shall cross or overlap only at nodes.

A.2.5 The angle between the axes of any strut and any tie entering a single node shall not be taken as less than 25 degrees.

A.2.6 Design of struts, ties, and nodal zones shall be based on

$$\phi F_n \geq F_u \tag{A-1}$$

where F_u is the force in a strut or tie, or the force acting on one face of a nodal zone, due to the factored loads; F_n is the nominal strength of the strut, tie, or nodal zone; and ϕ is the strength reduction factor specified in 9.3.2.6.

SECTION A.3
STRENGTH OF STRUTS

A.3.1 The nominal compressive strength of a strut without longitudinal reinforcement shall be taken as the smaller value of

$$F_{ns} = f_{cu} A_c \tag{A-2}$$

at the two ends of the strut, where A_c is the cross-sectional area at one end of the strut, and f_{cu} is the smaller of (a) and (b):

(a) the effective compressive strength of the concrete in the strut given in A.3.2;

(b) the effective compressive strength of the concrete in the nodal zone given in A.5.2.

A.3.2 The effective compressive strength of the concrete in a strut shall be taken as

$$f_{cu} = 0.85 \beta_s f_c^{'} \tag{A-3}$$

A.3.2.1 For a strut of uniform cross-sectional area over its length…..……. $\beta_s = 1.0$

A.3.2.2 For struts located such that the width of the midsection of the strut is larger than the width at the nodes (bottle-shaped struts):

(a) with reinforcement satisfying A.3.3 … ………………….. $\beta_s = 0.75$

(b) without reinforcement satisfying A.3.3 ……….........…… $\beta_s = 0.60\lambda$

where λ is given in 11.7.4.3.

A.3.2.3 For struts in tension members, or the tension flanges of members … $\beta_s = 0.40$

A.3.2.4 For all other cases ……………………………………………… $\beta_s = 0.60$

A.3.3 If the value of β_s specified in A.3.2.2(a) is used, the axis of the strut shall be crossed by reinforcement proportioned to resist the transverse tensile force resulting from the compression force spreading in the strut. It shall be permitted to assume the compressive force in the strut spreads at a slope of 2 longitudinal to 1 transverse to the axis of the strut.

A.3.3.1 For $f_c^{'}$ not greater than 40 MPa, the requirement of A.3.3 shall be permitted to be satisfied by the axis of the strut being crossed by layers of reinforcement that satisfy

$$\sum \frac{A_{si}}{bs_i} \sin \gamma_i \geq 0.003 \tag{A-4}$$

where A_{si}, is the total area of reinforcement at spacing s_i in a layer of reinforcement with bars at an angle γ_i to the axis of the strut.

A.3.3.2 The reinforcement required in A.3.3 shall be placed in either: two orthogonal directions at angles γ_1 and γ_2 to the axis of the strut, or in one direction at an angle γ to the axis of the strut. If the reinforcement is in only one direction, γ shall not be less than 40 deg.

A.3.4 If documented by tests and analyses, it shall be permitted to use an increased effective compressive strength of a strut due to confining reinforcement.

A.3.5 The se of compression reinforcement shall be permitted to increase the strength of a strut. Compression reinforcement shall be properly anchored, parallel to the axis of the strut, located within the strut, and enclosed in ties or spirals satisfying 7.10. In such cases, the strength of a longitudinally reinforced strut is:

$$F_{ns} = f_{cu} A_c + A_s^{'} f_s^{'} \qquad \text{(A-5)}$$

SECTION A.4
STRENGTH OF TIES

A.4.1 The nominal strength of a tie shall be taken as

$$F_{nt} = A_{st} f_y + A_{ps} \left(f_{se} + \Delta f_p \right) \qquad \text{(A-6)}$$

where $\left(f_{se} + \Delta f_p \right)$ shall not exceed f_{py} and A_{ps} is zero for nonprestressed members.

In Eq. (A-6), it shall be permitted to take Δf_p equal to 420 MPa for bonded prestressed reinforcement, or 70 MPa for unbonded prestressed reinforcement. Other values of Δf_p shall be permitted when justified by analysis.

A.4.2 The axis of the reinforcement in a tie shall coincide with the axis of the tie in the strut-and-tie model.

A.4.3 Tie reinforcement shall be anchored by mechanical devices, post-tensioning anchorage devices, standard hooks, or straight bar development as required by A.4.3.1 through A.4.3.4.

A.4.3.1 Nodal zones shall develop the difference between the tie force on one side of the node and the tie force on the other side.

A.4.3.2 At nodal zones anchoring one tie, the tie force shall be developed at the point where the centroid of the reinforcement in a tie leaves the extended nodal zone and enters the span.

A.4.3.3 At nodal zones anchoring two or more ties, the tie force in each direction shall be developed at the point where the centroid of the reinforcement in the tie leaves the extended nodal zone.

A.4.3.4 The transverse reinforcement required by A.3.3 shall be anchored in accordance with 12.13.

SECTION A.5
STRENGTH OF NODAL ZONES

A.5.1 The nominal compression strength of a nodal zone shall be

$$F_{nn} = f_{cu} A_n \qquad \text{(A-7)}$$

where f_{cu} is the effective compressive strength of the concrete in the nodal zone as given in A.5.2 and A_n is (a) or (b):

(a) the area of the face of the nodal zone that F_u acts on, taken perpendicular to the line of action of F_u, or

(b) the area of a section through the nodal zone, taken perpendicular to the line of action of the resultant force on the section.

A.5.2 Unless confining reinforcement is provided within the nodal zone and its effect is supported by tests and analysis, the calculated effective compressive stress on a face of a nodal zone due to the strut-and-tie forces shall not exceed the value given by:

$$f_{cu} = 0.85\beta_n f_c^{'} \qquad \text{(A-8)}$$

where the value of β_n is given in A.5.2.1 through A.5.2.3.

A.5.2.1 In nodal zones bounded by struts or bearing areas, or both...... $\beta_n = 1.0$;

A.5.2.2 In nodal zones anchoring one tie .. .. $\beta_n = 0.8$;

or

A.5.2.3 In nodal zones anchoring two or more ties .... $\beta_n = 0.60$.

A.5.3 In a three-dimensional strut-and-tie model, the area of each face of a nodal zone shall not be less than that given in A.5.1, and the shape of each face of the nodal zones shall be similar to the shape of the projection of the end of the strut onto the corresponding faces of the nodal zones.

APPENDIX B
ALTERNATIVE PROVISIONS FOR REINFORCED AND PRESTRESSED CONCRETE FLEXURAL AND COMPRESSION MEMBERS

SECTION B.0
NOTATION

A_g = gross area of section, mm²

A_{ps} = area of prestressed reinforcement in tension zone, mm²

A_s = area of tension reinforcement, mm^2

$A_s^{'}$ = area of compression reinforcement, mm² specified

b = width of compression face of member, mm

$f_c^{'}$ = specified compressive strength of concrete, MPa

f_{ps} = average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses), MPa

f_y = specified yield strength of nonpres-tressed reinforcement, MPa

d = distance from extreme compression fiber to centroid of tension reinforcement, mm

$d^{'}$ = distance from extreme compression fiber to centroid of compression reinforcement, mm

d_p = distance from extreme compression fiber to centroid of prestressed reinforcement, mm

d_s = distance from extreme tension fiber to centroid of tension reinforcement, mm

h = overall thickness of member, mm

P_b = nominal axial load strength at balanced strain conditions, N. See 10.3.2

P_n = nominal axial load strength at given eccentricity, N

β_1 = factor defined in 10.2.7.3

ρ = reinforcement ratio for nonprestressed tension reinforcement = A_s / bd

$\rho^{'}$ = reinforcement ratio for nonprestressed compression reinforcement = $A_s^{'} / bd$

ρ_b = reinforcement ratio producing balanced strain conditions. See 10.3.2.

ρ_p = ratio of prestressed reinforcement = A_{ps} / bd_p

ω = $\rho f_y / f_c^{'}$

$\omega^{'}$ = $\rho^{'} f_y / f_c^{'}$

ω_p = $\rho_p f_{ps} / f_c^{'}$

$\omega_w, \omega_{pw}, \omega_w^{'}$ = reinforcement indices for flanged sections. computed as for ω, ω_p and $\omega^{'}$ except that b shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only

SECTION B.1
SCOPE

B.1.1 Design for flexure and axial load by provisions of Appendix B shall be permitted. When Appendix B is used in design, B.8.4, B.8.4.1, B.8.4.2, and B.8.4.3 shall replace the corresponding numbered sections in Chapter 8; B.10.3.3 shall replace 10.3.3, 10.3.4, and 10.3.5, except 10.3.5.1 shall remain; B.18.8.1, B.18.8.2, and B.18.8.3 shall replace the corresponding numbered sections in Chapter 18; B.18.10.4, B.18.10.4.1, B.18.10.4.2, and B.18.10.4.3 shall replace 18.10.4, 18.10.4.1, and 18.10.4.2. If any section in this appendix is used, all sections in this appendix shall be substituted in the body of the code, and all other sections in the body of the code shall be applicable.

B.8.4 **Redistribution of negative moments in continuous nonprestressed flexural members**

For criteria on moment redistribution for prestressed concrete members, see B.18.10.4.

B.8.4.1 Except where approximate values for moments are used, it shall be permitted to increase or decrease negative moments calculated by elastic theory at supports of continuous flexural members for any assumed loading arrangement by not more than

$$20\left(1-\frac{\rho-\rho'}{\rho_b}\right) \textit{percent}$$

B.8.4.2 The modified negative moments shall be used for calculating moments at sections within the spans

B.8.4.3 Redistribution of negative moments shall be made only when the section at which moment is reduced is so designed that ρ or $\rho-\rho'$ is not greater than $0.5\rho_b$, where

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{600}{600+f_y}\right) \qquad \text{(B-1)}$$

B.10.3 **General principles and requirements**

B.10.3.3 For flexural members and members subject to combined flexure and compressive axial load when the design axial load strength ϕP_n is less than the smaller of $0.1 f_c' A_g$ or ϕP_b the ratio of reinforcement, ρ, provided shall not exceed 0.65 of the ratio ρ_b that would produce balanced strain conditions for the section under flexure without axial load. For members with compression reinforcement, the portion of ρ_b equalized by compression reinforcement need not be reduced by the 0.65 factor.

B.18.1 **Scope**

B.18.1.3 The following provisions of SBC 304 shall not apply to prestressed concrete, except as specifically noted: Sections 7.6.5, B.8.4, 8.10.2, 8.10.3, 8.10.4, 8.11, B.10.3.3, 10.5, 10.6, 10.9.1, and 10.9.2; Chapter 13; and Sections 14.3, 14.5, and 14.6.

B.18.8 Limits for reinforcement of flexural members

B.18.8.1 Ratio of prestressed and nonprestressed reinforcement used for computation of moment strength of a member, except as provided in B.18.8.2, shall be such that $\omega_p, [\omega_p + (d/d_p)(\omega - \omega')]$, or $[\omega_{pw} + (d/d_p)(\omega_w - \omega'_w)]$ is not greater than $0.36\beta_1$ except as permitted in B.18.8.2.

B.18.8.2 When a reinforcement ratio exceeds the limit specified in B.18.8.1 is provided, design moment strength shall not exceed the moment strength based on the compression portion of the moment couple.

B.18.8.3 Total amount of prestressed and nonprestressed reinforcement shall be adequate to develop a factored load at least 1.2 times the cracking load computed on the basis of the modulus of rupture f_r in 9.5.2.3. This provision shall be permitted to be waived for:

(a) two-way, unbonded post-tensioned slabs; and

(b) flexural members with shear and flexural strength at least twice that required by 9.2.

B.18.10 Statically indeterminate structures

B.18.10.1 Frames and continuous construction of prestressed concrete shall be designed for satisfactory performance at service load conditions and for adequate strength.

B.18.10.2 Performance at service load conditions shall be determined by elastic analysis, considering reactions, moments, shears, and axial forces produced by prestressing, creep, shrinkage, temperature change, axial deformation, restraint of attached structural elements, and foundation settlement.

B.18.10.3 Moments to be used to compute required strength shall be the sum of the moments due to reactions induced by prestressing (with a load factor of 1.0) and the moments due to factored loads. Adjustment of the sum of these moments shall be permitted as allowed in B.18.10.4.

B.18.10.4 Redistribution of negative moments in continuous prestressed flexural members

B.18.10.4.1 Where bonded reinforcement is provided at supports in accordance with 18.9, negative moments calculated by elastic theory for any assumed loading, arrangement shall be permitted to be increased or decreased by not more than

$$20\left(1 - \frac{\omega_p + \dfrac{d}{d_p}(\omega - \omega')}{0.36\beta_1}\right) \textit{percent}$$

B.18.10.4.2 The modified negative moments shall be used for calculating moments at sections within spans for the same loading arrangement.

B.18.10.4.3 Redistribution of negative moments shall be made only when the section at which moment is reduced is so designed that $\omega_p, (\omega_p + (d/d_p)(\omega - \omega'))$ or $(\omega_{pw} + (d/d_p)(\omega_w - \omega_w'))$, whichever is applicable, is not greater than $0.24\beta_1$.

APPENDIX C
TWO-WAY SLABS – COEFFICIENTS METHODS

SECTION C.0
GENERAL

C.0.1 There are several satisfactory methods for designing two-way slabs. Although they may give somewhat different results in details, the resulting floors give reasonable overall safety factors. Two methods which have been used extensively with satisfactory results are given in this appendix.

SECTION C.1
METHOD 1

C.1.1 **Notation**

C = moment coefficient for two-way slabs as given in Table 1.

m = ratio of short span to long span for two-way slabs.

S = length of short span for two-way slabs. The span shall be considered as the center-to-center distance between supports or the clear span plus twice the thickness of slab, whichever value is the smaller.

w = total factored load per sq. meter.

C.1.2 **Limitations**

These recommendations are intended to apply to slabs (solid or ribbed), isolated or continuous, supported on all four sides by walls or beams, in either case built monolithically with the slabs.

C.1.3 **Design strips**

A two-way slab shall be considered as consisting of strips in each direction as follows:

1. A middle strip one-half panel in width, symmetrical about panel center line and extending through the panel in the direction in which moments are considered.
2. A column strip one-half panel in width, occupying the two quarter-panel areas outside the middle strip.
3. Where the ratio of short to long span is less than 0.5, the middle strip in the short direction shall be considered as having a width equal to the difference between the long and short span, the remaining area representing the two column strips.

C.1.4 The critical sections for moment calculations are referred to as principal design sections and are located as follows:

1. For negative moment, along the edges of the panel at the faces of the supporting beams.
2. For positive moment, along the center lines of the panels.

C.1.5 **Bending moments**. The bending moments for the middle strips shall be computed from the formula.

$$M = CwS^2$$

C.1.5.1 The average moments per meter of width in the column strip shall be two-thirds of the corresponding moments in the middle strip. In determining the spacing of the reinforcement in the column strip, the moment may be assumed to vary from a maximum at the edge of the middle strip to a minimum at the edge of the panel.

C.1.5.2 Where the negative moment on one side of a support is less than 80 % of that on the other side, two-thirds of the difference shall be distributed in proportion to the relative stiffnesses of the slabs.

C.1.6 **Shear.** The shear stresses in the slab may be computed on the assumption that the load is distributed to the supports in accordance with C.1.6.1

C.1.6.1 **Supporting beams**. The loads on the supporting beams for a two-way rectangular panel may be assumed as the load within the tributary areas of the panel bounded by the intersection of 45-deg lines from the corners with the median line of the panel parallel to the long side.

C.1.6.2 The bending moments may be determined approximately by using an equivalent uniform load per linear meter of beam for each panel supported as follows:

For the short span: $$\frac{wS}{3}$$

For the long span: $$\frac{wS}{3}\ \frac{\left(3-m^2\right)}{2}$$

Table 1 - Moment Coefficients - Method 1

Moments	Short span Values of m 1.0	0.9	0.8	0.7	0.6	0.5	Long span all values of m
Case 1 – Interior panels Negative moment at – Continuous edge Discontinuous edge Positive moment at midspan	 0.033 - 0.025	 0.040 - 0.030	 0.048 - 0.036	 0.055 - 0.041	 0.063 - 0.047	 0.083 - 0.062	 0.033 - 0.025
Case 2 – One edge discontinuous Negative moment at – Continuous edge Discontinuous edge Positive moment at midspan	 0.041 0.021 0.031	 0.048 0.024 0.036	 0.055 0.027 0.041	 0.062 0.031 0.047	 0.069 0.035 0.052	 0.085 0.042 0.064	 0.041 0.021 0.031
Case 3 – Two edges discontinuous Negative moment at – Continuous edge Discontinuous edge Positive moment at midspan	 0.049 0.025 0.037	 0.057 0.028 0.043	 0.064 0.032 0.048	 0.071 0.036 0.054	 0.078 0.039 0.059	 0.090 0.045 0.068	 0.049 0.025 0.037
Case 4 –Three edges discontinuous Negative moment at – Continuous edge Discontinuous edge Positive moment at midspan	 0.058 0.029 0.044	 0.066 0.033 0.050	 0.074 0.037 0.056	 0.082 0.041 0.062	 0.090 0.045 0.068	 0.098 0.049 0.074	 0.058 0.029 0.044
Case 5 – Four edges discontinuous Negative moment at – Continuous edge Discontinuous edge Positive moment at midspan	 - 0.033 0.050	 - 0.038 0.057	 - 0.043 0.064	 - 0.047 0.072	 - 0.053 0.080	 - 0.055 0.083	 - 0.033 0.050

SECTION C.2
METHOD 2

C.2.0 **Notations**

ℓ_a = length of clear span in short direction

ℓ_b = length of clear span in long direction

C = moment coefficients for two-way slabs as given in Tables 2, 3, and 4. Coefficients have identifying indeces, such as $C_{a,neg}, C_{b,neg}, C_{a,dl}, C_{b,dl}, C_{a,ll}, C_{b,ll}$

m = ratio of short span to long span for two-way slabs

w = uniform load per sq m. For negative moments, w is the total factored dead load plus live load for use in Table 2. For positive moments, w is to be separated into dead and live loads for use in Tables 3 and 4.

C.2.1 **Limitations.** A two-way slab shall be considered as consisting of strips in each direction as follows:

1. A middle strip one-half panel in width, symmetrical about panel center line and extending through the panel in the direction in which moments are considered.

2. A column strip one-half panel in width, occupying the two quarter-panel areas outside the middle strip.

3. Where the ratio of short to long span is less than 0.5, the slab shall be considered as a one-way slab and is to be designed in accordance with Chapter 9 except that negative reinforcement, as required for a ratio of 0.5, shall be provided along the short edge.

4. At discontinuous edges, a negative moment one-third (1/3) of the positive moment is to be used.

C.2.2 Critical sections for moment calculations are located as follows:

1. For negative moment along the edges of the panel at the faces of the supports.

2. For positive moment, along the center lines of the panels.

C.2.3 Bending moments — The bending moments for the middle strips be computed by the use of Tables 2, 3, and 4 from:

$$M_a = C_a w \ell_a^2$$

and

$$M_b = C_b w \ell_b^2$$

The bending moments in the column strips shall be gradually reduced from the full value M_a and M_b from the edge of the middle strip to one-third (1/3) of these values at the edge of the panel.

where the negative moment on one side of a support is less than 80 percent of that on the other side, the difference shall be distributed in proportion to the relative stiffnesses of the slabs.

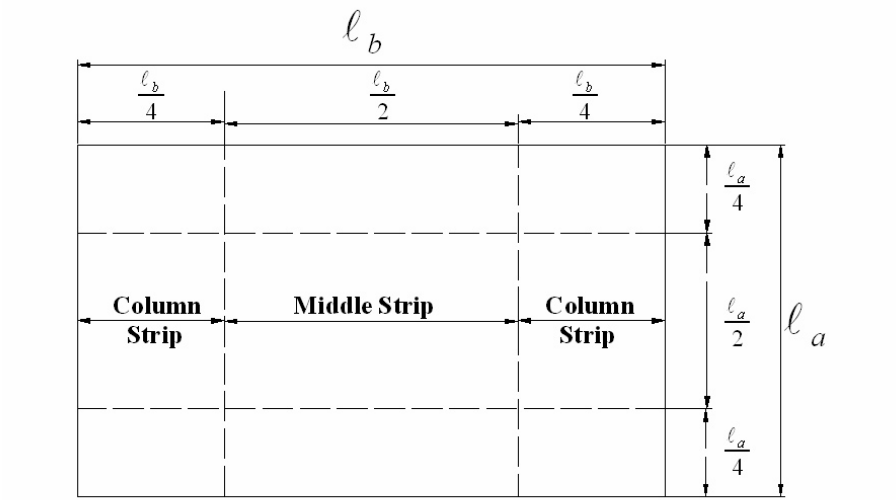
C.2.4 **Factored shear in slab systems with beams**

C.2.4.1 Beams shall be proportioned to resist shear caused by factored loads on tributary areas which are bounded by 45 deg lines drawn from the corners of the panels and the centerlines of the adjacent panels parallel to the long sides.

C.2.4.2 In addition to shears calculated according to C.2.4.1, beams shall be proportioned to resist shears caused by factored loads applied directly on beams.

C.2.4.3 Computation of slab shear strength on the assumption that load is distributed to supporting beams in accordance with C.2.4.1 shall be permitted. Resistance to total shear occurring on a panel shall be provided.

C.2.4.4 Shear strength shall satisfy the requirements of Chapter 11.



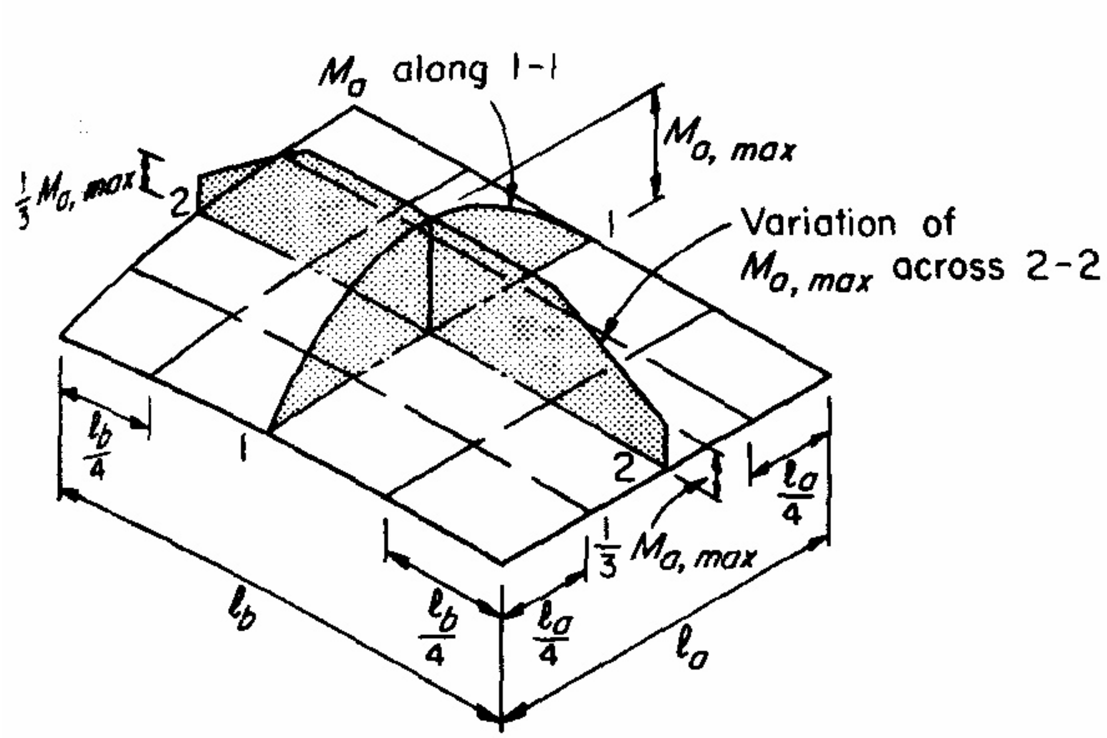


Table 2: Coefficients for negative moments in slabs (*)

$$M_{a,neg} = C_{a,neg} w \ell_a^2$$
$$M_{b,neg} = C_{b,neg} w \ell_b^2$$

Where w = total factored uniform dead plus live load

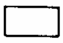
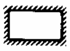
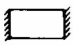
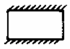
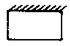
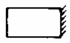
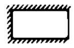
Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{a,neg}$		0.045		0.050	0.075	0.071		0.033	0.061
	$C_{b,neg}$		0.045	0.076	0.050			0.071	0.061	0.033
0.95	$C_{a,neg}$		0.050		0.055	0.079	0.075		0.038	0.065
	$C_{b,neg}$		0.041	0.072	0.045			0.067	0.056	0.029
0.90	$C_{a,neg}$		0.055		0.060	0.080	0.079		0.043	0.068
	$C_{b,neg}$		0.037	0.070	0.040			0.062	0.052	0.025
0.85	$C_{a,neg}$		0.060		0.066	0.082	0.083		0.049	0.072
	$C_{b,neg}$		0.031	0.065	0.034			0.057	0.046	0.021
0.80	$C_{a,neg}$		0.065		0.071	0.083	0.086		0.055	0.075
	$C_{b,neg}$		0.027	0.061	0.029			0.051	0.041	0.017
0.75	$C_{a,neg}$		0.069		0.076	0.085	0.088		0.061	0.078
	$C_{b,neg}$		0.022	0.056	0.024			0.044	0.036	0.014
0.70	$C_{a,neg}$		0.074		0.081	0.086	0.091		0.068	0.081
	$C_{b,neg}$		0.017	0.050	0.019			0.038	0.029	0.011
0.65	$C_{a,neg}$		0.077		0.085	0.087	0.093		0.074	0.083
	$C_{b,neg}$		0.014	0.043	0.015			0.031	0.024	0.008
0.60	$C_{a,neg}$		0.081		0.089	0.088	0.095		0.080	0.085
	$C_{b,neg}$		0.010	0.035	0.011			0.024	0.018	0.006
0.55	$C_{a,neg}$		0.084		0.092	0.089	0.096		0.085	0.086
	$C_{b,neg}$		0.007	0.028	0.008			0.019	0.014	0.005
0.50	$C_{a,neg}$		0.086		0.094	0.090	0.097		0.089	0.088
	$C_{b,neg}$		0.006	0.022	0.006			0.014	0.010	0.003

*A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

Table 3: Coefficients for dead-load positive moments in slabs (*)

$$M_{a,pos,dl} = C_{a,dl} w \ell_a^2$$
$$M_{b,pos,dl} = C_{b,dl} w \ell_b^2$$

Where w = total factored uniform dead load

Ratio $m = \frac{l_a}{l_b}$		Case 1 	Case 2 	Case 3 	Case 4 	Case 5 	Case 6 	Case 7 	Case 8 	Case 9 
1.00	$C_{a,dl}$	0.036	0.018	0.018	0.027	0.027	0.033	0.027	0.020	0.023
	$C_{b,dl}$	0.036	0.018	0.027	0.027	0.018	0.027	0.033	0.023	0.020
0.95	$C_{a,dl}$	0.040	0.020	0.021	0.030	0.028	0.036	0.031	0.022	0.024
	$C_{b,dl}$	0.033	0.016	0.025	0.024	0.015	0.024	0.031	0.021	0.017
0.90	$C_{a,dl}$	0.045	0.022	0.025	0.033	0.029	0.039	0.035	0.025	0.026
	$C_{b,dl}$	0.029	0.014	0.024	0.022	0.013	0.021	0.028	0.019	0.015
0.85	$C_{a,dl}$	0.050	0.024	0.029	0.036	0.031	0.042	0.040	0.029	0.028
	$C_{b,dl}$	0.026	0.012	0.022	0.019	0.011	0.017	0.025	0.017	0.013
0.80	$C_{a,dl}$	0.056	0.026	0.034	0.039	0.032	0.045	0.045	0.032	0.029
	$C_{b,dl}$	0.023	0.011	0.020	0.016	0.009	0.015	0.022	0.015	0.010
0.75	$C_{a,dl}$	0.061	0.028	0.040	0.043	0.033	0.048	0.051	0.036	0.031
	$C_{b,dl}$	0.019	0.009	0.018	0.013	0.007	0.012	0.020	0.013	0.007
0.70	$C_{a,dl}$	0.068	0.030	0.046	0.046	0.035	0.051	0.058	0.040	0.033
	$C_{b,dl}$	0.016	0.007	0.016	0.011	0.005	0.009	0.017	0.011	0.006
0.65	$C_{a,dl}$	0.074	0.032	0.054	0.050	0.036	0.054	0.065	0.044	0.034
	$C_{b,dl}$	0.013	0.006	0.014	0.009	0.004	0.007	0.014	0.009	0.005
0.60	$C_{a,dl}$	0.081	0.034	0.062	0.053	0.037	0.056	0.073	0.048	0.036
	$C_{b,dl}$	0.010	0.004	0.011	0.007	0.003	0.006	0.012	0.007	0.004
0.55	$C_{a,dl}$	0.088	0.035	0.071	0.056	0.038	0.058	0.081	0.052	0.037
	$C_{b,dl}$	0.008	0.003	0.009	0.005	0.002	0.004	0.009	0.005	0.003
0.50	$C_{a,dl}$	0.095	0.037	0.080	0.059	0.039	0.061	0.089	0.056	0.038
	$C_{b,dl}$	0.006	0.002	0.007	0.004	0.001	0.003	0.007	0.004	0.002

*A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

Table 4: Coefficients for live-load positive moments in slabs (*)

$$M_{a,pos,ll} = C_{a,ll} w \ell_a^2$$
$$M_{b,pos,ll} = C_{b,ll} w \ell_b^2$$

Where w = total factored uniform live load

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{a,ll}$	0.036	0.027	0.027	0.032	0.032	0.035	0.032	0.028	0.030
	$C_{b,ll}$	0.036	0.027	0.032	0.032	0.027	0.032	0.035	0.030	0.028
0.95	$C_{a,ll}$	0.040	0.030	0.031	0.035	0.034	0.038	0.036	0.031	0.032
	$C_{b,ll}$	0.033	0.025	0.029	0.029	0.024	0.029	0.032	0.027	0.025
0.90	$C_{a,ll}$	0.045	0.034	0.035	0.039	0.037	0.042	0.040	0.035	0.036
	$C_{b,ll}$	0.029	0.022	0.027	0.026	0.021	0.025	0.029	0.024	0.022
0.85	$C_{a,ll}$	0.050	0.037	0.040	0.043	0.041	0.046	0.045	0.040	0.039
	$C_{b,ll}$	0.026	0.019	0.024	0.023	0.019	0.022	0.026	0.022	0.020
0.80	$C_{a,ll}$	0.056	0.041	0.045	0.048	0.044	0.051	0.051	0.044	0.042
	$C_{b,ll}$	0.023	0.017	0.022	0.020	0.016	0.019	0.023	0.019	0.017
0.75	$C_{a,ll}$	0.061	0.045	0.051	0.052	0.047	0.055	0.056	0.049	0.046
	$C_{b,ll}$	0.019	0.014	0.019	0.016	0.013	0.016	0.020	0.016	0.013
0.70	$C_{a,ll}$	0.068	0.049	0.057	0.057	0.051	0.060	0.063	0.054	0.050
	$C_{b,ll}$	0.016	0.012	0.016	0.014	0.011	0.013	0.017	0.014	0.011
0.65	$C_{a,ll}$	0.074	0.053	0.064	0.062	0.055	0.064	0.070	0.059	0.054
	$C_{b,ll}$	0.013	0.010	0.014	0.011	0.009	0.010	0.014	0.011	0.009
0.60	$C_{a,ll}$	0.081	0.058	0.071	0.067	0.059	0.068	0.077	0.065	0.059
	$C_{b,ll}$	0.010	0.007	0.011	0.009	0.007	0.008	0.011	0.009	0.007
0.55	$C_{a,ll}$	0.088	0.062	0.080	0.072	0.063	0.073	0.085	0.070	0.063
	$C_{b,ll}$	0.008	0.006	0.009	0.007	0.005	0.006	0.009	0.007	0.006
0.50	$C_{a,ll}$	0.095	0.066	0.088	0.077	0.067	0.078	0.092	0.076	0.067
	$C_{b,ll}$	0.006	0.004	0.007	0.005	0.004	0.005	0.007	0.005	0.004

*A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

APPENDIX D
ANCHORING TO CONCRETE

SECTION D.0
NOTATION

A_{brg} = bearing area of the head of stud or anchor bolt, mm²

A_{No} = projected concrete failure area of one anchor, for calculation of strength in tension when not limited by edge distance or spacing, mm² (see 5.2.1)

A_N = projected concrete failure area of an anchor or group of anchors, for calculation of strength in tension, as defined in, mm² (see D.5.2.1) A_N shall not be taken greater than nA_{No}

A_{se} = effective cross-sectional area of anchor, mm²

A_{sl} = effective cross-sectional area of expansion or undercut anchor sleeve, if sleeve is within shear plane, mm²

A_{Vo} = projected concrete failure area of one anchor, for calculation of strength in shear, when not limited by corner influences, spacing, or member thickness, mm²(see D.6.2.1)

A_V = projected concrete failure area of an anchor or group of anchors, for calculation of strength in shear, mm² (see 0.6.2.1). A_V shall not be taken greater than nA_{Vo}.

c = distance from center of an anchor shaft to the edge of concrete, mm

c_1 = distance from the center of an anchor shaft to the edge of concrete in one direction, mm; where shear force is applied to anchor, c_1 is in the direction of the shear force.

c_2 = distance from center of an anchor shaft to the edge of concrete in the direction orthogonal to c_1, mm

$c_{\max}$ = the largest edge distance, mm

$c_{\min}$ = the smallest edge distance, mm

d_o = outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, mm (see also D.8.4)

$d_o^{'}$ = value substituted for d_o when an oversized anchor is used, mm (see D.8.4)

e_h = distance from the inner surface of the shaft of a J- or L-bolt to the outer tip of the J- or L-bolt, mm

$e_N^{'}$ = eccentricity of normal force on a group of anchors; the distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension, mm; $e_N^{'}$ is always positive

e_v' = eccentricity of shear force on a group of anchors; the distance between the point of shear force application and the centroid of the group of anchors resisting shear in the direction of the applied shear, mm

$f_c^{'}$ = specified compressive strength of concrete, MPa

f_{ct} = specified tensile strength of concrete, MPa

f_r = modulus of rupture of concrete, MPa (see 9.5.2.3)

f_t = calculated concrete tensile stress in a region of a member, MPa

f_y = specified yield strength of anchor steel, MPa

f_{ut} = specified tensile strength of anchor steel, MPa

f_{utsl} = specified tensile strength of anchor sleeve, MPa

h = thickness of member in which an anchor is anchored, measured parallel to anchor axis, mm

h_{ef} = effective anchor embedment depth, mm (see D.8.5)

k = coefficient for basic concrete breakout strength in tension

k_{cp} = coefficient for pryout strength

ℓ = load bearing length of anchor for shear, not to exceed $8d_o$, mm

= h_{ef} for anchors with a constant stiffness over the full length of the embedded section, such as headed studs or post-installed anchors with one tubular shell over the full length of the embedment depth

= $2d_o$ for torque-controlled expansion anchors with a distance sleeve separated from the expansion sleeve

n = number of anchors in a group

N_b = basic concrete breakout strength in tension of a single anchor in cracked concrete, N (see 5.2.2)

N_{cb} = nominal concrete breakout strength in tension of a single anchor, N (see D.5.2.1)

N_{cbg} = nominal concrete breakout strength in tension of a group of anchors, N (see D.5.2.1)

N_n = nominal strength in tension, N

N_p = pullout strength in tension of a single anchor in cracked concrete, N (see D.5.3.4 or D.5.3.5)

N_{pn} = nominal pullout strength in tension of a single anchor, N (see D.5.3.1)

N_{sb} = side-face blowout strength of a single anchor, N

N_{sbg} = side-face blowout strength of a group of anchors, N

N_s = nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, N (see D.5.1.1 or D.5.1.2)

N_u = factored tensile load, N

s = anchor center-to-center spacing, mm

s_o = spacing of the outer anchors along the edge in a group, mm

t = thickness of washer or plate, mm

V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete, N (see D.6.2.2 or D.6.2.3)

V_{cb} = nominal concrete breakout strength in shear of a single anchor, N (see D.6.2.1)

V_{cbg} = nominal concrete breakout strength in shear of a group of anchors, N (see D.6.2.1)

V_{cp} = nominal concrete pryout strength, N (see D.6.3)

V_n = nominal shear strength, N

V_s = nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, N (see D.6.1.1 or D.6.1.2)

V_u = factored shear load, N

ϕ = strength reduction factor (see D.4.4 and D.4.5)

ψ_1 = modification factor, for strength in tension, to account for anchor groups loaded eccentrically (see D.5.2.4)

ψ_2 = modification factor, for strength in tension, to account for edge distances smaller than $1.5h_{ef}$ (see D.5.2.5)

ψ_3 = modification factor, for strength in tension, to account for cracking (see D.5.2.6 and D.5.2.7)

ψ_4 = modification factor, for pullout strength, to account for cracking (see D.5.3.1 and D.5.3.6)

ψ_5 = modification factor, for strength in shear, to account for anchor groups loaded eccentrically (see D.6.2.5)

ψ_6 = modification factor, for strength in shear, to account for edge distances smaller than $1.5c_1$ (see D.6.2.6)

ψ_7 = modification factor, for strength in shear, to account for cracking (see D.6.2.7)

SECTION D.1
DEFINITIONS

Anchor. A steel element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads, including headed bolts, nooked bolts (J- or L-bolt), headed studs, expansion anchors, or undercut anchors.

Anchor group. A number of anchors of approximately equal effective embedment depth with each anchor spaced at less than three times its embedment depth from one or more adjacent anchors.

Anchor pullout strength. The strength corresponding to the anchoring device or a major component of the device sliding out from the concrete without breaking out a substantial portion of the surrounding concrete.

Attachment. The structural assembly, external to the surface of the concrete, that transmits loads to or receives loads from the anchor.

Brittle steel element. An element with a tensile test elongation of less than 14 percent, or reduction in area of less than 30 percent, or both.

Cast-in anchor. A headed bolt, headed stud, or hooked bolt installed before placing concrete.

Concrete breakout strength. The strength corresponding to a volume of concrete surrounding the anchor or group of anchors separating from the member.

Concrete pryout strength. The strength corresponding to formation of a concrete spall behind a short, stiff anchor with an embedded base that is displaced in the direction opposite to the applied shear force.

Distance sleeve. A sleeve that encases the center part of an undercut anchor, a torque-controlled expansion anchor, or a displacement-controlled expansion anchor, but does not expand.

Ductile steel element. An element with a tensile test elongation of at least 14 percent and reduction in area of at least 30 percent. A steel element meeting the requirements of ASTM A 307 shall be considered ductile.

Edge distance. The distance from the edge of the concrete surface to the center of the nearest anchor.

Effective embedment depth. The overall depth through which the anchor transfers force to or from the surrounding concrete. The effective embedment depth will normally be the depth of the concrete failure surface in tension applications. For cast-in headed anchor bolts and headed studs, the effective embedment depth is measured from the bearing contact surface of the head. (See Fig. RD.0)

Expansion anchor. A post-installed anchor, inserted into hardened concrete that transfers loads *to* or from the concrete by direct bearing or friction or both. Expansion anchors may be torque-controlled, where the expansion is achieved by a torque acting on the screw or bolt; or displacement-controlled, where the expansion is achieved by impact forces acting on a sleeve or plug and the expansion is controlled by the length of travel of the sleeve or plug.

Expansion sleeve. The outer part of an expansion anchor that is forced outward by the center part, either by applied torque or impact, to bear against the sides of the predrilled hole.

Five percent fractile. A statistical term meaning 90 percent confidence that there is 95 percent probability of the actual strength exceeding the nominal strength.

Hooked bolt. A cast-in anchor anchored mainly by mechanical interlock from the 90-deg bend (L-bolt) or 180-deg bend (J-bolt) at its lower end, having a minimum e_h of $3d_o$.

Headed stud. A steel anchor conforming to the requirements of AWS D1.1 and affixed to a plate or similar steel attachment by the stud arc welding process before casting.

Post-installed anchor. An anchor installed in hardened concrete. Expansion anchors and undercut anchors are examples of post-installed anchors.

Projected area. The area on the free surface of the concrete member that is used to represent the larger base of the assumed rectilinear failure surface.

Side-face blowout strength. The strength of anchors with deeper embedment but thinner side cover corresponding to concrete spalling on the side face around the embedded head while no major breakout occurs at the top concrete surface.

Specialty insert. Predesigned and prefabricated cast-in anchors specifically designed for attachment of bolted or slotted connections. Specialty inserts are often used for handling, transportation, and erection, but are also used for anchoring structural elements. Specialty inserts are not within the scope of this appendix.

Supplementary reinforcement. Reinforcement proportioned to tie a potential concrete failure prism to the structural member.

Undercut anchor. A post-installed anchor that develops its tensile strength from the mechanical interlock provided by undercutting of the concrete at the embedded end of the anchor. The undercutting is achieved with a special drill before installing the anchor or alternatively by the anchor itself during its installation.

SECTION D.2
SCOPE

D.2.1 This appendix provides design requirements for anchors in concrete used to transmit structural loads by means of tension, shear, or a combination of tension and shear between (a) connected structural elements; or (b) safety-related attachments and structural elements. Safety levels specified are intended for in-service conditions, rather than for short-term handling and construction conditions.

D.2.2 This appendix applies to both cast-in anchors and post-installed anchors. Specialty inserts, through bolts, multiple anchors connected to a single steel plate at the embedded end of the anchors, adhesive or grouted anchors, and direct anchors such as powder or pneumatic actuated nails or bolts, are not included. Reinforcement used as part of the embedment shall be designed in accordance with other parts of the code.

D.2.3 Headed studs and headed bolts having a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal or exceeding $1.4N_p$ (where N_p is given by Eq. (D-13)) are included. Hooked bolts that have a geometry that has been demonstrated to result in a pullout strength without the benefit of friction in uncracked concrete equal or exceeding $1.4N_p$ (where N_p is given by Eq. (D-14)) are included. Post-installed anchors that meet the assessment requirements of ACI 355.2 are included. The suitability of the post-installed anchor for use in concrete shall have been demonstrated by the ACI 355.2 prequalification tests.

D.2.4 Load applications that are predominantly high cycle fatigue or impact loads are not covered by this appendix.

SECTION D.3
GENERAL REQUIREMENTS

D.3.1 Anchors and anchor groups shall be designed for critical effects of factored loads as determined by elastic analysis. Plastic analysis approaches are permitted where nominal strength is controlled by ductile steel elements, provided that deformational compatibility is taken into account.

D.3.2 The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in 9.2.

D.3.3 When anchor design includes seismic loads, the additional requirements of D.3.3.1 through D.3.3.5 shall apply.

D.3.3.1 The provisions of Appendix D do not apply to the design of anchors in plastic hinge zones of concrete structures under seismic loads.

D.3.3.2 In regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories, post-installed structural anchors for use under D.2.3 shall have passed the Simulated Seismic Tests of ACI 355.2.

D.3.3.3 In regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories, the design strength of anchors shall be taken as $0.75\phi N_n$ and $0.75\phi V_n$ where ϕ is given in D.4.4 or D.4.5 and N_n and V_n are determined in accordance with D.4.1.

D.3.3.4 In regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories, anchors shall be designed to be governed by tensile or shear strength of a ductile steel element, unless D.3.3.5 is satisfied.

D.3.3.5 Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

D.3.4 All provisions for anchor axial tension and shear strength apply to normal weight concrete. When lightweight aggregate concrete is used, provisions for N_n and V_n shall be modified by multiplying all values of $\sqrt{f_c^{'}}$ affecting N_n and V_n by 0.75 for all-lightweight concrete and 0.85 for sand-lightweight concrete. Linear interpolation shall be permitted when partial sand replacement is used.

D.3.5 The values of $f_c^{'}$ used for calculation purposes in this appendix shall not exceed 70 MPa for cast-in anchors, and 55 MPa for post-installed anchors. Testing is required for post-installed anchors when used in concrete with $f_c^{'}$ greater than 55 MPa.

SECTION D.4
GENERAL REQUIREMENTS FOR STRENGTH OF ANCHORS

D.4.1 Strength design of anchors shall be based either on computation using design models that satisfy the requirements of D.4.2, or on test evaluation using the 5 percent fractile of test results for the following:

- **(a)** steel strength of anchor in tension (D.5.1);
- **(b)** steel strength of anchor in shear (D.6.1);
- **(c)** concrete breakout strength of anchor in tension (D.5.2) ;
- **(d)** concrete breakout strength of anchor in shear (D.6.2);
- **(e)** pullout strength of anchor in tension (D.5.3);
- **(f)** concrete side-face blowout strength of anchor in tension (D.5.4); and
- **(g)** concrete pryout strength of anchor in shear (D.6.3).

In addition, anchors shall satisfy the required edge distances, spacings, and thicknesses to preclude splitting failure, as required in D.8.

D.4.1.1 For the design of anchors, except as required in D.3.3,

$$\phi N_n \geq N_u \qquad \text{(D-1)}$$

$$\phi V_n \geq V_u \qquad \text{(D-2)}$$

D.4.1.2 In Eq. (D-1) and (D-2), ϕN_n and ϕV_n are the lowest design strengths determined from all appropriate failure modes. ϕN_n is the lowest design strength in tension of an anchor or group of anchors as determined from consideration of ϕN_s,

ϕnN_{pn} either ϕN_{sb} or ϕN_{sbg} and either ϕN_{cb} or ϕN_{cbg}. ϕV_n is the lowest design strength in shear of an anchor or a group of anchors as determined from consideration of ϕV_s either ϕV_{cb} or ϕV_{cbg} and ϕV_{cp}.

D.4.1.3 When both N_u and V_u are present, interaction effects shall be considered in accordance with D.4.3.

D.4.2 The nominal strength for any anchor or group of anchors shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests. The materials used in the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5 percent fractile of the basic individual anchor strength. For nominal strengths related to concrete strength, modifications for size effects, the number of anchors, the effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loadings of anchor groups, and presence or absence of cracking shall be taken into account. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

D.4.2.1 The effect of supplementary reinforcement provided to confine or restrain the concrete breakout, or both, shall be permitted to be included in the design models used to satisfy D.4.2.

D.4.2.2 For anchors with diameters not exceeding 50 mm, and tensile embedments not exceeding 635 mm in depth, the concrete breakout strength requirements shall be considered satisfied by the design procedure of D.5.2 and D.6.2.

D.4.3 Resistance to combined tensile and shear loads shall be considered in design using an interaction expression that results in computation of strength in substantial agreement with results of comprehensive tests. This requirement shall be considered satisfied by D.7.

D 4.4 Strength reduction factor for anchors in concrete shall be as follows:

a) Anchor governed by strength of a ductile steel element

i)	Tension loads	0.75
ii)	Shear loads	0.65

b) Anchor governed by strength of a brittle steel element

i)	Tension loads	0.65
ii)	Shear loads	0.60

c) Anchor governed by concrete breakout, side-face blowout, pullout, or pryout strength

	Condition A	Condition B
i) Shear loads	0.75	0.70
ii) Tension loads		
Cast-in headed studs, headed bolts, or hooked bolts	0.75	0.70
Post-installed anchors with category as determined from ACI 355.2		
Category 1 (Low sensitivity to installation and high reliability)	0.75	0.65
Category 2 (Medium sensitivity to installation	0.65	0.55

and medium reliability)		
Category 3 (High sensitivity to installation and lower reliability)	0.55	0.45

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.
Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.

SECTION D.5
DESIGN REQUIREMENTS FOR TENSILE LOADING

D.5.1 **Steel strength of anchor in tension**

D.5.1.1 The nominal strength, N_s of an anchor in tension as governed by the steel shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

D.5.1.2 The nominal strength N_s of an anchor or group of anchors in tension shall not exceed:

$$N_s = nA_{se}f_{ut} \qquad \text{(D-3)}$$

where f_{ut} shall not be taken greater than $1.9f_y$ or 860 MPa.

D.5.2 **Concrete breakout strength of anchor in tension**

D.5.2.1 The nominal concrete breakout strength, N_{cb} or N_{cbg}, of an anchor or group of anchors in tension shall not exceed:

for a single anchor:

$$N_{cb} = \frac{A_N}{A_{NO}}\psi_2\psi_3N_b \qquad \text{(D-4)}$$

for a group of anchors:

$$N_{cbg} = \frac{A_N}{A_{NO}}\psi_1\psi_2\psi_3N_b \qquad \text{(D-5)}$$

A_N is the projected area of the failure surface for the anchor or group of anchors that shall be approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward $1.5h_{ef}$ from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors. A_N shall not exceed nA_{No} where n is the number of tensioned anchors in the group. A_{No} is the projected area of the failure surface of a single anchor remote from edges:

$$A_{No} = 9h_{ef}^2 \qquad \text{(D-6)}$$

D.5.2.2 The basic concrete breakout strength N_b of a single anchor in tension in cracked concrete shall not exceed

$$N_b = k\sqrt{f_c'}\ h_{ef}^{1.5} \qquad \text{(D-7)}$$

where

k = 10 for cast-in anchors; and

k = 7 for post-installed anchors.

Alternatively, for cast-in headed studs and headed bolts with $280\,mm \le h_{ef} \le 635\,mm$, the basic concrete breakout strength of a single anchor in tension in cracked concrete shall not exceed

$$N_b = (3.8)\sqrt{f_c'}\;\; h_{ef}^{5/3} \tag{D-8}$$

D.5.2.3 For the special case of anchors in an application with three or four edges along with the largest edge distance $c_{\max} < 1.5h_{ef}$, the embedment depth h_{ef} used in Eq. (D-6) through (D-11) shall be limited to $c_{\max}/1.5$.

D.5.2.4 The modification factor for eccentrically loaded anchor groups is:

$$\psi_1 = \frac{1}{\left(1 + \dfrac{2e_N'}{3h_{ef}}\right)} \le 1 \tag{D-9}$$

Eq. (D-9) is valid for $e_N' \le s/2$.

If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension shall be considered when determining the eccentricity e_N' for use in Eq. (D-9).

In the case where eccentric loading exists about two axes, the modification factor ψ_1 shall be computed for each axis individually and the product of these factors used as ψ_1 in Eq. (D-5).

D.5.2.5 The modification factor for edge effects is:

$$\psi_2 = 1 \;\; if \;\; \mathrm{c}_{\min} \ge 1.5h_{ef} \tag{D-10}$$

$$\psi_2 = 0.7 + 0.3\frac{c_{\min}}{1.5h_{ef}} \;\; \text{if } \mathrm{c}_{\min} < 1.5h_{ef} \tag{D-11}$$

D.5.2.6 When an anchor is located in a region of a concrete member where analysis indicates no cracking ($f_t < f_r$) at service load levels, the following modification factor shall be permitted:

ψ_3 = 1.25 for cast-in anchors.

ψ_3 = 1.4 for post-installed anchors.

When analysis indicates cracking at service load levels, ψ_3 shall be taken as 1.0 for both cast-in anchors and post-installed anchors. Post-installed anchors shall be qualified for use in cracked concrete in accordance with ACI 355.2. The cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with 10.6.4, or equivalent crack control shall be provided by confining reinforcement.

D.5.2.7 When an additional plate or washer is added at the head of the anchor, it shall be permitted to calculate the projected area of the failure surface by projecting the failure surface outward $1.5h_{ef}$ from the effective perimeter of the plate or washer. The effective perimeter shall not exceed the value at a section projected outward more than t from the outer edge of the head of the anchor, where t is the thickness of the washer or plate.

D.5.3 Pullout strength of anchor in tension

D.5.3.1 The nominal pullout strength N_{pn} of an anchor in tension shall not exceed:

$$N_{pn} = \psi_4 N_p \qquad \text{(D-12)}$$

D.5.3.2 For post-installed expansion and undercut anchors, the values of N_p shall be based on the 5 percent fractile of results of tests performed and evaluated according to ACI 355.2. It is not permissible to calculate the pullout strength in tension for such anchors.

D.5.3.3 For single cast-in headed studs and headed bolts, it shall be permitted to evaluate the pullout strength in tension using D.5.3.4. For single J- or L-bolts, it shall be permitted to evaluate the pullout strength in tension using D.5.3.5. Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

D.5.3.4 The pullout strength in tension of a single headed stud or headed bolt N_p for use in Eq. (D-12), shall not exceed:

$$N_p = A_{brg} 8 f_c^{'} \qquad \text{(D-13)}$$

D.5.3.5 The pullout strength in tension of a single hooked bolt N_p for use in Eq. (D-12) shall not exceed:

$$N_p = 0.9 f_c^{'} e_h d_o \qquad \text{(D-14)}$$

where: $3d_o \le e_h \le 4.5d_o$

D.5.3.6 For an anchor located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ at service load levels, the following modification factor shall be permitted

$\psi_4 = 1.4$

Otherwise,

ψ_4 shall be taken as 1.0.

D.5.4 Concrete side-face blowout strength of a headed anchor in tension

D.5.4.1 For a single headed anchor with deep embedment close to an edge $(c < 0.4h_{ef})$ the nominal side-face blowout strength N_{sb} shall not exceed:

$$N_{sb} = (13.3c)\sqrt{A_{brg}}\sqrt{f_c^{'}} \qquad \text{(D-15)}$$

If the single headed anchor is located at a perpendicular distance, c_2, less than $3c$ from an edge, the value of N_{sb} shall be multiplied by the factor $(1 + c_2 / c)/4$ where $1 \le c_2 / c \le 3$.

D.5.4.2 For multiple headed anchors with deep embedment close to an edge $(c < 0.4h_{ef})$ and spacing between anchors less than $6c$, the nominal strength of the group of anchors for a side-face blowout failure N_{sbg} shall not exceed:

$$N_{sbg} = \left(1 + \frac{s_o}{6c}\right) N_{sb} \qquad \text{(D-16)}$$

where s_o = spacing of the outer anchors along the edge in the group; and N_{sb} is obtained from Eq. (D-15) without modification for a perpendicular edge distance.

SECTION D.6
DESIGN REQUIREMENTS FOR SHEAR LOADING

D.6.1 Steel strength of anchor in shear

D.6.1.1 The nominal strength of an anchor in shear as governed by steel V_s shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

D.6.1.2 The nominal strength V_s of an anchor or group of anchors in shear shall not exceed (a) through (c):

(a) for cast-in headed stud anchors

$$V_s = nA_{se} f_{ut} \qquad \text{(D-17)}$$

where f_{ut} shall not be taken greater than $1.9 f_y$ or 860 MPa

(b) for cast-in headed bolt and hooked bolt anchors

$$V_s = 0.6\, nA_{se} f_{ut} \qquad \text{(D-18)}$$

where f_{ut} shall not be taken greater than $1.9 f_y$ or 860 MPa.

(c) for post-installed anchors

$$V_s = n(0.6 A_{se} f_{ut} + 0.4 A_{si} f_{utsi}) \qquad \text{(D-19)}$$

where f_{ut} shall not be taken greater than $1.9 f_y$ or 860 MPa.

D.6.1.3 Where anchors are used with built-up grout pads, the nominal strengths of D.6.1.2 shall be multiplied by a 0.80 factor.

D.6.2 Concrete breakout strength of anchor in shear

D.6.2.1 The nominal concrete breakout strength, V_{cb} or V_{cbg} in shear of an anchor or group of anchors shall not exceed:

(a) for shear force perpendicular to the edge on a single anchor:

$$V_{cb} = \frac{A_v}{A_{vo}} \psi_6 \psi_7 V_b \qquad \text{(D-20)}$$

(b) for shear force perpendicular to the edge on a group of anchors:

$$V_{cbg} = \frac{A_v}{A_{vo}} \psi_5 \psi_6 \psi_7 V_b \qquad \text{(D-21)}$$

(c) for shear force parallel to an edge, V_{cb} or V_{cbg} shall be permitted to be twice the value for shear force determined from Eq. (D-20) or (D-21), respectively, with ψ_6 taken equal to 1.

(d) for anchors located at a corner, the limiting nominal concrete breakout strength shall be determined for each edge, and the minimum value shall be used.

V_b is the basic concrete breakout strength value for a single anchor. A_v is the projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. It shall be permitted to evaluate this area as the base of a truncated half pyramid projected on the side face of the member where the top of the half pyramid is given by the axis of the anchor row selected as critical. The value of c_1 shall be taken as the distance from the edge to this axis. A_v shall not exceed nA_{Vo} where n is the number of anchors in the group.

A_{Vo} is the projected area for a single anchor in a deep member and remote from edges in the direction perpendicular to the shear force. It shall be permitted to evaluate this area as the base of a half pyramid with a side length parallel to the edge of $3c_1$ and a depth of $1.5c_1$:

$$A_{vo} = 4.5(c_1)^2 \qquad \text{(D-22)}$$

Where anchors are located at varying distances from the edge and the anchors are welded to the attachment so as to distribute the force to all anchors, it shall be permitted to evaluate the strength based on the distance to the farthest row of anchors from the edge. In this case, it shall be permitted to base the value of c_1 on the distance from the edge to the axis of the farthest anchor row that is selected as critical, and all of the shear shall be assumed to be carried by this critical anchor row alone.

D.6.2.2 The basic concrete breakout strength V_b in shear of a single anchor in cracked concrete shall not exceed

$$V_b = (0.6)\left(\frac{\ell}{d_o}\right)^{0.2} \sqrt{d_o}\sqrt{f_c'}\ (c_1)^{1.5} \qquad \text{(D-23)}$$

D.6.2.3 For cast-in headed studs, headed bolts, or hooked bolts that are continuously welded to steel attachments having a minimum thickness equal to the greater of 10 mm or half of the anchor diameter, the basic concrete breakout strength V_b in shear of a single anchor in cracked concrete shall not exceed

$$V_b = (0.66)\left(\frac{\ell}{d_o}\right)^{0.2} \sqrt{d_o}\sqrt{f_c'}\ (c_1)^{1.5} \qquad \text{(D-24)}$$

provided that:

(a) for groups of anchors, the strength is determined based on the strength of the row of anchors farthest from the edge;

(b) the center-to-center spacing of the anchors is not less than 65 mm; and

(c) supplementary reinforcement is provided at the corners if $c_2 \le 1.5h_{ef}$.

D.6.2.4 For the special case of anchors influenced by three or more edges, the edge distance c_1 used in Eq. (D-22), (D-23), (D-24), (D-25), (D-26) and (D-27) shall be limited to $h/1.5$.

D.6.2.5 The modification factor for eccentrically loaded anchor groups is

$$\psi_5 = \frac{1}{1+\frac{2e_v'}{3c_1}} \leq 1 \qquad \text{(D-25)}$$

Equation (D-25) is valid for $e_v' \leq s/2$

D.6.2.6 The modification factor for edge effect is

$$\psi_6 = 1.0 \qquad \text{(D-26)}$$

if $c_2 \geq 1.5c_1$

$$\psi_6 = 0.7 + 0.3\frac{c_2}{1.5c_1} \qquad \text{(D-27)}$$

if $c_2 < 1.5c_1$

D.6.2.7 For anchors located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ at service loads, the following modification factor shall be permitted

$\psi_7 = 1.4$ for anchors located in a region of a concrete member where analysis indicates cracking at service load levels, the following modification factors shall be permitted:

$\psi_7 = 1.0$ for anchors in cracked cocrete with no supplimintary reinforcement or edge reinforcement smaller than a Dia 12 mm bar;

$\psi_7 = 1.2$ for anchors in cracked concrete with supplementary reinforcement of a Dia 12 mm bar or greater between the anchor and the edge; and

$\psi_7 = 1.4$ for anchors in cracked concrete with supplementary reinforcement of a Dia 12 mm bar or greater between the anchor and the edge, and with the supplementary reinforcement enclosed within stirrups spaced at not more than 100 mm.

D.6.3 Concrete pryout strength of anchor in shear

D.6.3.1 The nominal pryout strength V_{cp} shall not exceed

$$V_{cp} = k_{cp}N_{cb} \qquad \text{(D-28)}$$

where

$k_{cp} = 1.0$ for $h_{ef} < 65$ mm

$k_{cp} = 2.0$ for $h_{ef} \geq 65$ mm

and N_{cb} shall be determined from Eq. (D-4), N.

SECTION D.7
INTERACTION OF TENSILE AND SHEAR FORCES

Unless determined in accordance with D.4.3, anchors or groups of anchors that are subjected to both shear and axial loads shall be designed to satisfy the requirements of D.7.1 through D.7.3. The value of ϕN_n shall be as required in D.4.1.2. The value of ϕV_n shall be as defined in D.4.1.2.

D.7.1 If $V_u \leq 0.2\phi V_n$, then full strength in tension shall be permitted: $\phi N_n \geq N_u$.

D.7.2 If $N_u \leq 0.2\phi N_n$, then full strength in shear shall be permitted: $\phi V_n \geq V_u$.

D.7.3 If $V_u > 0.2\phi V_n$ and $N_u > 0.2\phi N_n$, then:

$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \leq 1.2 \qquad \text{(D-29)}$$

SECTION D.8
REQUIRED EDGE DISTANCES, SPACINGS, AND THICKNESSES TO PRECLUDE SPLITTING FAILURE

Minimum spacings and edge distances for anchors and minimum thicknesses of members shall conform to D.8.1 through D.8.5, unless supplementary reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with ACI 355.2 shall be permitted.

D.8.1 Unless determined in accordance with D.8.4, minimum center-to-center spacing of anchors shall be $4d_o$ for untorqued cast-in anchors, and $6d_o$ for torqued cast-in anchors and post-installed anchors.

D.8.2 Unless determined in accordance with D.8.4, minimum edge distances for cast-in headed anchors that will not be torqued shall be based on minimum cover requirements for reinforcement in 7.7. For cast-in headed anchors that will be torqued, the minimum edge distances shall be $6d_o$.

D.8.3 Unless determined in accordance with D.8.4, minimum edge distances for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement in 7.7, or the minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2, and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific ACI 355.2 test information, the minimum edge distance shall be taken as not less than:

Undercut anchors…………………………………………. $6d_o$

Torque-controlled anchors…………………………...…… $8d_o$

Displacement-controlled anchors………………….……… $10d_o$

D.8.4 For anchors where installation does not produce a splitting force and that will remain untorqued, if the edge distance or spacing is less than those specified in D.8.1 to D.8.3, calculations shall be performed by substituting for d_o a smaller value d_o' that meets the requirements of D.8.1 to D.8.3. Calculated forces applied to the anchor shall be limited to the values corresponding to an anchor having a diameter of d_o'.

D.8.5 The value of h_{ef} for an expansion or undercut post-installed anchor shall not exceed the greater of either 2/3 of the member thickness or the member thickness less 100 mm.

D.8.6 Project drawings and project specifications shall specify use of anchors with a minimum edge distance as assumed in design.

SECTION D.9
INSTALLATION OF ANCHORS

D.9.1 Anchors shall be installed in accordance with the project drawings and project specifications.

APPENDIX E
NOTATIONS

CODE NOTATION

a = depth of equivalent rectangular stress block as defined in 10.2.7.1, mm, Chapters 10, 12

a = shear span, distance between concentrated load and face of support, mm, Chapter 11

a = shear span, equal to the distance between a load and a support in a structure, mm, Appendix-A

A = area of that part of cross section between flexural tension face and center of gravity of gross section, mm², Chapter 18

$\mathbf{A_b}$ = area of an individual horizontal bar or wire, mm², Chapter 10

$\mathbf{A_b}$ = area of an individual bar, mm², Chapter 12

$\mathbf{A_{brg}}$ = bearing area of the head of stud or anchor bolt, mm², Appendix D

$\mathbf{A_c}$ = area of core of spirally reinforced compression member measured to outside diameter of spiral, mm², Chapter 10

$\mathbf{A_c}$ = area of concrete section resisting shear transfer, mm², Chapter 11

$\mathbf{A_c}$ = area of contact surface being investigated for horizontal shear, mm², Chapter 17

$\mathbf{A_c}$ = the effective cross-sectional area at one end of a strut in a strut-and-tie model, taken perpendicular to the axis of the strut, mm², Appendix A

$\mathbf{A_{cf}}$ = larger gross cross-sectional area of the slab-beam strips of the two orthogonal equivalent frames intersecting at a column of a two-way slab, mm², Chapter 18

$\mathbf{A_{ch}}$ = cross-sectional area of a structural member measured out-to-out of transverse reinforcement, mm², Chapter 21

$\mathbf{A_{cp}}$ = area enclosed by outside perimeter of concrete cross section, mm². See 11.6.1, Chapter 11

$\mathbf{A_{cp}}$ = area of concrete section, resisting shear, of an individual pier or horizontal wall segment, mm², Chapter 21

$\mathbf{A_{cv}}$ = gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, mm², Chapter 21

$\mathbf{A_f}$ = area of reinforcement in bracket or corbel resisting factored moment, $\left[V_u \mathrm{a} + \mathrm{N_{uc}}(h-d)\right]$, mm²,Chapter 11

$\mathbf{A_g}$ = gross area of column, mm², Chapter 16

$\mathbf{A_g}$ = gross area of section, mm². For a hollow section, $\mathrm{A_g}$ is the area of the concrete only and does not include the area of the void(s). See11.6.1, Chapter 11

$\mathbf{A_g}$ = gross area of section, mm², Chapters 9, 10,14,21, 15, Appendix B

$\mathbf{A_j}$ = effective cross-sectional area within a joint, see 21.5.3.1, in a plane parallel to plane of reinforcement generating shear in the joint, mm². The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of:
(a) beam width plus the joint depth
(b) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. See 21.53.1, Chapter 21

$\mathbf{A_h}$ = area of shear reinforcement parallel to flexural tension reinforcement, mm², Chapter 11

$\mathbf{A_\ell}$ = total area of longitudinal reinforcement to resist torsion, mm², Chapter 11

A_n = area of reinforcement in bracket or corbel resisting tensile force N_{uc} mm², Chapter 11

A_n = area of a face of a nodal zone or a section through a nodal zone, mm², Appendix A

A_N = projected concrete failure area of an anchor or group of anchors, for calculation of strength in tension, as defined in D.5.2.1, mm². A_N shall not be taken greater than nA_{No}. [See Fig. RD.5.1(b) in SBC 304C], Appendix D

A_{No} = projected concrete failure area of one anchor, for calculation of strength in tension when not limited by edge distance or spacing, as defined in D.5.2.1, mm². [See Fig. RD.5.1(a) in SBC 304C], Appendix D

A_o = gross area enclosed by shear flow path, mm², Chapter 11

A_{oh} = area enclosed by centerline of the outermost closed transverse torsional reinforcement, mm², Chapter 11

A_{ps} = area of prestressed reinforcement in a tie, mm², Appendix A

A_{ps} = area of prestressed reinforcement in tension zone, mm², Chapters 11, 18

A_s = area of longitudinal tension reinforcement in wall segment, mm^2, Chapter 14

A_s = area of nonprestressed tension reinforcement, mm², Chapters 8, 10, 11, 12 and 18

A_s = area of tension reinforcement, mm², Appendix B

A_s' = area of compression reinforcement, mm², Chapters 8, 9, 18, Appendix B

A_s' = area of compression reinforcement in a strut, mm², Appendix A

A_{se} = area of effective longitudinal tension reinforcement in wall segment, mm², as calculated by Eq. (14-8), Chapter 14

A_{se} = effective cross-sectional area of anchor, mm², Appendix D

A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing "s" and perpendicular to dimension h_c mm², Chapter 21

A_{si} = area of surface reinforcement in the i th layer crossing a strut, mm², Appendix A

A_{sl} = effective cross-sectional area of expansion or undercut anchor sleeve, if sleeve is within shear plane, mm², Appendix D

$A_{s,min}$ = minimum amount of flexural reinforcement, mm². See 10.5, Chapter 10

A_{st} = total area of longitudinal reinforcement, (bars or steel shapes), mm², Chapter 10

A_{st} = area of nonprestressed reinforcement in a tie, mm², Appendix A

A_t = area of structural steel shape, pipe, or tubing in a composite section, mm², Chapter 10

A_t = area of one leg of a closed stirrup resisting torsion within a distance, "s" mm², Chapter 11

A_{tr} = total cross-sectional area of all transverse reinforcement which is within the spacing "s" and which crosses the potential plane of splitting through the reinforcement being developed, mm², Chapter 12

A_v = area of shear reinforcement within a distance "s" or area of shear reinforcement perpendicular to flexural tension reinforcement within a distance "s" for deep flexural members, mm², Chapter 11

A_v = area of shear reinforcement within a distance "s", mm², Chapter 12

A_v = area of ties within a distance"s", mm², Chapter 17

A_V = projected concrete failure area of an anchor or group of anchors, for calculation of strength in shear, as defined in D.6.2.1, mm². A_V shall not be taken greater than nA_{Vo}. [See Fig. RD.6.2(b) in SBC 304C], Appendix D

A_{vd} = total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam, mm², Chapter 21

A_{vf} = area of shear-friction reinforcement, mm², Chapter 11

A_{vh} = area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2, mm², Chapter 11

A_{Vo} = projected concrete failure area of one anchor, for calculation of strength in shear, when not limited by corner influences, spacing, or member thickness, as defined in D.6.2.1, mm². [See Fig. RD.6.2(a) in SBC 304C], Appendix D

A_w = area of an individual wire to be developed or spliced, mm², Chapter 12

A_1 = loaded area, mm², Chapter 10

A_2 = the area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal, mm^2, Chapter 10

b = width of compression face of member, mm, Chapters 8 through 11, 18

b = width of member, mm, Appendix A

b = effective compressive flange width of a structural member, mm, Chapter 21

B_n = nominal bearing load, N

b_o = perimeter of critical section for slabs and footings, mm, Chapter 11

b_t = width of that part of cross section containing the closed stirrups resisting torsion, mm, Chapter 11

b_v = width of cross section at contact surface being investigated for horizontal shear, mm, Chapter 17

b_w = web width, mm, Chapter 10

b_w = web width, or diameter of circular section, mm, Chapters 11, 12, 21

b_1 = width of the critical section defined in 11.12.1.2 measured in the direction of the span for which moments are determined, mm, Chapters 11, 13

b_2 = Width of the critical section defined in 11.12.1.2 measured in the direction perpendicular to b_1, mm, Chapters 11, 13

c = distance from extreme compression fiber to neutral axis, mm, Chapter 9

c = distance from extreme compression fiber to neutral axis, mm, Chapters 10, 14, Appendix A

c = spacing or cover dimension, mm. See 12.2.4, Chapter 12

c = distance from the extreme compression fiber to neutral axis, see 10.2.7, calculated for the factored axial force and nominal moment strength, consistent with the design displacement δ_u, resulting in the largest neutral axis depth, mm, Chapter 21

c = distance from center of an anchor shaft to the edge of concrete, mm, Appendix D

$\boldsymbol{C}$ = cross-sectional constant to define torsional properties

$$C = \sum \left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3}$$

The constant C for T- or L-sections shall be permitted to be evaluated by dividing the section into separate rectangular parts and summing the values of C for each part, Chapter 13

$\mathbf{c_c}$ = clear cover from the nearest surface in tension to the surface of the flexural tension reinforcement, mm, Chapter 10

$\mathbf{c_c}$ = clear cover from the nearest surface in tension to the surface of the flexural tension steel, mm, Chapter 18

$\mathbf{C_m}$ = a factor relating actual moment diagram to an equivalent uniform moment diagram, Chapter 10

$\mathbf{c_{max}}$ = the largest edge distance, mm, Appendix D

$\mathbf{c_{min}}$ = the smallest edge distance, mm, Appendix D

$\mathbf{c_t}$ = dimension equal to the distance from the interior face of the column to the slab edge measured parallel to c_1, but not exceeding c_1, mm, Chapter 21

$\mathbf{c_1}$ = size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm, Chapters 11, 13, 21

$\mathbf{c_1}$ = distance from the center of an anchor shaft to the edge of concrete in one direction, mm; where shear force is applied to anchor, c_1 is in the direction of the shear force. [See Fig. RD.6.2(a) in SBC 304C], Appendix D

$\mathbf{c_2}$ = size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which moments are being determined, mm, Chapters 11, 13

$\mathbf{c_2}$ = distance from center of an anchor shaft to the edge of concrete in the direction orthogonal to c_1 mm, Appendix D.

$\boldsymbol{d}$ = distance from extreme compression fiber to centroid of tension reinforcement, mm, Chapters 7-10, 12, Appendix B

$\boldsymbol{d}$ = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, but need not be less than $0.80h$ for circular sections and prestressed members, mm, Chapter 11

$\boldsymbol{d}$ = distance from extreme compression fiber to centroid of tension reinforcement for entire composite section, mm, Chapter 17

$\boldsymbol{d}$ = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm, Chapter 14

$\boldsymbol{d}$ = distance from extreme compression fiber to centroid of nonprestressed tension reinforcement, mm, Chapter 18

$\boldsymbol{d}$ = effective depth of section, mm, Chapter 21

$\boldsymbol{d}$ = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm, Appendix A

$\boldsymbol{d'}$ = distance from extreme compression fiber to centroid of compression reinforcement, mm, Chapter 18, Appendix B

$\mathbf{D}$ = dead loads or related internal moments and forces, Chapters 9, 18, 20

$\boldsymbol{d_b}$ = nominal diameter of bar, wire, or prestressing strand, mm, Chapters 7, 12

$\boldsymbol{d_b}$ = bar diameter, mm, Chapter 21

$\boldsymbol{d_o}$ = outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, mm. (See also D.8.4), Appendix D

$\boldsymbol{d_o'}$ = value substituted for d_o when an oversized anchor is used, mm. (See D.8.4), Appendix D

d_p = diameter of pile at footing base, mm, Chapter 15

d_p = distance from extreme compression fiber to centroid of prestressed reinforcement, mm, Chapter 18

d_s = distance from extreme tension fiber to centroid of tension reinforcement, mm, Chapter 9, Appendix B

d_t = distance from extreme compression fiber to extreme tension steel, mm, Chapters 9, 10

e = base of Napierian logarithms, Chapter 18

E = load effects of earthquake, or related internal moments and forces, Chapters 9, 21

E_c = modulus of elasticity of concrete, MPa. See 8.5.1, Chapters 8-10, 14, 19

E_{cb} = modulus of elasticity of beam concrete, MPa, Chapter 13

E_{cs} = modulus of elasticity of slab concrete, MPa, Chapter 13

e_h = distance from the inner surface of the shaft of a J- or L-bolt to the outer tip of the J- or L-bolt, mm, Appendix D

EI = flexural stiffness of compression member. See Eq. (10-12) and Eq. (10-13), mm²-N, Chapter 10

e'_N = eccentricity of normal force on a group of anchors; the distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension, mm; e'_N is always positive [See Fig. RD.5.2(b) and (c) in SBC 304C], Appendix D

E_s = modulus of elasticity of reinforcement, MPa. See 8.5.2 and 8.5.3, Chapters 8, 10.

e'_V = eccentricity of shear force on a group of anchors; the distance between the point of shear force application and the centroid of the group of anchors resisting shear in the direction of the applied shear, mm, Appendix D

F = loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces, Chapter 9

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, MPa, Chapters 9, 11, 12, 18, 19, 21

f'_c = specified compressive strength of concrete, MPa, Chapters 4, 5, 8-12, 14, 18-21 Appendixes A, B, D

$\sqrt{f'_{ci}}$ = square root of compressive strength of concrete at time of initial prestress, MPa, Chapter 18

f'_{ci} = compressive strength of concrete at time of initial prestress, MPa, Chapters 7, 18

f'_{cr} = required average compressive strength of concrete used as the basis for selection of concrete proportions, MPa, Chapter 5

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, MPa, Chapters 5, 8, 9, 11,12

f_{ct} = specified tensile strength of concrete, MPa, Appendix D

f_{cu} = effective compressive strength of the concrete in a strut or a nodal zone, MPa, Appendix A

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, MPa, Chapter 11

f_{dc} = decompression stress. Stress in the prestressing steel when stress is zero in the concrete at the same level as the centroid of the tendons, MPa, Chapter 18

F_n = nominal strength of a strut, tie, or nodal zone, N, Appendix A

F_{nn} = nominal strength of a face of a nodal zone, N, Appendix A

F_{ns} = nominal strength of a strut, N, Appendix A

F_{nt} = nominal strength of a tie, N, Appendix A

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, MPa. (in a composite member, f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone), Chapter 11

f_{pc} = average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses), MPa, Chapter 18

f_{pe} = Compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads, MPa, Chapter 11

f_{ps} = stress in prestressed reinforcement at nominal strength, MPa, Chapters 12, 18

f_{pu} = specified tensile strength of prestressing steel, MPa, Chapters 11, 18

f_{py} = specified yield strength of prestressing steel, MPa, Chapter 18

f_r = modulus of rupture of concrete, MPa, see 9.5.2.3, Chapter 18, Appendix D

f_s = calculated stress in reinforcement at service loads, MPa, Chapter 10

f_s' = stress in compression reinforcement, MPa, Appendix A

f_{se} = effective stress in prestressed reinforcement (after allowance for all prestress losses), MPa, Chapter 18

f_{se} = effective stress in prestressed reinforcement (after allowance for all prestress losses), MPa, Chapter 12

f_{se} = effective stress after losses in prestressed reinforcement, MPa, Appendix A

f_t = extreme fiber stress in tension in the precompressed tensile zone, computed using gross section properties, MPa, Chapter 18

f_t = calculated concrete tensile stress in a region of a member, MPa, Appendix D

F_u = factored force acting in a strut, tie, bearing area, or nodal zone in a strut-and-tie model, N, Appendix A

f_{ut} = specified tensile strength of anchor steel, MPa, Appendix D

f_{utsl} = specified tensile strength of anchor sleeve, MPa, Appendix D

f_y = specified yield strength of reinforcement, MPa, Chapter 21

f_y = specified yield strength of anchor steel, MPa, Appendix D

f_y = specified yield strength of nonprestressed reinforcement, MPa, Chapters 3, 7, 12, 14, 18, 19, Appendixes A, B

f_{yh} = specified yield strength of circular tie, hoop, or spiral reinforcement, MPa, Chapter 11

f_{yh} = specified yield strength of transverse reinforcement, MPa, Chapter 21

$f_{y\ell}$ = yield strength of longitudinal torsional reinforcement, MPa, Chapter 11

f_{yt} = specified yield strength of transverse reinforcement, MPa, Chapter 12

f_{yv} = yield strength of closed transverse torsional reinforcement, MPa, Chapter 11

h = overall thickness of member, mm, Chapters 9-14, 17, 18, 20-21, Appendix B

h = thickness of shell or folded plate, mm, Chapter 19

h = height of member, mm, Appendix A

$\boldsymbol{h}$ = thickness of member in which an anchor is anchored, measured parallel to anchor axis, mm, Appendix D

$\boldsymbol{h_t}$ = effective height of tie, mm, Appendix A

$\boldsymbol{H}$ = loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces, Chapter 9

$\boldsymbol{h_c}$ = cross-sectional dimension of column core measured center-to-center of confining reinforcement, mm, Chapter 21

$\boldsymbol{h_{ef}}$ = effective anchor embedment depth, mm. (See D.8.5 and Fig. RD.1 in SBC 304C), Appendix D

$\boldsymbol{h_v}$ = total depth of shearhead cross section, mm, Chapter 11

$\boldsymbol{h_w}$ = total height of wall from base to top, mm, Chapter 11

$\boldsymbol{h_w}$ = height of entire wall or of the segment of wall considered, mm, Chapter 21

$\boldsymbol{h_x}$ = maximum horizontal spacing of hoop or crosstie legs on all faces of the column, mm, Chapter 21

$\boldsymbol{I}$ = moment of inertia of section resisting externally applied factored loads, mm^4,Chapter 11

$\boldsymbol{I_b}$ = moment of inertia about centroidal axis of gross section of beam as defined in 13.2.4, mm^4, Chapter 13

$\boldsymbol{I_{cr}}$ = moment of inertia of cracked section transformed to concrete, mm^4, Chapters 9, 14

$\boldsymbol{I_e}$ = effective moment of inertia for computation of deflection, mm^4, Chapters 9, 14

$\boldsymbol{I_g}$ = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4, Chapters 9, 10

$\boldsymbol{I_s}$ = moment of inertia about centroidal axis of gross section of slab, mm^4
= $h^3/12$ times width of slab defined in notations α and β_t, Chapter 13

$\boldsymbol{I_{se}}$ = moment of inertia of reinforcement about centroidal axis of member cross section, mm^4, Chapter 10

$\boldsymbol{I_t}$ = moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross section, mm^4, Chapter 10

$\boldsymbol{k}$ = effective length factor for compression members, Chapter 10

$\boldsymbol{k}$ = coefficient for basic concrete breakout strength in tension, Appendix D

$\boldsymbol{k}$ = effective length factor, Chapter 14

$\boldsymbol{K}$ = wobble friction coefficient per foot of tendon, Chapter 18

$\boldsymbol{k_{cp}}$ = coefficient for pryout strength, Appendix D

$\boldsymbol{k_t}$ = torsional stiffness of torsional member; moment per unit rotation. (See R13.75 in SBC 304C), Chapter 13

$\boldsymbol{k_{tr}}$ = transverse reinforcement index
$\dfrac{A_{tr} f_{yt}}{1500sn}$ (constant 1500 carries the unit, MPa), Chapter 12

ℓ = span length of beam or one-way slab, as defined in 8.7; clear projection of cantilever, mm, Chapter 9

ℓ = clear span, mm, Chapter 16

ℓ = load bearing length of anchor for shear, not to exceed $8d_o$ mm, Appendix D
= h_{ef} for anchors with a constant stiffness over the full length of the embedded section, such as headed studs or post-installed anchors with one tubular shell over the full length of the embedment depth, Appendix D
= $2d_o$ for torque-controlled expansion anchors with a distance sleeve separated from the expansion sleeve, Appendix D

$\boldsymbol{L}$ = live loads, or related internal moments and forces, Chapters 9, 18, 20

ℓ_a = additional embedment length at support or at point of inflection, mm, Chapter 12

ℓ_c = length of compression member in a frame, measured from center to center of the joints in the frame, mm, Chapter 10

ℓ_c = vertical distance between supports, mm, Chapter 14

ℓ_d = development length, in, Chapters 7, 12

ℓ_d = development length for a straight bar, mm, Chapter 21

ℓ_d = development length of deformed bars and deformed wire in tension, mm, Chapter 12

ℓ_d = development length, mm, Chapter 18

ℓ_{dc} = development length of deformed bars and deformed wire in compression, mm, Chapter 12

ℓ_{dh} = development length of standard hook in tension, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus radius of bend and one bar diameter), mm, Chapter 12

ℓ_{dh} = development length for a bar with a standard hook as defined in Eq. (21-6), mm, Chapter 21

ℓ_{hb} = basic development length of standard hook in tension, mm, Chapter 12

ℓ_n = length of clear span in long direction of two-way construction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases, mm, Chapter 9

ℓ_n = clear span, mm, Appendix A

ℓ_n = length of clear span in direction that moments are being determined, measured face-to-face of supports, mm, Chapter 13

ℓ_n = clear span measured face-to-face of supports, mm, Chapters 11, 21

ℓ_n = clear span for positive moment or shear and average of adjacent clear spans for negative moment, Chapter 8

ℓ_o = minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, mm, Chapter 21

$\boldsymbol{L_r}$ = roof live load, or related internal moments and forces, Chapter 9

ℓ_t = span of member under load test, mm (the shorter span for two-way slab systems). Span is the smaller of (a) distance between centers of supports, and (b) clear distance between supports plus thickness h of member. In Eq. (20-1), span for a cantilever shall be taken as twice the distance from support to cantilever end, Chapter 20

ℓ_u = unsupported length of compression member, mm, Chapter 10

ℓ_v = length of shearhead arm from centroid of concentrated load or reaction, mm, Chapter 11

ℓ_w = horizontal length of wall, mm, Chapters 11, 14

ℓ_w = length of entire wall or of segment of wall considered in direction of shear force, mm, Chapter 21

ℓ_x = length of prestressing steel element from jacking end to any point x, m. See Eq. (18-1) and (18-2), Chapter 18

ℓ_1 = length of span in direction that moments are being determined, measured center-to-center of supports, mm, Chapter 13

ℓ_2 = length of span transverse to l_1 measured center-to-center of supports. See also 13.6.2.3 and 13.6.2.4, mm, Chapter 13

$\boldsymbol{M}$ = maximum unfactored moment due to service loads, including $P\Delta$ effects, N-mm, Chapter 14

$\boldsymbol{M_a}$ = maximum moment in member at stage deflection is computed, N-mm, Chapters 9, 14

$\boldsymbol{M_c}$ = factored moment to be used for design of compression member, N-mm, Chapter 10

$\boldsymbol{M_c}$ = moment at the face of the joint, corresponding to the nominal flexural strength of the column framing into that joint, calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength, N-mm. See 21.4.2.2, Chapter 21

$\boldsymbol{M_{cr}}$ = moment causing flexural cracking at section due to externally applied loads. See 11.4.2.1, Chapters, 11, 14

$\boldsymbol{M_{cr}}$ = cracking moment, N-mm. See 9.5.2.3, Chapter 9

$\boldsymbol{M_g}$ = moment at the face of the joint, corresponding to the nominal flexural strength of the girder including slab where in tension, framing into that joint, N-mm. See 21.4.2.2,Chapter 21

$\boldsymbol{M_m}$ = modified moment, N-mm, Chapter 11

$\boldsymbol{M_{\max}}$ = maximum factored moment at section due to externally applied loads, N-mm, Chapter 11

$\boldsymbol{M_n}$ = nominal moment strength at section, N-mm
$A_s f_y \left(d - \mathrm{a}/2\right)$, Chapter 12

$\boldsymbol{M_n}$ = nominal moment strength at section, N-mm, Chapter14

$\boldsymbol{M_o}$ = total factored static moment, N-mm, Chapter 13

$\boldsymbol{M_p}$ = required plastic moment strength of shearhead cross section, N-mm, Chapter 11

$\boldsymbol{M_{pr}}$ = probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least $1.25 f_y$ and a strength reduction factor ϕ of 1.0, N-mm, Chapter 21

$\boldsymbol{M_s}$ = moment due to loads causing appreciable sway, N-mm, Chapter 10

$\boldsymbol{M_s}$ = portion of slab moment balanced by support moment, N-mm, Chapter 21

$\boldsymbol{M_{sa}}$ = maximum unfactored applied moment due to service loads, not including $P\Delta$ effects, N-mm, Chapter 14

$\boldsymbol{M_u}$ = factored moment at section, N-mm, Chapters 10, 11, 13, 21

$\boldsymbol{M_u}$ = factored moment at section including $P\Delta$ effects, N-mm, Chapter 14

$\boldsymbol{M_{ua}}$ = moment at the midheight section of the wall due to factored lateral and eccentric vertical loads, N-mm, Chapter 14

$\boldsymbol{M_v}$ = moment resistance contributed by shearhead reinforcement, N-mm, Chapter 11

$\boldsymbol{M_1}$ = smaller factored end moment on a compression member, positive if member is bent in single curvature, negative if bent in double curvature, N-mm, Chapter 10

$\boldsymbol{M_{1ns}}$ = factored end moment on a compression member at the end at which M_1 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm, Chapter 10

$\boldsymbol{M_{1s}}$ = factored end moment on compression member at the end at which M_1 acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm, Chapter 10

$\boldsymbol{M_2}$ = larger factored end moment on compression member, always positive N-mm, Chapter 10

$\boldsymbol{M_{2,\min}}$ = minimum value of M_2, N-mm, Chapter 10

$\boldsymbol{M_{2ns}}$ = factored end moment on compression member at the end at which M_2 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm, Chapter 10

$\boldsymbol{M_{2s}}$ = factored end moment on compression member at the end at which M_2 acts, due to loads that cause appreciable sidesway, calculated using a first order elastic frame analysis, N-mm, Chapter 10

$\boldsymbol{n}$ = number of bars or wires being spliced or developed along the plane of splitting, Chapter 12

$\boldsymbol{n}$ = modular ratio of elasticity, but not less than 6
E_s / E_c,Chapter 14

$\boldsymbol{n}$ = number of monostrand anchorage devices in a group, Chapter 18

$\boldsymbol{n}$ = number of anchors in a group, Appendix D

$\boldsymbol{N_b}$ = basic concrete breakout strength in tension of a single anchor in cracked concrete, as defined in D.5.2.2, N, Appendix D

$\boldsymbol{N_c}$ = tensile force in concrete due to unfactored dead load plus live load $(D+L)$, N, Chapter 18

$\boldsymbol{N_{cb}}$ = nominal concrete breakout strength in tension of a single anchor, as defined in D.5.2.1, N, Appendix D

$\boldsymbol{N_{cbg}}$ = nominal concrete breakout strength in tension of a group of anchors, as defined in D.5.2.1, N, Appendix D

$\boldsymbol{N_n}$ = nominal strength in tension, N, Appendix D

$\boldsymbol{N_p}$ = pullout strength in tension of a single anchor in cracked concrete, as defined in D.5.3.4 or D.5.3.5, N, Appendix D

$\boldsymbol{N_{pn}}$ = nominal pullout strength in tension of a single anchor, as defined in D.5.3.1, N, Appendix D

$\boldsymbol{N_s}$ = nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, as defined in D.5.1.1 or D.5.1.2, N, Appendix D

$\boldsymbol{N_{sb}}$ = side-face blowout strength of a single anchor,N, Appendix D

$\boldsymbol{N_{sbg}}$ = side-face blowout strength of a group of anchors, N, Appendix D

$\boldsymbol{N_u}$ = factored tensile load, N, Appendix D

$\boldsymbol{N_u}$ = factored axial load normal to cross section occurring simultaneously with $\mathrm{V_u}$ or T_u to be taken as positive for compression, N, Chapter 11

$\boldsymbol{N_{uc}}$ = factored tensile force applied at top of bracket or corbel acting simultaneously with $\mathrm{V_u}$, to be taken as positive for tension, N, Chapter 11

$\boldsymbol{P_b}$ = nominal axial load strength at balanced strain conditions, N. See 10.3.2, Chapters 9, 10, Appendix B

$\boldsymbol{P_c}$ = critical load, N. See Eq. (10-10), Chapter 10

$\boldsymbol{p_{cp}}$ = outside perimeter of the concrete cross section, *mm.* See 11.6.1, Chapter 11

$\boldsymbol{p_h}$ = perimeter of centerline of outermost closed transverse torsional reinforcement, mm, Chapter 11

$\boldsymbol{P_n}$ = nominal axial load strength at given eccentricity, N, Chapters 9, 10, Appendix B

$\boldsymbol{P_o}$ = nominal axial load strength at zero eccentricity, N, Chapter 10

$\boldsymbol{P_{nw}}$ = nominal axial load strength of wall designed by 14.4, N, Chapter 14

$\boldsymbol{P_s}$ = unfactored axial load at the design (midheight) section including effects of self-weight, N, Chapter 14

P_s = prestressing force at jacking end, N, Chapter 18

P_{su} = factored prestressing force at the anchorage device, N, Chapter 18

P_u = factored axial load at given eccentricity, N
$\leq \phi P_n$ Chapter 10

P_u = factored axial load, N, Chapter 14

P_x = prestressing force at any point x, N, Chapter 18

Q = stability index for a story. See 10.11.4, Chapter 10

r = radius of gyration of cross section of a compression member, mm, Chapter 11

R = rain load, or related internal moments and forces, Chapter 9

s = standard deviation, MPa, Chapter 5

s = spacing of shear or torsion reinforcement in direction parallel to longitudinal reinforcement, mm, Chapter 11

s = maximum center-to-center spacing of transverse reinforcement within ℓ_d, mm, Chapter 12

s = center-to-center spacing of flexural tension reinforcement nearest to the extreme tension face, mm (where there is only one bar or wire nearest to the extreme tension face, "s" is the width of the extreme tension face), Chapter 10

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, mm, Chapter 21

s = spacing of ties measured along the longitudinal axis of the member, mm, Chapter 17

s = center-to-center spacing of flexural tension steel near the extreme tension face, mm. Where there is only one bar or tendon near the extreme tension face, s is the width of extreme tension face, Chapter 18

s = anchor center-to-center spacing, mm, Appendix D

S_e = moment, shear, or axial force at connection corresponding with development of probable strength at intended yield locations, based on the governing mechanism of inelastic lateral deformation, considering both gravity and earthquake load effects, Chapter 21

s_i = spacing of reinforcement in the ith layer adjacent to the surface of the member, mm, Appendix A

S_n = nominal flexural, shear, or axial strength of the connection, Chapter 21

s_o = maximum spacing of transverse reinforcement, mm, Chapter 21

s_o = spacing of the outer anchors along the edge in a group, mm, Appendix D

s_{sk} = spacing of skin reinforcement, mm, Chapter 10

s_w = spacing of wire to be developed or spliced, mm, Chapter 12

s_x = longitudinal spacing of transverse reinforcement within the length ℓ_o mm, Chapter 21

S_y = yield strength of connection, based on f_y for moment, shear, or axial force, Chapter 21

s_1 = spacing of vertical reinforcement in wall, mm, Chapter 11

s_2 = spacing of shear or torsion reinforcement in direction perpendicular to longitudinal reinforcement-or spacing of horizontal reinforcement in wall, mm, Chapter 11

t = thickness of a wall of a hollow section, mm, Chapter 11

t = thickness of washer or plate, mm, Appendix D

$\boldsymbol{T}$ = cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete, Chapter 9
$\boldsymbol{T_n}$ = nominal torsional moment strength, N-mm, Chapter 11
$\boldsymbol{T_u}$ = factored torsional moment at section, N-mm, Chapter 11
$\boldsymbol{U}$ = required strength to resist factored loads or related internal moments and forces, Chapter 9
$\boldsymbol{V_b}$ = basic concrete breakout strength in shear of a single anchor in cracked concrete, as defined in D.6.2.2 or D.6.2.3, N, Appendix D
$\boldsymbol{V_c}$ = nominal shear strength provided by concrete, N, See 11.12.2.1, Chapters 8, 11, 13, 21
$\boldsymbol{V_{cb}}$ = nominal concrete breakout strength in shear of a single anchor, as defined in D.6.2.1, N, Appendix D
$\boldsymbol{V_{cbg}}$ = nominal concrete breakout strength in shear of a group of anchors, as defined in D.6.2.1, N, Appendix D
$\boldsymbol{V_{ci}}$ = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, N, Chapter 11
$\boldsymbol{V_{cp}}$ = nominal concrete pryout strength, as defined in D.6.3, N, Appendix D
$\boldsymbol{V_{cw}}$ = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web, N, Chapter 11
$\boldsymbol{V_d}$ = shear force at section due to unfactored dead load, N, Chapter 11
$\boldsymbol{V_e}$ = design shear force determined from 21.3.4.1 or 21.4.5.1, N, Chapter 21
$\boldsymbol{V_i}$ = factored shear force at section due to externally applied loads occurring simultaneously with $M_{\max}$, N, Chapter 11
$\boldsymbol{V_n}$ = nominal shear strength, N, Chapters 11, 21, Appendix C
$\boldsymbol{v_n}$ = nominal shear stress, MPa. See 11.12.6.2, Chapter 11
$\boldsymbol{V_{nh}}$ = nominal horizontal shear strength, N, Chapter 17
$\boldsymbol{V_p}$ = vertical component of effective prestress force at section, N, Chapter 11
$\boldsymbol{V_s}$ = nominal shear strength provided by shear reinforcement, N, Chapter 11
$\boldsymbol{V_s}$ = nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, as defined in D.6.1.1 or D.6.1.2, N, Appendix D
$\boldsymbol{V_u}$ = factored shear force at section, N, Chapters 11-13, 17, 21
$\boldsymbol{V_u}$ = factored shear load, N, Appendix D
$\boldsymbol{V_u}$ = factored horizontal shear in a story, N, Chapter 10
$\boldsymbol{W}$ = wind load, or related internal moments and forces, Chapter 9
$\boldsymbol{w_c}$ = unit weight of concrete, kg/m^3, Chapters 8, 9
$\boldsymbol{w_d}$ = factored dead load per unit area, Chapter 13
$\boldsymbol{w_\ell}$ = factored live load per unit area, Chapter 13
$\boldsymbol{w_u}$ = factored load per unit length of beam or per unit area of slab, Chapter 8
$\boldsymbol{w_u}$ = factored load per unit area, Chapter 13
$\boldsymbol{x}$ = shorter overall dimension of rectangular part of cross section, mm, Chapter 13
$\boldsymbol{y}$ = longer overall dimension of rectangular part of cross section, mm, Chapter 13
$\boldsymbol{y_t}$ = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, mm, Chapters 9, 11
$\boldsymbol{\alpha}$ = angle between inclined stirrups and longitudinal axis of member, Chapter 11
$\boldsymbol{\alpha}$ = reinforcement location factor. See 12.2.4, Chapter 12

α = ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of the beam, Chapter 13

= $\dfrac{E_{cb}I_b}{E_{cs}I_s}$

α = total angular change of tendon profile in radians from tendon jacking end to any point x, Chapter 18

α = angle between the diagonal reinforcement and the longitudinal axis of a diagonally reinforced coupling beam, Chapter 21

α_c = coefficient defining the relative contribution of concrete strength to wall strength. See Eq. (21-7), Chapter 21

α_f = angle between shear-friction reinforcement and shear plane, Chapter 11

α_m = average value of α for all beams on edges of a panel, Chapter 9

α_s = constant used to compute V_c in slabs and footings, Chapter 11

α_v = ratio of flexural stiffness of shearhead arm to that of the surrounding composite slab section. See 11.12.4.5, Chapter 11

α_1 = α in direction of ℓ_1 Chapter 13

α_2 = α in direction of ℓ_2 Chapter 13

β = ratio of clear spans in long to short direction of two-way slabs, Chapter 9

β = coating factor. See 12.2.4, Chapter 12

β = ratio of long side to short side of footing, Chapter 15

β_b = ratio of area of reinforcement cut off to total area of tension reinforcement at section, Chapter 12

β_1 = factor defined in 10.2.7.3, Chapters 8, 10, 18, Appendixes A, B

β_c = ratio of long side to short side of concentrated load or reaction area, Chapter 11

β_d = (a) for nonsway frames, β_d is the ratio of the maximum factored axial sustained load to the maximum factored axial load associated with the same load combination;

(b) for sway frames, except as required in (c) of this definition, β_d is the ratio of the maximum factored sustained shear within a story to the maximum factored shear in that story; and

(c) for stability checks of sway frames carried out in accordance with 10.13.6, β_d is the ratio of the maximum factored sustained axial load to the maximum factored axial load, Chapter 10

β_n = factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone, Appendix A

β_p = constant used to compute V_c in prestressed slabs, Chapter 11

β_s = factor to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete in a strut, Appendix A

β_t = ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of beam, center-to-center of supports, Chapters 9, 13

= $\dfrac{E_{cb}C}{2E_{cs}I_s}$

γ_i = angle between the axis of a strut and the bars in the ith layer of reinforcement crossing that strut, Appendix A

γ = reinforcement size factor. See 12.2.4, Chapter 12

γ_f = fraction of unbalanced moment transferred by flexure at slab-column connections. See 13.5.3.2, Chapters 11, 13

γ_p = factor for type of prestressing steel, Chapter 18
0.55 for f_{py} / f_{pu} not less than 0.80
0.40 for f_{py} / f_{pu} not less than 0.85
0.28 for f_{py} / f_{pu} not less than 0.90

γ_v = fraction of unbalanced moment transferred by eccentricity of shear at slab-column connections. See 11.12.6.1, Chapters 11, 13
= $1-\gamma_f$

δ_{ns} = moment magnification factor for frames braced against sidesway, to reflect effects of member curvature between ends of compression member, Chapter 10

δ_s = moment magnification factor for frames not braced against sidesway, to reflect lateral drift resulting from lateral and gravity loads, Chapter 10

δ_u = design displacement, mm, Chapter 21

ε_t = net tensile strain in extreme tension steel at nominal strength, Chapters 8-10

$\Delta_{f\max}$ = maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test, mm. See Eq. (20-3), Chapter 20

Δf_p = increase in stress in prestressing steel due to factored loads, MPa, Appendix A

Δf_{ps} = stressing in prestressing steel at service loads less decompression stress, MPa, Chapter 18

$\Delta_{\max}$ = measured maximum deflection, mm. See Eq. (20-l), Chapter 20

Δ_o = relative lateral deflection between the top and bottom of a story due to V_u, computed using a first-order elastic frame analysis and stiffness values satisfying 10.11.1, mm, Chapter 10

$\Delta_{r\max}$ = measured residual deflection, mm. See Eq. (20-2) and (20-3), Chapter 20

Δ_s = maximum deflection at or near midheight due to service loads, mm, Chapter 14

Δ_u = deflection at midheight of wall due to factored loads, mm, Chapter 14

ε_s = the strain in the longitudinal reinforcement in a compression zone or a longitudinally reinforced strut, Appendix A

η = number of identical arms of shearhead, Chapter 11

θ = angle of compression diagonals in truss analogy for torsion, Chapter 11

θ = angle between the axis of a strut or compression field and the tension chord of the member, Appendix A

λ = lightweight aggregate concrete factor. See 12.2.4, Chapter 12

λ = multiplier for additional long-term deflection as defined in 9.5.2.5, Chapter 9

λ = correction factor related to unit weight of concrete. See 11.7.4.3, Chapters 11, 17, 18, Appendix A

μ = curvature friction coefficient, Chapter 18

μ = coefficient of friction. See 11.7.4.3, Chapter 11

ξ = time-dependent factor for sustained load. See 9.5.2.5, Chapter 9

ρ = ratio of nonprestressed tension reinforcement
A_s / bd Chapters 8-11, 13, 18, 21, Appendix B

ρ = ratio of tension reinforcement
$A_s / (\ell_w d)$, Chapter 14

ρ' = ratio of nonprestressed compression reinforcement A_s' / bd, Chapters 8, 9, Appendix B

ρ' = ratio of compression reinforcement A_s' / bd, Chapter 18

ρ_b = Reinforcement ratio producing balanced strain conditions. See 10.3.2, Chapters 8-10, 13, 14, Appendix B

ρ_g = ratio of total reinforcement area to cross-sectional area of column, Chapter 21

ρ_h = ratio of horizontal shear reinforcement area to gross concrete area of vertical section, Chapter 11

ρ_n = ratio of vertical shear reinforcement area to gross concrete area of horizontal section, Chapter 11

ρ_n = ratio of area of distributed reinforcement parallel to the plane of A_{cv} to gross concrete area perpendicular to that reinforcement, Chapter 21

ρ_p = ratio of prestressed reinforcement A_{ps} / bd_p , Chapter 18

ρ_s = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member, Chapter 10

ρ_s = ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out), Chapter 21

ρ_v = ratio of tie reinforcement area to area of contact surface, Chapter 17
= $A_v / b_v s$

ρ_v = ratio of area of distributed reinforcement perpendicular to the plane of A_{cv} to gross concrete area A_{cv} Chapter 21

ρ_w = $A_s / b_w d$, Chapter 11

ϕ = strength reduction factor, Chapters 8-11, 13, 14, 17-19, 21 Appendixes A, B, D

ϕ_k = stiffness reduction factor. See 10.12.3, Chapter 10

ψ_1 = modification factor, for strength in tension, to account for anchor groups loaded eccentrically, as defined in D.5.2.4, Appendix D

ψ_2 = modification factor, for strength in tension, to account for edge distances smaller than $1.5h_{ef}$, as defined in D.5.2.5, Appendix D

ψ_3 = modification factor, for strength in tension, to account for cracking, as defined in D.5.2.6 and D.5.2.7, Appendix D

ψ_4 = modification factor, for pullout strength, to account for cracking, as defined in D.5.3.1 and D.5.3.6, Appendix D

ψ_5 = modification factor, for strength in shear, to account for anchor groups loaded eccentrically, as defined in D.6.2.5, Appendix D

ψ_6 = modification factor, for strength in shear, to account for edge distances smaller than $1.5c_1$,as defined in D.6.2.6, Appendix D

ψ_7 = modification factor, for strength in shear, to account for cracking, as defined in D.6.2.7, Appendix D

ω = $\rho f_y / f_c'$ Chapter 18. Appendix B

ω' = $\rho' f_y / f_c'$, Chapter 18

ω_p = $\rho_p' f_y / f_c'$, Chapter 18

$\omega_w, \omega_{pw}, \omega_w'$ = reinforcement indexes for flanged sections computed as for ω, ω_p and ω' except that "b" shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only, Chapter 18.

APPENDIX F
STEEL REINFORCEMENT INFORMATION

As an aid to users of the SBC 304, information on sizes, areas, and weights of various steel reinforcement are presented.

STANDARD REINFORCING BARS

Bar designation	**Nominal diameter, mm**	**Nominal area, mm^2**	**Nominal mass, kg/m**
Dia 6	6	28	0.222
Dia 8	8	50	0.395
Dia 10	10	79	0.617
Dia 12	12	113	0.888
Dia 14	14	154	1.21
Dia 16	16	201	1.58
Dia 18	18	254	2.00
Dia 20	20	314	2.47
Dia 22	22	380	2.98
Dia 25	25	491	3.85
Dia 28	28	616	4.83
Dia 32	32	804	6.31
Dia 36	36	1018	7.99
Dia 40	40	1257	9.87
Dia 45	45	1590	12.5
Dia 50	50	1963	15.4

ASTM STANDARD PRESTRESSING TENDONS

Type*	Nominal Diameter, mm	Nominal Area mm²	Nominal Mass kg/m
Seven-wire Strand (Grade 1750)	6.35	23.22	0.182
	7.94	37.42	0.294
	9.53	51.61	0.405
	11.11	69.68	0.548
	12.7	92.9	0.73
	15.24	139.35	1.094
Seven-wire Strand (Grade 3290)	9.53	54.84	0.432
	11.11	74.19	0.582
	12.7	98.71	0.775
	15.24	140	1.102
Prestress Wire	4.88	18.7	0.146
	4.98	19.4	0.149
	6.35	32	0.253
	7.01	39	0.298
Prestress Bars (Plain)	19	284	2.23
	22	387	3.04
	25	503	3.97
	29	639	5.03
	32	794	6.21
	35	955	7.52
Prestress Bars (Deformed)	15	181	1.46
	20	271	2.22
	26	548	4.48
	32	806	6.54
	36	1019	8.28

* Availability of some tendon sizes should be investigated in advance.

STANDARD WIRE REINFORCEMENT

Wire designation	Nominal diameter, mm	Nominal area, mm^2	Nominal mass, kg/m	Area (As , mm^2) per meter Center-to-center spacing, mm								
				50	75	100	150	200	250	300	350	400
WD 4.0	4	12.6	0.099	252	168	126	84	63	50	42	36	32
WD 4.5	4.5	15.9	0.125	318	212	159	106	80	64	53	45	40
WD 5.0	5	19.6	0.154	392	261	196	131	98	78	65	56	49
WD 5.5	5.5	23.8	0.187	476	317	238	159	119	95	79	68	60
WD 6.0	6	28.3	0.222	566	377	283	189	142	113	94	81	71
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APPENDIX G
DESIGN AIDS

G.1 Design reference materials illustrating applications of the code requirements may be found in the following documents. The design aids listed may be obtained from the sponsoring organization.

G.1.1 **"ACI Design Handbook,"** ACI Committee 340, Publication SP-17(97), American Concrete Institute, Farmington Hills, Mich., 1997, 482 pp.

G.1.1.1 (Provides tables and charts for design of eccentrically loaded columns by the Strength Design Method. Provides design aids for use in the engineering design and analysis of reinforced concrete slab systems canying loads by two-way action. Design aids are also provided for the selection of slab thickness and for reinforcement required to control deformation and assure adequate shear and flexural strengths.)

G.1.2 **"ACI Detailing Manual-1994,"** ACI Committee 315, Publication SP-66(94), American Concrete Institute, Farmington Hills, Mich., 1994, 244 pp.

G.1.2.1 (Includes the standard, ACI 315-92, and report, ACI 315R-94. Provides recommended methods and standards for preparing engineering drawings, typical details, and drawings placing reinforcing steel in reinforced concrete structures. Separate sections define responsibilities of both engineer and reinforcing bar detailer.)

G.1.3 **"Guide to Durable Concrete (ACI 201.2R-92),"** ACI Committee 201, American Concrete Institute, Farmington Hills, Mich., 1992, 41 pp.

G.1.3.1 (Describes specific types of concrete deterioration. It contains a discussion of the mechanisms involved in deterioration and the recommended requirements for individual components of the concrete, quality considerations for concrete mixtures, construction procedures, and influences of the exposure environment. Section R4.4.1 discusses the difference in chloride-ion limits between ACI 201.2R-92 and the code.)

G.1.4 **"Guide for the Design of Durable Parking Structures (362.1R-97),"**ACI Committee 362, American Concrete Institute, Farmington Hills, Mich., 1997, 40 pp.

G.1.4.1 (Summarizes practical information regarding design of parking structures for durability. It also includes information about design issues related to parking structure construction and maintenance.)

G.1.5 **"CRSI Handbook,"** Concrete Reinforcing Steel Institute, Schaumburg, Ill., 8th Edition, 1996, 960 pp.

G.1.5.1 (Provides tabulated designs for structural elements and slab systems. Design examples are provided to show the basis of and use of the load tables. Tabulated designs are given for beams; square, round and rectangular columns; one-way slabs; and one-way joist construction. The design tables for two-way slab systems include flat plates, flat slabs and waffle slabs. The chapters on foundations provide design tables for square footings, pile caps, drilled piers (caissons) and cantilevered retaining walls. (Other design aids are presented for crack control; and development of reinforcement and lap splices.)

G.1.6 **"Reinforcement Anchorages and Splices,"** Concrete Reinforcing Steel Institute, Schaumberg, Ill., 4th Edition, 1997,100 pp.

G.1.6.1 (Provides accepted practices in splicing reinforcement. The use of lap splices, mechanical splices, and welded splices are described. Design data are presented for development and lap splicing of reinforcement.)

G.1.7 **"Structural Welded Wire Reinforcement Manual of Standard Practice,"** Wire Reinforcement Institute, Findlay, Ohio, 4th Edition, Apr. 1992, 31 pp.

G.1.7.1 (Describes wire fabric material, gives nomenclature and wire size and weight tables. Lists specifications and properties and manufacturing limitations. Contains latest code requirements as code affects welded wire. Also gives development length and splice length tables. Manual contains customary units and soft metric units.)

G.1.8 **"Structural Welded Wire Reinforcement Detailing Manual,"** Wire Reinforcement Institute, Findlay, Ohio, 1994, 252 pp.

G.1.8.1 (Updated with current technical fact sheets inserted. The manual, in addition to including ACI 318 provisions and design aids, also includes: detailing guidance on welded wire reinforcement in one- way and two-way slabs; precast/prestressed concrete components; columns and beams; cast-in-place walls; and slabs-on-ground. In addition, there are tables to compare areas and spacings of high-strength welded wire with conventional reinforcing.)

G.1.9 **"Strength Design of Reinforced Concrete Columns,"** Portland Cement Association, Skokie, Ill., 1978, 48 pp.

G.1.9.1 (Provides design tables of column strength in terms of load in kips versus moment in ft-kips for concrete strength of 5000 psi and Grade 60 reinforcement. Design examples are included. Note that the PCA design tables do not include the strength reduction factor ϕ in the tabulated values; M_u / ϕ and P_u / ϕ must be used when designing with this aid.

G.1.10 **"PCI Design Handbook-Precast and Prestressed Concrete,"** Precast/ Prestressed Concrete Institute, Chicago, 5th Edition, 1999, 630 pp.

G.1.10.1 (Provides load tables for common industry products, and procedures for design and analysis of precast and prestressed elements and structures composed of these elements. Provides design aids and examples.)

G.1.11 **"Design and Typical Details of Connections for Precast and Prestressed Concrete,"** Precast/Prestressed Concrete Institute, Chicago, 2nd Edition, 1988, 270 pp.

G.1.11.1 (Updates available information on design of connections for both structural and architectural products, and presents a full spectrum of typical details. Provides design aids and examples.)

G.1.12 **"PTI Post-Tensioning Manual,"** Post-Tensioning Institute, Phoenix, 5th Edition, 1990, 406 pp.

G.1.12.1 (Provides comprehensive coverage of post-tensioning systems, specifications, and design aid construction concepts.)

G.1.13 **"PTI Design of Post-Tensioned Slabs,"** Post-Tensioning Institute, Phoenix, 2nd Edition, Apr. 1984, 56 pp.

G.1.13.1 (Illustrates application of the code requirements for design of one-way and two-way post-tensioned slabs. Detailed design examples are presented.)

Gratitude

In appreciation and gratitude

to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence

& Aviation and Inspector General

For their continuous support and gracious consideration,

the Saudi Building Code National Committee (SBCNC)

is honored to present the first issue of

the Saudi Building Code (SBC).

PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11th June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Requirements for Concrete Structures (SBC 304) were developed based on ICC code in addition to American Concrete Institute (ACI) materials. ACI grants permission to the SBCNC to include ACI materials in the SBC, and ACI is not responsible for any modifications or changes that SBCNC has made to accommodate local conditions.

The development process of SBC 304 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made on ACI, such as Durability Requirements, the simplified methods for the design of two-way slab system of Appendix C, expanding some topics such as Hot Weather, taking into considerations the properties of local material such as the Saudi steel and the engineering level for those involved in the building sector.

As a follow-up to the *Saudi Building Code*, SBCNC offers a companion document, the *Saudi Building Code Concrete Structural Requirements Commentary* (SBC 304C). The basic appeal of the Commentary is thus: it provides in a small package thorough coverage of many issues likely to be dealt with when using the *Saudi Building Code Steel Structural Requirements* (SBC 304) and then supplements that coverage with technical background. Reference lists, information sources and bibliographies are also included.

Strenuous effort has been made to keep the vast quantity of material accessible and its method of presentation useful. With a comprehensive yet concise summary of each section, the Commentary provides a convenient reference for regulations applicable to the construction of buildings and structures. In the chapters that follow, discussions focus on the full meaning and implications of the *Concrete Structural Requirements* (SBC 304) text. Guidelines suggest the most effective method of application, and the consequences of not adhering to the SBC 304 text. Illustrations are provided to aid understanding; they do not necessarily illustrate the only methods of achieving *code* compliance.

The format of the Commentary includes the section, table and figure which is applicable to the same section in the SBC 304C. The numbers of the section, table and figure in the commentary begin with the letter R. The Commentary reflects the most up-to-date text of the 2007 *Saudi Building Code concrete structural requirements* (SBC 304C). American Concrete Institute (ACI) grants permission to the SBCNC to include all or portions of ACI codes and standards in the SBC, and ACI is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Readers should note that the Commentary (SBC 304C) is to be used in conjunction with the *Saudi Building Code concrete structural requirements (SBC 304)* and not as a substitute for the code. **The Commentary is advisory only**; the code official alone possesses the authority and responsibility for interpreting the code.

Comments and recommendations are encouraged, for through your input, it can improve future editions.

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CHAPTER 1
GENERAL REQUIREMENTS

SECTION R1.1
SCOPE

R1.1.1 The Saudi Building Code Commentary for Concrete Structures referred to as SBC 304C, provides comments on minimum requirements set forth in SBC 304. Hence, SBC 304C is not intended to be used independent of SBC 304, but rather as a companion to provide background information on code provisions.

The term "structural concrete" is used to refer to all concrete used for structural purposes. This covers the spectrum of structural applications of concrete from concrete containing nonprestressed reinforcement, prestressing steel, or composite steel shapes, pipe, or tubing.

Prestressed concrete is included under the definition of reinforced concrete. Provisions of the code apply to prestressed concrete except for those that are stated to apply specifically to nonprestressed concrete.

Chapter 21 of the code contains special provisions for design and detailing of earthquake resistant structures. See R1.1.8.

Appendix A of the SBC 304 contains provisions for the design of regions near geometrical discontinuities, or abrupt changes in loadings.

Appendix B of the SBC 304 contains provisions for reinforcement limits based on $0.65\rho_b$ determination of the strength reduction factor ϕ**,** and moment redistribution. The provisions are applicable to reinforced and prestressed concrete members. Designs made using the provisions of Appendix B are equally acceptable as those based on the body of the code, provided the provisions of Appendix B are used in their entirety.

Appendix C of the SBC 304 contains simplified coefficient methods for the design of two-way slabs.

Appendix D of the SBC 304 contains provisions for anchoring to concrete.

R1.1.4 Some special structures involve unique design and construction problems that are not covered by the code. However, many code provisions, such as the concrete quality and design principles, are applicable for these structures. Detailed recommendations for design and construction of some special structures are given in Refs. 1.1 to 1.5.

R1.1.5 The design and installation of piling fully embedded in the ground is regulated by the SBC. For portions of piling in air or water, or in soil not capable of providing adequate lateral restraint throughout the piling length to prevent buckling, the design provisions of this code govern where applicable. Further details about design and construction of piles may be found in Refs. 1.6 to 1.8.

R1.1.7 **Concrete on steel form deck.** In steel framed structures, it is common practice to cast concrete floor slabs on stay-in-place steel form deck. In all cases, the deck serves as the form and may, in some cases, serve an additional structural function.

R1.1.7.1 In its most basic application, the steel form deck serves as a form, and the concrete serves a structural function and, therefore, are to be designed to carry all superimposed loads.

R1.1.8 Special provisions for earthquake resistance

R1.1.8.1 For structures located in regions of low seismic risk, or for structures assigned to low seismic performance or design categories, no special design or detailing is required; the general requirements of the main body of the code apply for proportioning and detailing of reinforced concrete structures. It is expected that concrete structures proportioned by the main body of the code will provide a level of toughness adequate for low earthquake intensity.

R1.1.8.2 Intermediate and special concrete moment frames and shear walls proportioned to resist seismic effects require special reinforcement details. The special proportioning and detailing requirements of Chapter 21 are intended to provide a monolithic reinforced concrete or precast concrete structure with adequate "toughness" to respond inelastically under severe earthquake motions. See also R21.2.1.

SECTION R1.2
CONSTRUCTION DOCUMENTS

R1.2.1 SBC 304 lists some of the more important items of information that should be included in the design drawings, details, or specifications. The SBC does not imply an all inclusive list, and additional items may be required by the building official.

R1.2.2 Documented computer output is acceptable in lieu of manual calculations. When a computer program has been used by the designer, only skeleton data should normally be required. This should consist of sufficient input and output data and other information to allow the building official to perform a detailed review and make comparisons using another program or manual calculations. Input data should be identified as to member designation, app lied loads, and span lengths. The related output data should include member designation and the shears, moments, and reactions at key points in the span. For column design, it is desirable to include moment magnification factors in the output where applicable.

The code permits model analysis to be used to supplement structural analysis and design calculations. Documentation of the model analysis should be provided with the related calculations. Model analysis should be performed by an engineer having experience in this technique.

R1.2.3 Building official is the term used to identify the person charged with administration and enforcement of the provisions of the building code. However, such terms as building commissioner or building inspector are variations of the title, and the term building official as used in SBC is intended to include those variations as well as others that are used in the same sense.

SECTION R1.3
INSPECTION

R1.3.1 By inspection, the code does not mean that the inspector should supervise the construction. Rather it means that the one employed for inspection should visit the project with the frequency necessary to observe the various stages of work and ascertain that it is being done in compliance with contract documents and code requirements. The frequency should be at least enough to provide general knowledge of each operation, whether this be several times a day or once in several days.

Inspection in no way relieves the contractor from his obligation to follow the plans and specifications and to provide the designated quality and quantity of materials and workmanship for all job stages. The inspector should be present as frequently as deems necessary to judge whether the quality and quantity of the work complies with the contract documents; to counsel on possible ways of obtaining the desired results; to see that the general system proposed for formwork appears proper (though it remains the contractor's responsibility to design and build adequate forms and to leave them in place until it is safe to remove them); to see that reinforcement is properly installed; to see that concrete is of the correct quality, properly placed, and cured; and to see that tests for quality control are being made as specified.

Recommended procedures foe organization and conduct of concrete inspection are given in Ref. 1.9. Detailed methods of inspecting concrete construction are given in Ref. 1.10.

SECTION R1.4
APPROVAL OF SPECIAL SYSTEMS OF DESIGN OR CONSTRUCTION

New methods of design, new materials, and new uses of materials should undergo a period of development before being specifically covered in a code. Hence, good systems or components might be excluded from use by implication if means were not available to obtain acceptance.

For special systems considered under this section, specific tests, load factors, deflection limits, and other pertinent requirements should be set by the board of examiners, and should be consistent with the intent of the SBC.

The provisions of this section do not apply to model tests used to supplement calculations under 1.2.2 or to strength evaluation of existing structures under Chapter 20.

CHAPTER 2
DEFINITIONS

R2.1 For consistent application of the code, it is necessary that terms be defined where they have particular meanings in the code. The definitions given are for use in application of this code only and do not always correspond to ordinary usage. A glossary of most used terms relating to cement manufacturing, concrete design and construction, and research in concrete is contained in Reference 2.1.

Anchorage device - Most anchorage devices for post-tensioning are standard manufactured devices available from commercial sources. In some cases, designers or constructors develop "special" details or assemblages that combine various wedges and wedge plates for anchoring prestressing steel with specialty end plates or diaphragms.

Anchorage zone - The terminology "ahead of' and "behind" the anchorage device is illustrated in Fig. R18.13.1(b).

Basic anchorage devices are those devices that are so proportioned that they can be checked analytically for compliance with bearing stress and stiffness requirements without having to undergo the acceptance testing program required of special anchorage devices.

Column - The term compression member is used in the code to define any member in which the primary stress is longitudinal compression. Such a member needs not be vertical but may have any orientation in space. Bearing walls, columns, and pedestals qualify as compression members under this definition.

The differentiation between columns and walls in the code is based on the principal use rather than on arbitrary relationships of height and cross-sectional dimensions. The code, however, permits walls to be designed using the principles stated for column design (see 14.4), as well as by the empirical method (see 14.5).

While a wall always encloses or separates spaces, it may also be used to resist horizontal or vertical forces or bending. For example, a retaining wall or a basement wall also supports various combinations of loads.

A column is normally used as a main vertical member carrying axial loads combined with bending and shear. It may, however, form a small part of an enclosure or separation.

In this code, compressive strength of concrete is based on 150 × 300 mm cylindrical specimens (ASTM C39). Cubic specimens may be used for evaluating concrete compressive strength subjected to the provision of Section R. 5.1.2.

Concrete, structural lightweight - By code definition, sand-lightweight concrete is structural lightweight concrete with all of the fine aggregate replaced by sand. This definition may not be in agreement with usage by some material suppliers or contractors where the majority, but not all, of the lightweight fines are replaced by sand. For proper application of the code provisions, the replacement limits should be stated, with interpolation when partial sand replacement is used.

Deformed reinforcement - Deformed reinforcement is defined as that meeting the deformed bar specifications of 3.5.3.1, or the specifications of 3.5.3.3, 3.5.3.4, 3.5.3.5, or 3.5.3.6. No other bar or fabric qualifies. This definition permits accurate statement of anchorage lengths. Bars or wire not meeting the deformation requirements or fabric not meeting the spacing requirements are "plain reinforcement," for code purposes, and may be used only for spirals.

Loads - A number of definitions for loads are given as the code contains requirements that are to be met at various load levels. The terms dead load and live load refer to the unfactored loads (service loads) specified or defined by the SBC 301. Service loads (loads without load factors) are to be used where specified in the code to proportion or investigate members for adequate serviceability, as in 9.5, Control of Deflections. Loads used to proportion a member for adequate strength are defined as factored loads. Factored loads are service loads multiplied by the appropriate load factors specified in 9.2 for required strength.

Prestressed concrete - Reinforced concrete is defined to include prestressed concrete. Although the behavior of a prestressed member with unbonded tendons may vary from that of members with continuously bonded tendons, bonded and unbonded prestressed concrete are combined with conventionally reinforced concrete under the generic term "reinforced concrete." Provisions common to both prestressed and conventionally reinforced concrete are integrated to avoid overlapping and conflicting provisions.

Sheathing - Typically, sheathing is a continuous, seamless, high-density polyethylene material extruded directly on the coated pre-stressing steel.

Special anchorage devices are any devices (monostrand or multistrand) that do not meet the relevant Post Tensioning Institute (PTI) or AASHTO bearing stress and, where applicable, stiffness requirements. Most commercially marketed multibearing surface anchorage devices are Special Anchorage Devices. As provided in 18.15.1, such devices can be used only when they have been shown experimentally to be in compliance with the AASHTO requirements. This demonstration of compliance will ordinarily be furnished by the device manufacturer.

Strength, nominal - Strength of a member or cross section calculated using standard assumptions and strength equations, and nominal (specified) values of material strengths and dimensions is referred to as "nominal strength." The subscript n is used to denote the nominal strengths; nominal axial load strength P_n nominal moment strength M_n, and nominal shear strength V_u. "Design strength" or usable strength of a member or cross section is the nominal strength reduced by the strength reduction factor ϕ .

The required axial load, moment, and shear strengths used to proportion members are referred to either as factored axial loads, factored moments, and factored shears, or required axial loads, moments, and shears. The factored load effects are calculated from the applied factored loads and forces in such load combinations as are stipulated in the code (see 9.2).

The subscript u is used only to denote the required strengths; required axial load strength P_u required moment strength M_u, and required shear strength V_u, calculated from the applied factored loads and forces.

The basic requirement for strength design may be expressed as follows:
Design strength $\geq$ Required strength

$$\phi P_n \geq P_u$$
$$\phi M_n \geq M_u$$
$$\phi V_n \geq V_u$$

For additional discussion on the concepts and nomenclature for strength design see commentary Chapter 9.

CHAPTER 3
MATERIALS

SECTION R3.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or N.

SECTION R3.1
TESTS OF MATERIALS

R3.1.3 The record of tests of materials and of concrete should be retained for at least 5 years after completion of the project. Completion of the project is the date at which the owner accepts the project or when the certificate of occupancy is issued, whichever date is later.

SECTION R3.2
CEMENTS

R3.2.2 Depending on the circumstances, the provision of 3.2.2 may require only the same type of cement or may require cement from the identical source. The latter would be the case if the standard deviation of strength tests used in establishing the required strength margin was based on a cement from a particular source. If the standard deviation was based on tests involving a given type of cement obtained from several sources, the former interpretation would apply.

SECTION R3.3
AGGREGATES

R3.3.1 Aggregates conforming to the ASTM specifications are not always economically available and, in some instances, noncomplying materials have a long history of satisfactory performance. Such nonconforming materials are permitted with special approval when acceptable evidence of satisfactory performance is provided. Satisfactory performance in the past, however, does not guarantee good performance under other conditions and in other localities. Whenever possible, aggregates conforming to the designated specifications should be used.

R3.3.2 The size limitations on aggregates are provided to ensure proper encasement of reinforcement and to minimize honeycombing. Note that the limitations on maximum size of the aggregate may be waived if, in the judgment of the engineer, the workability and methods of consolidation of the concrete are such that the concrete can be placed without honeycombs or voids.

SECTION R3.4
WATER

R3.4.1 Almost any natural water that is drinkable (potable) and has no pronounced taste or odor is satisfactory as mixing water for making concrete. Impurities in mixing water, when excessive, may affect not only setting time, concrete strength, and volume stability (length change), but may also cause efflorescence or corrosion of reinforcement. Where possible, water with high concentrations of dissolved solids should be avoided.

Salts or other deleterious substances contributed from the aggregate or admixtures are additive to the amount which might be contained in the mixing water. These additional amounts are to be considered in evaluating the acceptability of the total impurities that may be deleterious to concrete or steel.

R.3.4.3 Nonpotable water for mixing or curing concrete should be substantially free from contamination particularly that raise the chloride and sulfate content of concrete. Water containing less than 2000 ppm of total dissolved solid can generally be used satisfactory for making concrete.

SECTION R3.5
STEEL REINFORCEMENT

R3.5.1 Materials permitted for use as reinforcement are specified. Other metal elements, such as inserts, anchor bolts, or plain bars for dowels at isolation or contraction joints, are not normally considered to be reinforcement under the provisions of SBC 304.

R3.5.2 Welding of reinforcing bars is not recommended. However, when needed, the procedure and provisions given in AWS D1.4 or equivalent may be used. See 3.8.2.

R3.5.3 Deformed reinforcement

R3.5.3.2 ASTM A 615M includes provisions for Grade 520 bars in sizes with Dia 20 mm and larger.

The 0.35 percent strain limit is necessary to ensure that the assumption of an elasto-plastic stress-strain curve in 10.2.4 will not lead to unconservative values of the member strength.

The 0.35 strain requirement is not applied to reinforcing bars having yield strengths of 420 MPa or less. For steels having strengths of 300 MPa, the assumption of an elasto-plastic stress-strain curve is well justified by extensive test data. For higher strength steels, up to 420 MPa, the stress-strain curve may or may not be elasto-plastic as assumed in 10.2.4, depending on the properties of the steel and the manufacturing process. However, when the stress-strain curve is not elasto-plastic, there is limited experimental evidence to suggest that the actual steel stress at ultimate strength may not be enough less than the specified yield strength to warrant the additional effort of testing to the more restrictive criterion applicable to steels having f_y greater than 420 MPa. In such cases, the ϕ-factor can be expected to account for the strength deficiency.

R3.5.3.5 Welded plain wire fabric should be made of wire conforming to "Specification for Steel Wire, Plain, for Concrete Reinforcement" (ASTM A 82). ASTM A 82 has a minimum yield strength of 482 MPa. The SBC has assigned a yield strength value of 420 MPa, but makes provision for the use of higher yield strengths provided the stress corresponds to a strain of 0.35 percent.

R3.5.3.6 Welded deformed wire fabric should be made of wire conforming to "Specification for Steel Wire, Deformed, for Concrete Reinforcement" (ASTM A

496). ASTM A 496 has a minimum yield strength of 482 MPa. The SBC 304 has assigned a yield strength value of 420 MPa, but makes provision for the use of higher yield strengths provided the stress corresponds to a strain of 0.35 percent.

R3.5.3.7 Galvanized reinforcing bars (A 767M), epoxy-coated reinforcing bars (A 775M) and epoxy-coated prefabricated reinforcing bars (A 934M) are used, especially for conditions where corrosion resistance of reinforcement is of particular concern. They have typically been used in parking decks, bridge decks, and other highly corrosive environments.

R3.5.4 **Plain reinforcement**. Plain bars and plain wire are permitted only for spiral reinforcement (either as lateral reinforcement for compression members, for torsion members, or for confining reinforcement for splices).

R3.5.5 **Prestressing steel**

R3.5.5.1 Because low-relaxation prestressing steel is addressed in a supplement to ASTM A 421, which applies only when low-relaxation material is specified, the appropriate ASTM reference is listed as a separate entity.

SECTION R3.6
ADMIXTURE

R3.6.3 Admixtures containing any chloride, other than impurities from admixture ingredients, should not be used in prestressed concrete or in concrete with aluminum embedments. Concentrations of chloride ion may produce corrosion of embedded aluminum (e.g., conduit), especially if the aluminum is in contact with embedded steel and the concrete is in a humid environment. Serious corrosion of galvanized steel sheet and galvanized steel stay-in-place forms occurs, especially in humid environments or where drying is inhibited by the thickness of the concrete or coatings or impermeable coverings. See 4.4.1 for specific limits on chloride ion concentration in concrete.

R3.6.7 Ground granulated blast-furnace slag conforming to ASTM C 989 is used as an admixture in concrete in much the same way as fly ash. Generally, it should be used with Portland cements conforming to ASTM C 150, and only rarely would it be appropriate to use ASTM C 989 slag with an ASTM C 595M blended cement that already contains a pozzolan or slag. Such use with ASTM C 595M cements might be considered for massive concrete placements where slow strength gain can be tolerated and where low heat of hydration is of particular importance. ASTM C 989 includes appendices which discuss effects of ground granulated blast-furnace slag on concrete strength, sulfate resistance, and alkali-aggregate reaction.

R3.6.8 The use of admixtures in concrete containing ASTM C 845 expansive cements has reduced levels of expansion or increased shrinkage values. See Ref. 3.2.

SECTION 3.8
REFERENCED STANDARDS

The ASTM standard specifications listed are the latest editions at the time these SBC 304 provisions were adopted. Since these specifications are revised frequently, generally in minor details only, the user of the SBC 304 should check directly with the sponsoring organization if it is desired to reference the latest edition. However, such a procedure obligates the user of the specification to evaluate if any changes in the later edition are significant in the use of the specification.

Standard specifications or other material to be legally adopted by reference into SBC should refer to a specific document. This can be done by simply using the complete serial designation since the first part indicates the subject and the second part the year of adoption. All standard documents referenced in the SBC 304 are listed in section 3.8, with the title and complete serial designation. In other sections of the SBC 304, the designations do not include the date so that all may be kept up-to-date by simply revising section 3.8.

CHAPTER 4
DURABILITY REQUIREMENTS

This chapter emphasizes the importance of considering durability requirements before the designer selects f'_c and concrete cover over the reinforcing steel. Concrete exposed to sulfate-bearing soils or groundwater, seawater, or for preventing corrosion of reinforcing steel or salt weathering should be designed for maximum water-cementitious materials ratio, minimum cementitious materials content and appropriate type of cement. Maximum water-cementitious materials ratios of 0.40 to 0.50 that may be required for concretes exposed to sulfate-bearing soils or groundwaters, or for preventing corrosion of reinforcement will typically be equivalent to requiring an f'_c of about 35 to 28 MPa, respectively. Generally, the required average concrete strength, f'_{cr}, will be 5 to 8 MPa higher than the specified compressive strength, f'_c.

Since it is difficult to accurately determine the water-cementitious materials ratio of concrete during production, the f'_c specified should be reasonably consistent with the water-cementitious materials ratio selected for durability. Selection of an f'_c that is consistent with the water-cementitious materials ratio selected for durability will help ensure that the required water-cementitious materials ratio is actually obtained in the field. Because the usual emphasis on inspection is for strength, test results substantially higher than the specified strength may lead to a lack of concern for quality and production of concrete that exceeds the maximum water-cementitious materials ratio. Thus, an f'_c of less than 20 MPa and a maximum water-cementitious materials ratio of 0.45 should not be specified for structures exposed to aggressive environments.

The SBC 304 does not include provisions for especially severe exposures, such as acids or high temperatures, and is not concerned with aesthetic considerations, such as surface finishes. These items are beyond the scope of the SBC 304 and should be covered specifically in the project specifications. Concrete ingredients and proportions are to be selected to meet the minimum requirements stated in the SBC 304 and the additional requirements of the contract documents.

SECTION R4.2
FREEZING AND THAWING EXPOSURE

Freeze-thaw conditions are rarely observed in the climatic conditions of the Kingdom.

SECTION R4.3
SULFATE EXPOSURE

R4.3.1 Concrete exposed to injurious concentrations of sulfates from soil or groundwater should be made with sulfate-resisting cement. Table 4.3.1 lists the appropriate types of cement and the maximum water-cementitious materials ratios, minimum cementitious materials contents and minimum compressive strength for various exposure conditions. In selecting cement for sulfate resistance, the principal consideration is its tricalcium aluminate (C_3A) content. For moderate exposures, Type II cement is limited to a maximum C_3A content of 8.0 percent under ASTM

C 150. For severe exposures, Type V cement with a maximum C_3A content of 5 percent is specified. Type V cement may be used when Type II cement is not available.

ASTM C 1012[4.1] can be used to evaluate the sulfate resistance of concrete mixtures using combinations of cementitious materials.

In addition to proper selection of cement, other requirements for durable concrete exposed to injurious concentrations of sulfate are essential, such as low water-cementitious materials ratio, minimum cementitious materials content, strength, adequate consolidation, uniformity, adequate cover over reinforcing steel and sufficient moist curing to develop the potential properties of concrete.

TABLE 4.3.1 – REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-BEARING SOILS OR WATER

Sulfate exposure	Water soluble sulfate (SO_4) in soil, percent by weight	Sulfate (SO_4) in water, ppm	Cement type	Maximum water-cementitious materials ratio, by weight	Minimum cementitious materials content, kg/m^3	Minimum f'_c, MPa
Negligible	$0.00 \le SO_4 < 0.10$	$0 \le SO_4 < 150$	–	–	–	–
Moderate	$0.10 \le SO_4 < 0.20$	$150 \le SO_4 < 1500$	II	0.50	330	28
Severe+	$0.20 \le SO_4 \le 2.00$	$1500 \le SO_4 \le 10{,}000$	V	0.45	350	30
Very severe+	$SO_4 > 2.00$	$SO_4 > 10{,}000$	V plus pozzolan++	0.45	350	30

\+ If sulfate ions are associated with magnesium ions, supplementary protection, such as application of a barrier coating, is required.

++ Pozzolan that conforms to relevant ASTM standards or that is shown to improve the sulfate resistance by service records should only be used.

SECTION R4.4
CORROSION PROTECTION OF REINFORCEMENT

R4.4.1 Test procedures for determining chloride concentration in concrete should conform to those given in ASTM C 1218. An initial evaluation may be obtained by testing individual concrete ingredients for water-soluble chloride ion content. If the water-soluble chloride ion content, calculated on the basis of concrete proportions, exceeds those permitted in Table 4.4.1, it may be necessary to test samples of the hardened concrete for water-soluble chloride ion content. Some of the total chloride ions present in the ingredients will either be insoluble or will react with the cement during hydration and become insoluble under the test procedures described in ASTM C 1218. Additional information on the effect of chloride on the corrosion of reinforcement steel are given in Ref. 4.2 and 4.3.

When concrete is tested for water-soluble chloride ion content, the test should be made at an age of 28 to 42 days.

The limits in Table 4.4.1 are applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.

TABLE 4.4.1 – MAXIMUM CHLORIDE ION CONTENT FOR CORROSION PROTECTION OF REINFORCEMENT

Type of member	Maximum water-soluble chloride ion (cl^-) in concrete, percent by weight of cement*
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

* Determined according to ASTM C 1218.

R4.4.2 When reinforced concrete structures are exposed to external sources of chlorides, the water-cementitious materials ratio, cementitious materials content, and specified compressive strength, f_c', of Table 4.4.2 are the minimum requirements that are to be considered. Epoxy- or zinc-coated bars or slag meeting ASTM C 989 or fly ash meeting ASTM C 618 or silica fume meeting ASTM C 1240 with an appropriate high-range water reducer, ASTM C 494M, Types F and G, or ASTM C 1017M can provide additional protection [4.4]. When epoxy-coated steel bars are used, they should be according to ASTM A 775 specifications.

The requirements for minimum concrete cover over the reinforcing steel of 7.7 in conjunction with 7.7.5 should be considered.

The requirements for protection of concrete against carbonation are not provided as it is expected that the use of quality concrete and adequate cover over reinforcing steel, as specified in the SBC 304, will minimize this problem.

R4.4.5 In the coastal areas, such as in Jeddah, Yanbu, Dammam, Jizan, and others, the substructures are exposed to chloride- and sulfate-bearing soil and/or groundwater. In such situations, the requirements of 4.5 shall be considered.

TABLE 4.4.2 – REQUIREMENTS FOR CONCRETE EXPOSED TO CHLORIDE-BEARING SOIL AND WATER

Chloride exposure	Water soluble chloride (cl^-) in soil, percent by weight	Water soluble chloride (cl^-) in water, ppm	Cement type	Maximum water-cementitious materials ratio	Minimum cementitious materials content, kg/m^3	Minimum f_c', MPa
Negligible	Upto 0.05	Up to 500	–	–	–	–
Moderate	0.05 to 0.1	500 to 2,000	–	0.50	330	28
Severe	0.1 to 0.5	2,000 to 10,000	I	0.45	350	30
Very severe	More than 0.5	More than 10,000	I + pozzolan$^+$	0.40	370	35

+Pozzolan that conforms to relevant standards shall only be used.

SECTION R4.5
SULFATE PLUS CHLORIDE EXPOSURES

R4.5.1 Structures exposed to environments containing both sulfate and chloride salts are prone to both sulfate attack and reinforcement corrosion. For such environments, the concrete mixture proportions should be selected for the severest of the exposure conditions, in terms of chloride or sulfate concentration, in Tables 4.3.1

or 4.4.2. In such situations, the lowest applicable maximum water-cementitious materials ratio and highest pertinent minimum cementitious materials content for the severest exposure conditions should be selected.

Since reinforcement corrosion is the major form of concrete deterioration, in a chloride-sulfate environment, as sulfate ions do not penetrate deeper into the concrete cover, it is suggested to use the cement type specified in Table 4.4.2, rather than that dictated by the severity of the exposure conditions.

SECTION R4.6
SABKHA EXPOSURES

R4.6.1 Sabkha is a saline flat land underlain by sand, silt or clay and is often encrusted with salt. These soils either border partially land-locked seas or cover a number of continental depressions. Both of these types are usually formed in hot, arid climates and are associated with shallow groundwater table. Sabkha terrains are typically located in Jubail, Rastanura, Abqaiq, Dammam, and Shaibah along the Arabian Gulf coast. They are prevalent in Jeddah, Jizan, Qunfudhah, Al-Lith, Rabigh, Yanbu in Western Province as well as in Wadi As-Sirhan, around Al-Qasim. and around Riyadh.

The salinity of sabkha soil is three to five times that of seawater from the same vicinity. The high chloride concentration in these soils accelerates concrete deterioration due to reinforcement corrosion. In such situations, in addition to utilizing quality concrete, the structural components need to be protected by appropriate measures, such as tanking or epoxy-based coating.

SECTION R4.7
SALT WEATHERING

R4.7.1 Concrete exposed to splash in a marine environment and soil with shallow groundwater table or water from irrigation is susceptible to deterioration due to salt weathering in the hot and arid environment of the Kingdom. In addition to utilizing quality concrete, it may be necessary to provide additional protective measures, such as the application of an appropriate barrier coating.

In marine structures, the protection should be provided in the splash zone. Tanking or application of a barrier coating in portions exposed to soil is necessary for the substructures.

CHAPTER 5
CONCRETE QUALITY, MIXING AND PLACING

The requirements for proportioning concrete mixtures are based on the philosophy that concrete should provide both adequate durability (Chapter 4) and strength. The criteria for acceptance of concrete are based on the philosophy that the SBC 304 is intended primarily to protect the safety of the public. Chapter 5 describes procedures by which concrete of adequate strength can be obtained, and provides procedures for checking the quality of the concrete during and after its placement in the work.

Chapter 5 also prescribes minimum criteria for mixing and placing concrete.
The provisions of 5.2, 5.3, and 5.4, together with Chapter 4, establish required mixture proportions. The basis for determining the adequacy of concrete strength is in 5.6.

SECTION R5.1
GENERAL

R5.1.1 The basic premises governing the designation and evaluation of concrete strength are presented. It is emphasized that the average strength of concrete produced should always exceed the specified value of f_c' used in the structural design calculations. This is based on probabilistic concepts, and is intended to ensure that adequate concrete strength will be developed in the structure. The durability requirements prescribed in Chapter 4 are to be satisfied in addition to attaining the average concrete strength in accordance with 5.3.2.

R5.1.2 Cubic specimens (150 × 150 × 150) in accordance with SASO 79 may be used in evaluating the compressive strength, using the following correction factor: $f_c' = k(f_{cubic})$ where $k = 0.8$.

R5.1.4 Sections 9.5.2.3 (modulus of rupture), 11.2 (concrete shear strength) and 12.2.4 (development of reinforcement) require modification in the design criteria for the use of lightweight aggregate concrete. Two alternative modification procedures are provided. One alternative is based on laboratory tests to determine the relationship between splitting tensile strength f_{ct} and specified compressive strength f_c' for the lightweight concrete. For a lightweight aggregate from a given source, it is intended that appropriate values of f_{ct} be obtained in advance of design.

R5.1.5 Tests for splitting tensile strength of concrete (as required by 5.1.4) are not intended for control of, or acceptance of the strength of concrete in the field. Indirect control will be maintained through the normal compressive strength test requirements provided by 5.6.

SECTION R5.2
SELECTION OF CONCRETE PROPORTIONS

Recommendations for selecting proportions for concrete are given in detail in Reference 5.1. (Provides two methods for selecting and adjusting proportions for normalweight concrete: the estimated weight and absolute volume methods.

Example calculations are shown for both methods. Proportioning of heavyweight concrete by the absolute volume method is presented in an appendix.)

Recommendations for lightweight concrete are given in Reference 5.2. (Provides a method of proportioning and adjusting structural grade concrete containing lightweight aggregates.)

R5.2.1 The selected water-cementitious materials ratio should be low enough, or in the case of lightweight concrete the compressive strength high enough to satisfy both the strength criteria (see 5.3 or 5.4) and the special exposure requirements (Chapter 4). The SBC 304 does not include provisions for especially severe exposures, such as acids or high temperatures, and is not concerned with aesthetic considerations such as surface finishes. These items are beyond the scope of the SBC 304 and should be covered specifically in the project specifications. Concrete ingredients and proportions are to be selected to meet the minimum requirements stated in the SBC 304 and the additional requirements of the contract documents.

R5.2.3 The SBC 304 emphasizes the use of field experience or laboratory trial mixtures (see 5.3) as the preferred method for selecting concrete mixture proportions.

SECTION R5.3 PROPORTIONING ON THE BASIS OF FIELD EXPERIENCE OR TRIAL MIXTURES, OR BOTH

In selecting a suitable concrete mixture there are three basic steps. The first is the determination of the standard deviation. The second is the determination of the required average strength. The third is the selection of mixture proportions required to produce that average strength, either by conventional trial mixture procedures or by a suitable experience record. Fig. R5.3 is a flow chart outlining the mixture selection and documentation procedure.

The mixture selected should yield an average strength appreciably higher than the specified strength f'_c. The degree of mixture overdesign depends on the variability of the test results.

R5.3.1 **Standard deviation.** When a concrete production facility has a suitable record of 30 consecutive tests of similar materials and conditions expected, the standard deviation is calculated from those results in accordance with the following formula:

$$s = \left[\frac{\sum \left(x_i - \bar{x} \right)^2}{(n-1)} \right]^{1/2}$$

where:

s = standard deviation, MPa

x_i = individual strength tests as defined in 5.6.2.4

$\bar{x}$ = average of n strength test results

n = number of consecutive strength tests

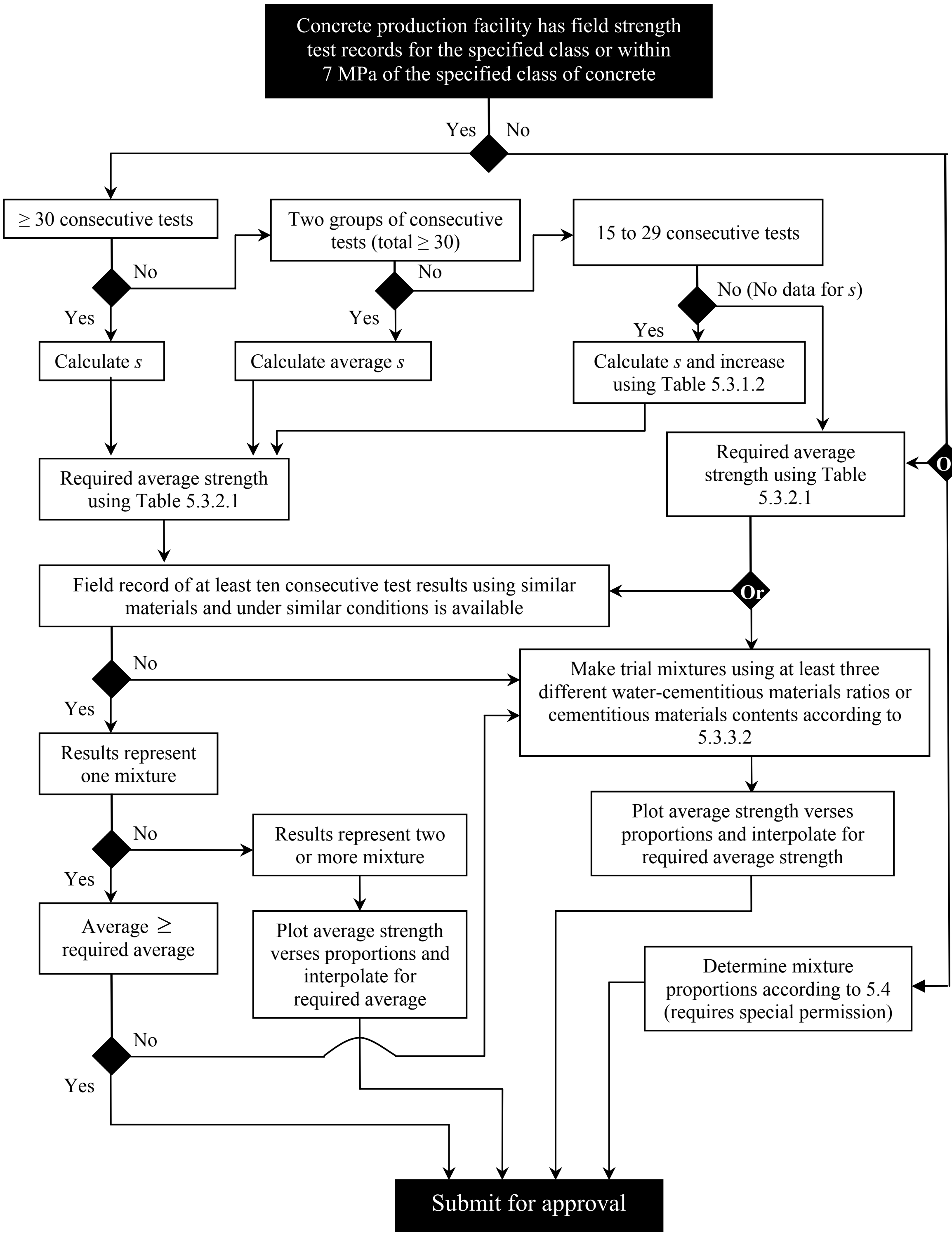


Fig. R5.3 - Flow chart for selection and documentation of concrete proportions

The standard deviation is used to determine the average strength required in 5.3.2.1.

If two test records are used to obtain at least 30 tests, the standard deviation used shall be the statistical average of the values calculated from each test record in accordance with the following formula:

$$\bar{s} = \left\{ \frac{(n_1 - 1)(s_1)^2 + (n_2 - 1)(s_2)^2}{(n_1 + n_2 - 2)} \right\}^{1/2}$$

where:

$\bar{s}$ = statistical average standard deviation where two test records are used to estimate the standard deviation

s_1, s_2 = standard deviations calculated from two test records, 1 and 2, respectively

n_1, n_2 = number of tests in each test record, respectively

If less than 30, but at least 15 tests are available, the calculated standard deviation is increased by the factor given in Table 5.3.1.2. This procedure results in a more conservative (increased) required average strength. The factors in Table 5.3.1.2 are based on the sampling distribution of the standard deviation and provide protection (equivalent to that from a record of 30 tests) against the possibility that the smaller sample under estimates the true or universe population standard deviation.

The standard deviation used in the calculation of required average strength should be developed under conditions "similar to those expected" [see 5.3.1.1(a)]. This requirement is important to ensure acceptable concrete.

Concrete for background tests to determine standard deviation is considered to be "similar" to that required if made with the same general types of ingredients under no more restrictive conditions of control over material quality and production methods than on the proposed work, and if its specified strength does not deviate more than 7 MPa from the f_c' required [see 5.3.1.1(b)]. A change in the type of concrete or a major increase in the strength level may increase the standard deviation. Such a situation might occur with a change in type of aggregate (i.e., from natural aggregate to lightweight aggregate or vice versa) or a change from non-air-entrained concrete to air-entrained concrete. Also, there may be an increase in standard deviation when the average strength level is raised by a significant amount, although the increment of increase in standard deviation should be somewhat less than directly proportional to the strength increase. When there is reasonable doubt, any estimated standard deviation used to calculate the required average strength should always be on the conservative (high) side.

Note that the SBC 304 uses the standard deviation in mega Pascal instead of the coefficient of variation in percent. The latter is equal to the former expressed as a percent of the average strength.

Even when the average strength and standard deviation are of the levels assumed, there will be occasional tests that fail to meet the acceptance criteria prescribed in 5.6.3.3 (perhaps 1 test in 100).

TABLE 5.3.1.2-MODIFICATION FACTOR FOR STANDARD DEVIATION WHEN LESS THAN 30 TESTS ARE AVAILABLE

No. of tests*	Modification factor for standard deviation†
Less than 15	**Use table 5.3.2.2**
15	1.16
20	1.08
25	1.03
30 or more	1.00

* Interpolate for intermediate numbers of tests.

†Modified standard deviation to be used to determine required average strength f_{cr}' from 5.3.2.1.

R5.3.2 Required average strength

R5.3.2.1 Once the standard deviation has been determined, the required average compressive strength is obtained from the larger value computed from Eq. (5-1) and (5-2) for f_c' of 35 MPa or less, or the larger value computed from Eq. (5-1) and (5-3) for f_c' over 35 MPa. Equation (5-1) is based on a probability of 1-in-100 that the average of three consecutive tests may be below the specified compressive strength f_c'. Equation (5-2) is based on a similar probability that an individual test may be more than 3.5 MPa below the specified compressive strength f_c'. Equation (5-3) is based on the same 1-in-100 probability that an individual test may be less than $0.90 f_c'$. These equations assume that the standard deviation used is equal to the population value appropriate for an infinite or very large number of tests. For this reason, use of standard deviations estimated from records of 100 or more tests is desirable. When 30 tests are available, the probability of failure will likely be somewhat greater than 1-in-100. The additional refinements required to achieve the 1-in-100 probability are not considered necessary, because of the uncertainty inherent in assuming that conditions operating when the test record was accumulated will be similar to conditions when the concrete will be produced.

R5.3.2.2 Recent evaluation of production control of ready mix concrete in the Kingdom has shown that values of standard deviation provided by ready mix concrete plants under-estimate the true standard deviation substantially.[5.3,5.4] Therefore, it is prudent to use the required average strength based on Table 5.3.2.2 until the true standard deviation can be established by qualified independent laboratory.

TABLE 5.3.2.2-REQUIRED AVERAGE COMPRESSIVE STRENGTH WHEN DATA ARE NOT AVAILABLE TO ESTABLISH A STANDARD DEVIATION

Specified compressive strength, f_c', MPa	Required average compressive strength, f_{cr}', MPa
20 to 35	$f_c' + 8.5$
Over 35	$1.10 f_c' + 5.0$

R.5.3.3 **Documentation of average strength**. Once the required average strength f_{cr}', is known, the next step is to select mixture proportions that will produce an average strength at least as great as the required average strength, and also meet special exposure requirements of Chapter 4. The documentation may consist of a strength test record, several strength test records, or suitable laboratory or field trial mixtures. Generally, if a test record is used, it will be the same one that was used for computation of the standard deviation. However, if this test record shows either lower or higher average strength than the required average strength, different proportions may be necessary or desirable. In such instances, the average from a record of as few as 10 tests may be used, or the proportions may be established by interpolation between the strengths and proportions of two such records of consecutive tests. All test records for establishing proportions necessary to produce the average strength are to meet the requirements of 5.3.3.1 for "similar materials and conditions."

For strengths over 35 MPa where the average strength documentation is based on laboratory trial mixtures, it may be appropriate to increase f'_{cr}, calculated in Table 5.3.2.2 to allow for a reduction in strength from laboratory trials to actual concrete production.

SECTION R5.4
PROPORTIONING WITHOUT FIELD EXPERIENCE OR TRIAL MIXTURES

R5.4.1 When no prior experience (5.3.3.1) or trial mixture data (5.3.3.2) meeting the requirements of these sections is available, other experience may be used only when special permission is given. Because combinations of different ingredients may vary considerably in strength level, this procedure is not permitted for f'_c greater than 35 MPa and the required average strength should exceed f'_c by 8.5 MPa. The purpose of this provision is to allow work to continue when there is an unexpected interruption in concrete supply and there is not sufficient time for tests and evaluation or in small structures where the cost of trial mixture data is not justified.

SECTION R5.6
EVALUATION AND ACCEPTANCE OF CONCRETE

Once the mixture proportions have been selected and the job started, the criteria for evaluation and acceptance of the concrete can be obtained from 5.6.
An effort has been made in the SBC 304 to provide a clear-cut basis for judging the acceptability of the concrete, as well as to indicate a course of action to be followed when the results of strength tests are not satisfactory.

R5.6.1 Laboratory and field technicians can establish qualifications by becoming certified through certification programs. Field technicians in charge of sampling concrete; testing for slump, unit weight, yield, air content, and temperature; and making and curing test specimens should be certified in accordance with the requirements of ASTM C 1077,[5.5] or an equivalent program. Concrete testing laboratory personnel should be certified in accordance with the requirements of ASTM C 1077 or an equivalent program.

Testing reports should be promptly distributed to the owner, registered design professional responsible for the design, contractor, appropriate subcontractors, appropriate suppliers, and building official to allow timely identification of either compliance or the need for corrective action.

R5.6.2 **Frequency of testing**

R5.6.2.1 The following three criteria establish the required minimum sampling frequency for each class of concrete:

(a) Once each day a given class is placed, nor less than

(b) Once for each 120 m^3 of each class placed each day, nor less than

(c) Once for each 500 m^2 of slab or wall surface area placed each day.

In calculating surface area, only one side of the slab or wall should be considered. Criteria (c) will require more frequent sampling than once for each 120 m^3 placed if the average wall or slab thickness is less than 250 mm.

R5.6.2.2 Samples for strength tests are to be taken on a strictly random basis if they are to measure properly the acceptability of the concrete. To be representative, the choice of times of sampling, or the batches of concrete to be sampled, are to be made on the basis of chance alone, within the period of placement. Batches should not be sampled on the basis of appearance, convenience, or other possibly biased criteria, because the statistical analyses will lose their validity. Not more than one test (average of two cylinders made from a sample, 5.6.2.4) should be taken from a single batch, and water may not be added to the concrete after the sample is taken.

ASTM D 3665[5.6] describes procedures for random selection of the batches to be tested.

R5.6.3 **Laboratory-cured specimens**

R5.6.3.3 A single set of criteria is given for acceptability of strength and is applicable to all concrete used in structures designed in accordance with the SBC 304, regardless of design method used. The concrete strength is considered to be satisfactory as long as averages of any three consecutive strength tests remain above the specified f_c' and no individual strength test falls below the specified f_c' by more than 3.5 MPa if f_c' is 35 MPa or less, or falls below f_c' by more than 10 percent if f_c' is over 35 MPa. Evaluation and acceptance of the concrete can be judged immediately as test results are received during the course of the work. Strength tests failing to meet these criteria will occur occasionally (probably about once in 100 tests) even though concrete strength and uniformity are satisfactory. Allowance should be made for such statistically expected variations in deciding whether the strength level being produced is adequate.

R5.6.3.4 When concrete fails to meet either of the strength requirements of 5.6.3.3, steps should be taken to, increase the average of the concrete test results. If sufficient concrete has been produced to accumulate at least 15 tests, these should be used to establish a new target average strength as described in 5.3.

If fewer than 15 tests have been made on the class of concrete in question, the new target strength level should be at least as great as the average level used in the initial selection of proportions. If the average of the available tests made on the project equals or exceeds the level used in the initial selection of proportions, a further increase in average level is required.

The steps taken to increase the average level of test results will depend on the particular circumstances, but could include one or more of the following:

(a) An increase in cementitious materials content;

(b) Changes in mixture proportions;

(c) Reductions in or better control of levels of slump supplied;

(d) A reduction in delivery time;

(e) Closer control of air content;

(f) An improvement in the quality of the testing, including strict compliance with standard test procedures.

Such changes in operating and testing procedures, or changes in cementitious materials content, or slump should not require a formal resubmission under the procedures of 5.3; however, important changes in sources of cement, aggregates, or admixtures should be accompanied by evidence that the average strength level will be improved.

R5.6.4 Field-cured specimens

R5.6.4.1 Strength tests of cylinders cured under field conditions may be required to check the adequacy of curing and protection of concrete in the structure.

R5.6.4.4 Positive guidance is provided in the SBC 304 concerning the interpretation of tests of field-cured cylinders. Research has shown that cylinders protected and cured to simulate good field practice should test not less than about 85 percent of standard laboratory moist-cured cylinders. This percentage has been set as a rational basis for judging the adequacy of field curing. The comparison is made between the actual measured strengths of companion job-cured and laboratory-cured cylinders, not between job-cured cylinders and the specified value of $f_c^{'}$. However, results for the job-cured cylinders are considered satisfactory if the job-cured cylinders exceed the specified $f_c^{'}$ by more than 3.5 MPa, even though they fail to reach 85 percent of the strength of companion laboratory-cured cylinders.

R5.6.5 **Investigation of low-strength test results.** Instructions are provided concerning the procedure to be followed when strength tests have failed to meet the specified acceptance criteria. For obvious reasons, these instructions cannot be dogmatic. The building official should apply judgment as to the significance of low test results and whether they indicate need for concern. If further investigation is deemed necessary, such investigation may include nondestructive tests, or in extreme cases, strength tests of cores taken from the structure.

Nondestructive tests of the concrete in place, such as by probe penetration, impact hammer, ultrasonic pulse velocity or pull out may be useful in determining whether or not a portion of the structure actually contains low-strength concrete. Such tests are of value primarily for comparisons within the same job rather than as quantitative measures of strength. For cores, if required, conservatively safe acceptance criteria are provided that should ensure structural adequacy for virtually any type of construction.[5.7-5.10] Lower strength may, of course, be tolerated under many circumstances, but this again becomes a matter of judgment on the part of the building official and design engineer. When the core tests fail to provide assurance of structural adequacy, it may be practical, particularly in the case of floor or roof systems, for the building official to require a load test (Chapter 20). Short of load tests, if time and conditions permit, an effort may be made to improve the strength of the concrete in place by supplemental wet curing. Effectiveness of such a treatment should be verified by further strength evaluation using procedures previously discussed.

A core obtained through the use of a water-cooled bit results in a moisture gradient between the exterior and interior of the core being created during drilling. This adversely affects the core's compressive strength.[5.11] The restriction on the commencement of core testing provides a minimum time for the moisture gradient to dissipate.

Core tests having an average of 85 percent of the specified strength are realistic. To expect core tests to be equal to f'_c is not realistic, since differences in the size of specimens, conditions of obtaining samples, and procedures for curing, do not permit equal values to be obtained.

The SBC 304, as stated, concerns itself with assuring structural safety, and the instructions in 5.6 are aimed at that objective. It is not the function of the SBC 304 to assign responsibility for strength deficiencies, whether or not they are such as to require corrective measures.

Under the requirements of this section, cores taken to confirm structural adequacy will usually be taken at ages later than those specified for determination of f'_c.

SECTION R5.7
PREPARATION OF EQUIPMENT AND PLACE OF DEPOSIT

Recommendations for mixing, handling and transporting, and placing concrete are given in Reference 5.12 (Presents methods and procedures for control, handling and storage of materials, measurement, batching tolerances, mixing, methods of placing, transporting, and forms.)

Attention is directed to the need for using clean equipment and for cleaning forms and reinforcement thoroughly before beginning to deposit concrete. In particular, sawdust, nails, wood pieces, and other debris that may collect inside the forms should be removed. Reinforcement should be thoroughly cleaned of dirt, loose rust, mill scale, or other coatings. Water should be removed from the forms.

SECTION R5.8
MIXING

Concrete of uniform and satisfactory quality requires the materials to be thoroughly mixed until uniform in appearance and all ingredients are distributed. Samples taken from different portions of a batch should have essentially the same unit weight, air content, slump, and coarse aggregate content. Test methods for uniformity of mixing are given in ASTM C 94. The necessary time of mixing will depend on many factors including batch size, stiffness of the batch, size and grading of the aggregate, and the efficiency of the mixer. Excessively long mixing times should be avoided to guard against grinding of the aggregates.

SECTION R5.9
CONVEYING

Each step in the handling and transporting of concrete needs to be controlled to maintain uniformity within a batch and from batch to batch. It is essential to avoid segregation of the coarse aggregate from the mortar or of water from the other ingredients.

The SBC 304 requires the equipment for handling and transporting concrete to be capable of supplying concrete to the place of deposit continuously and reliably under all conditions and for all methods of placement. The provisions of 5.9 apply to all placement methods, including pumps, belt conveyors, pneumatic systems, wheelbarrows, buggies, crane buckets, and tremies.

Serious loss in strength can result when concrete is pumped through pipe made of aluminum or aluminum alloy 5.13 Hydrogen gas generated by the reaction between the cement alkalies and the aluminum eroded from the interior of the pipe surface has been shown to cause strength reduction as much as 50 percent. Hence, equipment made of aluminum or aluminum alloys should not be used for pump lines, tremies, or chutes other than short chutes such as those used to convey concrete from a truck mixer.

SECTION R5.10
PLACING

Re-handling concrete can cause segregation of the materials. Hence the SBC 304 cautions against this practice. Re-tempering of partially set concrete with the addition of water should not be permitted, unless authorized. This does not preclude the practice (recognized in ASTM C 94) of adding water to mixed concrete to bring it up to the specified slump range so long as prescribed limits on the maximum mixing time and water-cementitious materials ratio are not violated.

Recommendations for consolidation of concrete are given in detail in Ref. 5.14 (Presents current information on the mechanism of consolidation and gives recommendations on equipment characteristics and procedures for various classes of concrete).

SECTION R5.11
CURING

Recommendations for curing concrete are given in detail in Ref. 5.15 (Presents basic principles of proper curing and describes the various methods, procedures, and materials for curing of concrete.)

R5.11.3 **Accelerated curing.** The provisions of this section apply whenever an accelerated curing method is used, whether for precast or cast-in-place elements. The compressive strength of steam-cured concrete is not as high as that of similar concrete continuously cured under moist conditions at moderate temperatures. Also the modulus of elasticity E_c, of steam-cured specimens may vary from that of specimens moist-cured at normal temperatures. When steam curing is used, it is advisable to base the concrete mixture proportions on steam-cured test cylinders.

Accelerated curing procedures require careful attention to obtain uniform and satisfactory results. Preventing moisture loss during the curing is essential.

R5.11.4 In addition to requiring a minimum curing temperature and time for normal- and high-early-strength concrete, the SBC 304 provides a specific criterion in 5.6.4 for judging the adequacy of field curing. At the test age for which the strength is specified (usually 28 days), field-cured cylinders should produce strength not less than 85 percent of that of the standard, laboratory-cured cylinders. For a reasonably valid comparison to be made, field-cured cylinders and companion laboratory-cured cylinders should come from the same sample. Field-cured cylinders should be cured under conditions identical to those of the structure. If the structure is protected from the elements, the cylinder should be protected.

Cylinders related to members not directly exposed to weather should be cured adjacent to those members and provided with the same degree of protection and method of curing. The field cylinders should not be treated more favorably than the elements they represent. (See 5.6.4 for additional information.) If the field-cured cylinders do not provide satisfactory strength by this comparison, measures should be taken to improve the curing. If the tests indicate a possible serious deficiency in strength of concrete in the structure, core tests may be required, with or without supplemental wet curing, to check the structural adequacy, as provided in 5.6.5.

SECTION R5.13
HOT WEATHER REQUIREMENTS

R5.13.1 Hot weather is any combination of high ambient temperature, high concrete temperature; low relative humidity; wind speed; and solar radiation that tends to impair the quality of fresh or hardened concrete.

Potential problems for concrete in the freshly mixed state are likely to include: (a) increased water demand, (b) increased rate of slump loss, (c) increased rate of setting, and (d) increased tendency for plastic shrinkage cracking.
Potential deficiencies to concrete in the hardened state may include: (a) decreased long-term strength, (b) increased tendency for drying shrinkage and differential thermal cracking, and (c) decreased durability.
A detailed description of the hot weather factors and their effect on concrete properties is given in Ref. 5.16.

R5.13.2 Local Experience has shown that when reasonable precautions are employed by the batching plants, concrete with a temperature of less than 35 $^\circ$C can be delivered to jobsite [5.3].

Some of the precautions that may be employed to control concrete temperature include: (*i*) shading the aggregate stockpiles and bins, (*ii*) sprinkling water to cool the coarse aggregates, (*iii*) cooling the mixing water, (*iv*) using ice as part of the mixing water, and (*v*) painting trucks, silos and other related equipments, with white or light color. The contribution of aggregates, cement and water to the temperature of the freshly mixed concrete is related to their temperature, specific heat, and quantity of each material.

Tanks and pipelines carrying mixing water should be buried, insulated, shaded, or painted white or with light color to keep water at the lowest possible temperature. Silos and bins should be painted with heat-reflective paint to minimize absorption of heat. The surface of truck mixer should be painted white to minimize solar heat gain.

R5.13.3 Chemical admixtures have been found to be beneficial in offsetting some of the undesirable characteristics of concrete placed during periods of high ambient temperatures. The benefits may include lower mixing water, extended period of use, and strengths comparable to, or higher than, those of concrete without admixtures, placed at lower temperatures [5.16].

Admixtures meeting the requirements of ASTM C 494, Type D, have both water-reducing and set-retarding properties and are widely used under hot weather conditions. They can be included in concrete in varying proportions and in combination with other admixtures[5.16].

Some high-range, water-reducing and retarding admixtures (ASTM C 494, Type G) and plasticizing retarding admixtures (ASTM C 1017, Type II), often referred to as superplasticizers, can provide significant benefits under hot weather conditions. In particular, improved handling characteristic of concrete permits rapid placement and consolidation, and the period between mixing and placing can thus be reduced [5.16].

R5.13.4 Transporting and placing concrete should be done as quickly as practicable. Delays contribute to loss of slump and an increase in the concrete temperature. Sufficient labor and equipment must be available at the jobsite to handle and place concrete immediately upon delivery.

Water-reducing and retarding admixtures formulated for extended slump retention should be considered if longer delivery periods are anticipated.
Concrete placement may be scheduled at times other than during daylight hours. Night-time production and placement require good planning and good lighting.

R5.13.5 High-range water-reducing admixtures or superplasticizers may be utilized to regain the desired workability.

R5.13.6 Plastic shrinkage cracking is usually associated with hot-weather concreting. However, it can occur at ambient conditions that produce rapid evaporation of moisture from the concrete surface. These cracks can occur on the surface of freshly placed concrete while it is being finished or shortly thereafter, if the rate of water evaporation is more than the rate of bleeding. These cracks that appear mostly on horizontal surfaces can be substantially eliminated if preventive measures are taken.

Precautions to avoid plastic shrinkage cracking may include: erecting wind breakers and sun shades, fog spraying of form and reinforcement, dampening the sub-grade and forms, or placing concrete at the lowest practicable temperature, and time. After the completion of placing and finishing operations, concrete should be protected from high temperature, direct sun light, low humidity, and drying winds.
When the rate of evaporation exceeds 1 kg/m^2 per hour, precautionary measures are essential. Pozzolanic cement concrete is particularly prone to plastic shrinkage. Therefore, protection from premature drying is essential for pozzolanic cement concrete even at low evaporation rates.

The probability of plastic shrinkage cracks to occur may be increased if the time of setting of concrete is delayed due to the use of an excessive dosage of retarding admixture.

R5.13.7 Curing is more critical under hot weather conditions. Early curing is essential when pozzolan cement concrete is utilized.

Of the different curing procedures, moist-curing is the best. It can be provided by ponding, covering with clean sand kept continuously wet or continuous sprinkling of water. A more practical method of moist-curing is that of covering the pre-wetted concrete with an impervious sheeting or application of absorptive mats or fabric kept continuously wet with a soaker hose or similar means [5.16].

R5.13.8 The specimens should be representative of the concrete as delivered. High temperature, low relative humidity, and drying winds are particularly detrimental to the fresh concrete used for making tests and molding specimens. Leaving the specimens of fresh concrete exposed to sun, wind, or dry air will invalidate the test results.

Particular attention should be given to the protection and curing of strength test specimens used as a basis for acceptance of concrete. Due to their small size in relation to most parts of the structure, test specimens are influenced more readily by changes in ambient temperatures. Extra effort is needed in hot weather to maintain strength test specimens at a temperature of 16 to 27°C and to prevent moisture loss during the initial curing period [5.16].

CHAPTER 6
FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION JOINTS

SECTION R6.1
DESIGN OF FORMWORK

Only minimum performance requirements for formwork, necessary to provide for public health and safety, are prescribed in Chapter 6. Formwork for concrete, including proper design, construction, and removal, demands sound judgment and planning to achieve adequate forms that are both economical and safe. Detailed information on formwork for concrete is given in Reference 6.1. (Provides recommendations for design, construction, and materials for formwork, forms for special structures, and formwork for special methods of construction. Directed primarily to contractors, the suggested criteria will aid engineers in preparing job specifications for the contractors.)

***Formwork for Concrete*[6.2] -** A how-to-do-it handbook for contractors, engineers, and architects following the guidelines established in Ref. 6.2. Planning, building, and using formwork are discussed, including tables, diagrams, and formulas for form design loads.

SECTION R6.2
REMOVAL OF FORMS, SHORES, AND RESHORING

In determining the time for removal of forms, consideration should be given to the construction loads and to the possibilities of deflections.[6.3] The construction loads are frequently at least as great as the specified live loads. At early ages, a structure may be adequate to support the applied loads but may deflect sufficiently to cause permanent damage.

Evaluation of concrete strength during construction may be demonstrated by field-cured test cylinders or other procedures approved by the building official such as:

(a) Tests of cast-in-place cylinders in accordance with "Standard Test Method for Compressive Strength of Concrete Cylinders Cast-in-Place in Cylindrical Molds" (ASTM C 873[6.4]). (This method is limited to use in slabs where the depth of concrete is from 125 to 300 mm);

(b) Penetration resistance in accordance with "Standard Test Method for Penetration Resistance of Hardened Concrete" (ASTM C 803[6.5]);

(c) Pullout strength in accordance with "Standard Test Method for Pullout Strength of Hardened Concrete" (ASTM C 900[6.6]);

(d) Maturity factor measurements and correlation in accordance with ASTM C 1074.[6.7]

Procedures (b), (c), and (d) require sufficient data, using job materials, to demonstrate correlation of measurements on the structure with compressive strength of molded cylinders or drilled cores.

Where the structure is adequately supported on shores, the side forms of beams, girders, columns, walls, and similar vertical forms may generally be removed after 12 h of cumulative curing time, provided the side forms support no loads other than the lateral pressure of the plastic concrete. Cumulative curing time represents the sum of time intervals, not necessarily consecutive, during which the temperature of the air surrounding the concrete is above 10°C. The 12-h cumulative curing time is based on regular cements and ordinary conditions; the use of special cements or unusual conditions may require adjustment of the given limits. For example, concrete made with Type II or V (ASTM C 150) or ASTM C 595 cements, concrete containing retarding admixtures, and concrete to which ice was added during mixing (to lower the temperature of fresh concrete) may not have sufficient strength in 12 h and should be investigated before removal of formwork.

The removal of formwork for multistory construction should be a part of a planned procedure considering the temporary support of the whole structure as well as that of each individual member. Such a procedure should be worked out prior to construction and should be based on a structural analysis taking into account the following items, as a minimum:

(a) The structural system that exists at the various stages of construction and the construction loads corresponding to those stages;

(b) The strength of the concrete at the various ages during construction;

(c) The influence of deformations of the structure and shoring system on the distribution of dead loads and construction loads during the various stages of construction;

(d) The strength and spacing of shores or shoring systems used, as well as the method of shoring, bracing, shore removal, and reshoring including the minimum time intervals between the various operations;

(e) Any other loading or condition that affects the safety or serviceability of the structure during construction.

For multistory construction, the strength of the concrete during the various stages of construction should be substantiated by field-cured test specimens or other approved methods.

SECTION R6.3
CONDUITS AND PIPES EMBEDDED IN CONCRETE

R6.3.1 Conduits, pipes, and sleeves not harmful to concrete can be embedded within the concrete, but the work should be done in such a manner that the structure will not be endangered. Empirical rules are given in 6.3 for safe installations under common conditions; for other than common conditions, special designs should be made. The contractor should not be permitted to install conduits, pipes, ducts, or sleeves that are not shown on the plans or not approved by the engineer.

For the integrity of the structure, it is important that all conduit and pipe fittings within the concrete be carefully assembled as shown on the plans or called for in the job specifications.

R6.3.2 The SBC prohibits the use of aluminum in structural concrete unless it is effectively coated or covered. Aluminum reacts with concrete and, in the presence of chloride-ions, may also react electrolytically with steel, causing cracking and/or spatting of the concrete. Aluminum electrical conduits present a special problem since stray electric current accelerates the adverse reaction.

SECTION R6.4
CONSTRUCTION JOINTS

For the integrity of the structure, it is important that all construction joints be defined in construction documents and constructed as required. Any deviations should be approved by the engineer or architect.

R6.4.2 The use of neat cement on vertical joints is not permitted since it is rarely practical and can be detrimental where deep forms and steel congestion prevent proper access. Often wet blasting and other procedures are more appropriate. Because the SBC sets only minimum standards, the engineer may have to specify special procedures if conditions warrant. The degree to which mortar batches are needed at the start of concrete placement depends on concrete proportions, congestion of steel, vibrator access, and other factors.

R6.4.3 Construction joints should be located where they will cause the least weakness in the structure. When shear due to gravity load is not significant, as is usually the case in the middle of the span of flexural members, a simple vertical joint may be adequate. Lateral force design may require special design treatment of construction joints. Shear keys, intermittent shear keys, diagonal dowels, or the shear transfer method of 11.7 may be used whenever a force transfer is required.

R6.4.5 Delay in placing concrete in members supported by columns and walls is necessary to prevent cracking at the interface of the slab and supporting member caused by bleeding and settlement of plastic concrete in the supporting member.

R6.4.6 Separate placement of slabs and beams, haunches, and similar elements is permitted when shown on the drawings and where provision has been made to transfer forces as required in 6.4.3.

CHAPTER 7
DETAILS OF REINFORCEMENT

Recommended methods and standards for preparing design drawings, typical details, and drawings for the fabrication and placing of reinforcing steel in reinforced concrete structures are given in the ACI Detailing Manual (Reference 7.1).

All provisions in the SBC 304 relating to bar, wire, or strand diameter (and area) are based on the nominal dimensions of the reinforcement as given in the appropriate ASTM specification. Nominal dimensions are equivalent to those of a circular area having the same weight per meter as the ASTM designated bar, wire, or strand sizes. Cross-sectional area of reinforcement is based on nominal dimensions.

SECTION R7.1
STANDARD HOOKS

R7.1.3 Standard stirrup and tie hooks are limited to Dia 25 mm bars and smaller, and the 90-deg hook with $6d_b$ extension is further limited to Dia 16 mm bars and smaller, in both cases as the result of research showing that larger bar sizes with 90-deg hooks and $6d_b$ extensions tend to pop out under high load.

SECTION R7.2
MINIMUM BEND DIAMETERS

Standard bends in reinforcing bars are described in terms of the inside diameter of bend since this is easier to measure than the radius of bend. The primary factors affecting the minimum bend diameter are feasibility of bending without breakage and avoidance of crushing the concrete inside the bend.

R7.2.2 The minimum $4d_b$ bend for the bar sizes commonly used for stirrups and ties is based on accepted industry practice. Use of a stirrup bar size not greater than Dia 16 mm for either the 90-deg or 135-deg standard stirrup hook will permit multiple bending on standard stirrup bending equipment.

R7.2.3 Welded wire fabric, of plain or deformed wire, can be used for stirrups and ties. The wire at welded inter-sections does not have the same uniform ductility and bend-ability as in areas which were not heated. These effects of the welding temperature are usually dissipated in a distance of approximately four wire diameters. Minimum bend diameters permitted are in most cases the same as those required in the ASTM bend tests for wire material (ASTM A 82 and A 496).

SECTION R7.3
BENDING

R7.3.1 The engineer may be the design engineer or the engineer employed by the owner to perform inspection. For unusual bends with inside diameters less than ASTM bend test requirements, special fabrication may be required.

R7.3.2 Construction conditions may make it necessary to bend bars that have been embedded in concrete. Such field bending should not be done without authorization of the engineer. The engineer should determine whether the bars should be bent cold or if heating should be used. Bends should be gradual and should be straightened as required.

Tests[7.2, 7.3] have shown that A 615M Grade 300 and Grade 420 reinforcing bars can be cold bent and straightened up to 90 deg at or near the minimum diameter specified in 7.2. If cracking or breakage is encountered, heating to a maximum temperature of 800°C may avoid this condition for the remainder of the bars. Bars that fracture during bending or straightening can be spliced outside the bend region.

Heating should be performed in a manner that will avoid damage to the concrete. If the bend area is within approximately 150 mm of the concrete, some protective insulation may need to be applied. Heating of the bar should be controlled by temperature-indicating crayons or other suitable means. The heated bars should not be artificially cooled (with water or forced air) until after cooling to at least 300°C.

SECTION R7.4
SURFACE CONDITIONS OF REINFORCEMENT

R7.4.3 Guidance for evaluating the degree of rusting on strand is given in Reference 7.4.

SECTION R7.5
PLACING REINFORCEMENT

R7.5.1 Reinforcement, including tendons, and post-tensioning ducts should be adequately supported in the forms to prevent displacement by concrete placement or workers. Beam stirrups should be supported on the bottom form of the beam by positive supports such as continuous longitudinal beam bolsters. If only the longitudinal beam bottom reinforcement is supported, construction traffic can dislodge the stirrups as well as any prestressing tendons tied to the stirrups.

R7.5.2 Generally accepted practice, as reflected in Ref. 7.5 has established tolerances on total depth (formwork or finish) and fabrication of truss bent reinforcing bars and closed ties, stirrups, and spirals. The engineer should specify more restrictive tolerances than those permitted by the SBC 304 when necessary to minimize the accumulation of tolerances resulting in excessive reduction in effective depth or cover.

More restrictive tolerances have been placed on minimum clear distance to formed soffits because of its importance for durability and fire protection, and because bars are usually supported in such a manner that the specified tolerance is practical.
More restrictive tolerances than those required by the SBC 304 may be desirable for prestressed concrete to achieve camber control within limits acceptable to the designer or owner. In such cases, the engineer should specify the necessary tolerances. Recommendations are given in Ref. 7.6.

R7.5.2.1 The SBC 304 specifies a tolerance on depth d, an essential component of strength of the member. Because reinforcing steel is placed with respect to edges of members and formwork surfaces, the depth d is not always conveniently measured in the field. Engineers should specify tolerances for bar placement, cover, and member size. See ACI 117.[7.5]

R7.5.4 "Tack" welding (welding crossing bars) can seriously weaken a bar at the point welded by creating a metallurgical notch effect. This operation can be performed safely only when the material welded and welding operations are under continuous competent control, as in the manufacture of welded wire fabric.

SECTION R7.6
SPACING LIMITS FOR REINFORCEMENT

Since the development length is a function of the bar spacing, it may be desirable to use larger than minimum bar spacing in some cases. The minimum limits are established to permit concrete to flow readily into spaces between bars and between bars and forms without honeycomb, and to ensure against concentration of bars on a line that may cause shear or shrinkage cracking.

R7.6.6 **Bundled bars.** Bond research[7.7] showed that bar cutoffs within bundles should be staggered. Bundled bars should be tied, wired, or otherwise fastened together to ensure remaining in position whether vertical or horizontal.

A limitation that bars larger than Dia 32 mm not be bundled in beams or girders is a practical limit for application to building size members. Conformance to the crack control requirements of 10.6 will effectively preclude bundling of bars larger than Dia 32 mm as tensile reinforcement. The SBC phrasing "bundled in contact to act as a unit," is intended to preclude bundling more than two bars in the same plane. Typical bundle shapes are triangular, square, or L-shaped patterns for three- or four-bar bundles. As a practical caution, bundles more than one bar deep in the plane of bending should not be hooked or bent as a unit. Where end hooks are required, it is preferable to stagger the individual bar hooks within a bundle.

R7.6.7 **Tendons and ducts**

R7.6.7.1 The allowed decreased spacing in this section for transfer strengths of 28 MPa or greater is based on Reference 7.8, 7.9.

R7.6.7.2 When ducts for prestressing steel in a beam are arranged closely together vertically, provision should be made to prevent the prestressing steel from breaking through the duct when tensioned. Horizontal disposition of ducts should allow proper placement of concrete. A clear spacing of one and one-third times the size of the coarse aggregate, but not less than 25 mm, has proven satisfactory. Where concentration of tendons or ducts tends to create a weakened plane in the concrete cover, reinforcement should be provided to control cracking.

SECTION R7.7
CONCRETE PROTECTION FOR REINFORCEMENT

Concrete cover as protection of reinforcement against weather and other effects is measured from the concrete surface to the outermost surface of the steel to which the cover requirement applies. Where minimum cover is prescribed for a class of structural member, it is measured to the outer edge of stirrups, ties, or spirals if transverse reinforcement encloses main bars; to the outermost layer of bars if more than one layer is used without stirrups or ties; or to the metal end fitting or duct on post-tensioned prestressing steel.

The condition "concrete surfaces exposed to earth or weather" refers to direct exposure to moisture changes and not just to temperature changes. Slab or thin shell soffits are not usually considered directly exposed unless subject to alternate wetting and drying, including that due to condensation conditions or direct leakage from exposed top surface, run off, or similar effects.

Alternative methods of protecting the reinforcement from weather may be provided if they are equivalent to the additional concrete cover required by the SBC 304. When approved by the building official under the provisions of 1.4, reinforcement with alternative protection from the weather may have concrete cover not less than the cover required for reinforcement not exposed to weather.

The development lengths given in Chapter 12 are now a function of the bar cover. As a result, it may be desirable to use larger than minimum cover in some cases.

R7.7.3 **Precast concrete (manufactured under plant control conditions).** The lesser cover thicknesses for precast construction reflect the greater convenience of control for proportioning, placing, and curing inherent in precasting. The term "manufactured under plant control conditions" does not specifically imply that precast members should be manufactured in a plant. Structural elements precast at the job site will also qualify under this section if the control of form dimensions, placing of reinforcement, quality control of concrete, and curing procedure are equal to that normally expected in a plant.

Concrete cover to pretensioned strand as described in this section is intended to provide minimum protection against weather and other effects. Such cover may not be sufficient to transfer or develop the stress in the strand, and it may be necessary to increase the cover accordingly.

R7.7.5 **Corrosive environments.** Where concrete will be exposed to external sources of chlorides in service, such as brackish water, seawater, or spray from these sources, concrete should be proportioned to satisfy the special exposure requirements of Chapter 4. These include maximum water-cementitious materials ratio, minimum strength for normal weight and lightweight concrete, maximum chloride ion in concrete, and cement type. Additionally, for corrosion protection, a minimum concrete cover for reinforcement of 50 mm for walls and slabs and 60 mm for other members is recommended. For precast concrete manufactured under plant control conditions, a minimum cover of 40 and 50 mm, respectively, is recommended.

R7.7.5.1 Corrosive environments are defined in Chapter 4 of this SBC 304. Additional information on corrosion in parking structures is given in Ref. 7.10, "Design of Parking Structures," pp. 21-26.

R7.7.6 **Future extension.** Exposed reinforcements, inserts and plates, etc, intended for bonding with future extensions, should be adequately protected against corrosion and any other corrosive ambient attack. The protection measures can include but may not be limited to the following:

a) for accessible roofs, the column rebars should be bent inside the roof screeds in such a manner that they can be re-exposed and lapped for future extensions, as and when necessary.

b) for non-accessible roofs, the column rebars can be covered inside low-height stub columns of lean concrete, in such manner so that they can be re-exposed and lapped for future extensions, as and when necessary.

c) for all other elements like beams, slab projections, etc. the rebars should desirably be protected in lean concrete, in such manner so that they can be re-exposed and lapped for future extensions, as and when necessary.

SECTION R7.8
SPECIAL REINFORCEMENT DETAILS FOR COLUMNS

R7.8.2 **Steel cores**. The 50 percent limit on transfer of compressive load by end bearing on ends of structural steel cores is intended to provide some tensile capacity at such splices (up to 50 percent), since the remainder of the total compressive stress in the steel core are to be transmitted by dowels, splice plates, welds, etc. This provision should ensure that splices in composite compression members meet essentially the same tensile capacity as required for conventionally reinforced concrete compression members.

SECTION R7.9
CONNECTIONS

Confinement is essential at connections to ensure that the flexural capacity of the members can be developed without deterioration of the joint under repeated loadings.[7.11,7.12]

SECTION R7.10
LATERAL REINFORCEMENT FOR COMPRESSION MEMBERS

R7.10.3 Precast columns with cover less than 40 mm, prestressed columns without longitudinal bars, columns of concrete with small size coarse aggregate, wall-like columns, and other special cases may require special designs for lateral reinforcement. Plain or deformed wire, WD 5.5, or larger, may be used for ties or spirals. If such special columns are considered as spiral columns for load strength in design, the ratio of spiral reinforcement ρ_s is to conform to 10.9.3.

R7.10.4 **Spirals.** For practical considerations in cast-in-place construction, the minimum diameter of spiral reinforcement is 10 mm. This is the smallest size that can be used in a column with 40 mm or more cover and having concrete strengths of 20 MPa or more if the minimum clear spacing for placing concrete is to be maintained.

Standard spiral sizes are 10, 14, and 16 mm diameter for hot rolled or cold drawn material, plain or deformed.

The SBC 304 allows spirals to be terminated at the level of lowest horizontal reinforcement framing into the column. However, if one or more sides of the column are not enclosed by beams or brackets, ties are required from the termination of the spiral to the bottom of the slab or drop panel. If beams or brackets enclose all sides of the column but are of different depths, the ties should extend from the spiral to the level of the horizontal reinforcement of the shallowest beam or bracket framing into the column. These additional ties are to enclose the longitudinal column reinforcement and the portion of bars from beams bent into the column for anchorage. See also 7.9.

Spirals should be held firmly in place, at proper pitch and alignment, to prevent displacement during concrete placement. When spacers are used, the following may be used for guidance: For spiral bar or wire smaller than 16 mm diameter, a minimum of two spacers should be used for spirals less than 0.5 m in diameter, three pacers for spirals 0.5 to 0.75 m in diameter, and four spacers for spirals greater than 0.75 m in diameter. For spiral bar or wire 16 mm diameter or larger, a minimum of three spacers should be used for spirals 600 mm or less in diameter, and four spacers for spirals greater than 600 mm in diameter. The project specifications or subcontract agreements should be clearly written to cover the supply of spacers or field tying of the spiral reinforcement.

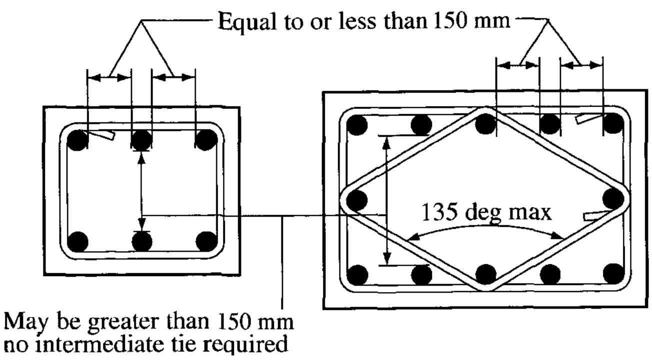


Fig. R7.10.5 - Sketch to clarify measurements between laterally supported column bars.

R7.10.5 **Ties.** All longitudinal bars in compression should be enclosed within lateral ties. Where longitudinal bars are arranged in a circular pattern, only one circular tie per specified spacing is required. This requirement can be satisfied by a continuous circular tie (helix) at larger pitch than required for spirals under 10.9.3, the maximum pitch being equal to the required tie spacing (see also 7.10.4.3).

Since spliced bars and bundled bars were not included in the tests of Reference 7.13, is prudent to provide a set of ties at each end of lap spliced bars, above and below end-bearing splices, and at minimum spacings immediately below sloping regions of offset bent bars.

Standard tie hooks are intended for use with deformed bars only, and should be staggered where possible. See also 7.9.

Continuously wound bars or wires can be used as ties provided their pitch and area are at least equivalent to the area and spacing of separate ties. Anchorage at the end of a continuously wound bar or wire should be by a standard hook as for separate bars or by one additional turn of the tie pattern. A circular continuously wound bar or wire is considered a spiral if it conforms to 7.10.4, otherwise it is considered a tie.

R7.10.5.5 Ties may be terminated only when elements frame into all four sides of square and rectangular columns; for round or polygonal columns, such elements frame into the column from four directions

R7.10.5.6 Confinement of anchor bolts that are placed in the top of columns or pedestal improves load transfer from the anchor bolts to the column or pier for situations where the concrete cracks in the vicinity of the bolts. Such cracking can occur due to unanticipated forces caused by temperature, restrained shrinkage, and similar effects

SECTION R7.11
LATERAL REINFORCEMENT FOR FLEXURAL MEMBERS

R7.11.1 Compression reinforcement in beams and girders should be enclosed to prevent buckling; similar requirements for such enclosure have remained essentially unchanged through several editions of the SBC 304, except for minor clarification.

SECTION R7.12
SHRINKAGE AND TEMPERATURE REINFORCEMENT

R7.12.1 Shrinkage and temperature reinforcement is required at right angles to the principal reinforcement to minimize cracking and to tie the structure together to ensure its acting as assumed in the design. The provisions of this section are intended for structural slabs only; they are not intended for soil supported slabs on grade.

R7.12.1.2 The area of shrinkage and temperature reinforcement required by 7.12 has been satisfactory where shrinkage and temperature movements are permitted to occur. For cases where structural walls or large columns provide significant restraint to shrinkage and temperature movements, it may be necessary to increase the amount of reinforcement normal to the flexural reinforcement in 7.12.1.2 (see Reference 7.14). Top and bottom reinforcement are both effective in controlling cracks. Control strips during the construction period, which permit initial shrinkage to occur without causing an increase in stresses, are also effective in reducing cracks caused by restraint.

R7.12.2 The amounts specified given for deformed bars and welded wire fabric are empirical but have been used satisfactorily for many years. Splices and end anchorages of shrinkage and temperature reinforcement are to be designed for the full specified yield strength in accordance with 12.1, 12.15, 12.18, and 12.19.

R7.12.3 Prestressed reinforcement requirements have been selected to provide an effective force on the slab approximately equal to the yield strength force for nonprestressed shrinkage and temperature reinforcement. This amount of prestressing, 1.0 MPa on the gross concrete area, has been successfully used on a large number of projects. When the spacing of tendons used for shrinkage and temperature reinforcement exceeds 1.4 m, additional bonded reinforcement is required at slab edges where the prestressing forces are applied in order to adequately reinforce the area between the slab edge and the point where compressive stresses behind individual anchorages have spread sufficiently such that the slab is uniformly in compression. Application of the provisions of 7.12.3 to monolithic cast in-place post-tensioned beam and slab construction is illustrated in Fig. R7.12.3.

Tendons used for shrinkage and temperature reinforcement should be positioned vertically in the slab as close as practicable to the center of the slab. In cases where the shrinkage and temperature tendons are used for supporting the principal tendons, variations from the slab centroid are permissible; however, the resultant of the shrinkage and temperature tendons should not fall outside the kern area of the slab.

The designer should evaluate the effects of slab shortening to ensure proper action. In most cases, the low level of prestressing recommended should not cause difficulties in a properly detailed structure. Special attention may be required where thermal effects become significant.

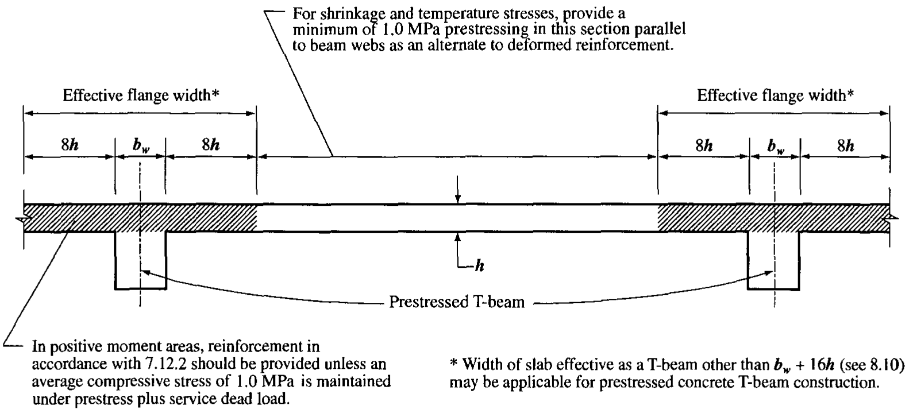


Fig. R7.12.3 - Prestressing used for shrinkage and temperature.

SECTION R7.13
REQUIREMENTS FOR STRUCTURAL INTEGRITY

Experience has shown that the overall integrity of a structure can be substantially enhanced by minor changes in detailing of reinforcement. It is the intent of this section of the SBC 304 to improve the redundancy and ductility in structures so that in the event of damage to a major supporting element or an abnormal loading event, the resulting damage may be confined to a relatively small area and the structure will have a better chance to maintain overall stability.

R7.13.2 With damage to a support, top reinforcement hat is continuous over the support, but not confined by stirtrups, will tend to tear out of the concrete and will not provide the catenary action needed to bridge the damaged support. By making a portion of the bottom reinforcement continuous, catenary action can be provided.

Requiring continuous top and bottom reinforcement in perimeter or spandrel beams provides a continuous tie around the structure. It is not the intent to require a tensile tie of continuous reinforcement of constant size around the entire perimeter of a structure, but simply to require that one half of the top flexural reinforcement required to extend past the point of inflection by 12.12.3 be further extended and spliced at or near midspan. Similarly, the bottom reinforcement required to extend into the support by 12.11.1 should be made continuous or spliced with bottom reinforcement from the adjacent span. If the depth of a continuous beam changes at a support, the bottom reinforcement in the deeper member should be terminated with a standard hook and bottom reinforcement in the shallower member should be extended into and fully developed in the deeper member.

R7.13.3 The SBC 304 requires tension ties for precast concrete buildings of all heights. Details should provide connections to resist applied loads. Connection details that rely solely on friction caused by gravity forces are not permitted.

Connection details should be arranged so as to minimize the potential for cracking due to restrained creep, shrinkage and temperature movements. For information on connections and detailing requirements, see Reference 7.15.

Ref. 7.16 recommends minimum tie requirements for precast concrete bearing wall buildings.

CHAPTER 8
ANALYSIS AND DESIGN GENERAL CONSIDERATIONS

SECTION R8.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

The definition of net tensile strain in 2.1 excludes strains due to effective prestress, creep, shrinkage, and temperature.

SECTION R8.1
DESIGN METHODS

R8.1.1 The strength design method requires service loads or related internal moments and forces to be increased by specified load factors (required strength) and computed nominal strengths to be reduced by specified strength reduction factors (design strength).

R8.1.2 Designs in accordance with Appendix B are equally acceptable, provided the provisions of Appendix B are used in their entirety.

R8.1.3 The SBC 304 includes specific provisions for anchoring to concrete in Appendix D of SBC 304.

SECTION R8.2
LOADING

The provisions in the SBC 304 are for all types of loads as specified in SBC 301.
Roofs should be designed with sufficient slope or camber to ensure adequate drainage accounting for any long-term deflection of the roof due to the dead loads, or the loads should be increased to account for all likely accumulations of water. If deflection of roof members may result in ponding of water accompanied by increased deflection and additional ponding, the design should ensure that this process is self-limiting.

R8.2.3 Any reinforced concrete wall that is monolithic with other structural elements is considered to be an "integral part." Partition walls may or may not be integral structural parts. If partition walls may be removed, the primary lateral load resisting system should provide all of the required resistance without contribution of the removable partition. However, the effects of all partition walls attached to the structure should be considered in the analysis of the structure because they may lead to increased design forces in some or all elements. Special provisions for seismic design are given in Chapter 21.

R8.2.4 Information is accumulating on the magnitudes of these various effects, especially the effects of column creep and shrinkage in tall structures,[8.1] and on procedures for including the forces resulting from these effects in design.

SECTION R8.3
METHOD OF ANALYSIS

R8.3.1 Factored loads are service loads multiplied by appropriate load factors. For the strength design method, elastic analysis is used to obtain moments, shears, and reactions.

R8.3.3 The approximate moments and shears give reasonably conservative values for the stated conditions if the flexural members are part of a frame or continuous construction. Because the load patterns that produce critical values for moments in columns of frames differ from those for maximum negative moments in beams, column moments should be evaluated separately.

These moment coefficients were derived by elastic analysis assuming rigid supports and considering alternative placement of live load to yield maximum negative or positive moments at the critical sections. For one-way slab systems, a supporting beam or girder may be considered rigid if it is supported by columns or walls and has a total depth not less than about three times the solid slab thickness or three times the equivalent joist-slab thickness.

R8.3.4 The strut-and-tie model in Appendix A is based on the assumption that portions of concrete structures can be analyzed and designed using hypothetical pin-jointed trusses consisting of struts and ties connected at nodes. This design method can be used in the design of regions where the basic assumptions of flexure theory are not applicable, such as regions near force discontinuities arising from concentrated forces or reactions, and regions near geometric discontinuities, such as abrupt changes in cross section.

SECTION R8.4
REDISTRIBUTION OF NEGATIVE MOMENTS IN CONTINUOUS FLEXURAL MEMBERS

Moment redistribution is dependent on adequate ductility in plastic hinge regions. These plastic hinge regions develop at points of maximum moment and cause a shift in the elastic moment diagram. The usual result is a reduction in the values of negative moments in the plastic hinge region and an increase in the values of positive moments from those computed by elastic analysis. Because negative moments are determined for one loading arrangement and positive moments for another, each section has a reserve capacity that is not fully utilized for any one loading condition. The plastic hinges permit the utilization of the full capacity of more cross sections of a flexural member at ultimate loads.

Using conservative values of limiting concrete strains and lengths of plastic hinges derived from extensive tests, flexural members with small rotation capacity were analyzed for moment redistribution up to 20 percent, depending on the reinforcement ratio. The results were found to be conservative. Studies (reference 8.2 and 8.3) support this conclusion and indicate that cracking and deflection of beams designed for moment redistribution are not significantly greater at service loads than for beams designed by the elastic theory distribution of moments. Also, these studies indicated that adequate rotation capacity for the moment redistribution allowed by the SBC 304 is available if the members satisfy the SBC 304 requirements.

Moment redistribution may not be used for slab systems designed by the Direct Design Method (see 13.6.1.7).

The SBC 304 specifies the permissible redistribution percentage in terms of the net tensile strain ε_t. Reference 8.4 compares moment redistribution percentage in terms of reinforcement indices and the net tensile strain ε_t.

SECTION R8.5
MODULUS OF ELASTICITY

R8.5.1 Studies leading to the expression for modulus of elasticity of concrete in 8.5.1 are summarized in Reference 8.5 where, E_c, was defined as the slope of the line drawn from a stress of zero to a compressive stress of $0.45 f_c^{'}$. The modulus for concrete is sensitive to the modulus of the aggregate and may differ from the specified value. Measured values range typically from 120 to 80 percent of the specified value. Methods for determining Young's modulus for concrete are described in Reference 8.6.

SECTION R8.6
STIFFNESS

R8.6.1 Ideally, the member stiffnesses *EI* and *GJ* should reflect the degree of cracking and inelastic action that has occurred along each member before yielding. However, the complexities involved in selecting different stiffnesses for all members of a frame would make frame analyses inefficient in design offices. Simpler assumptions are required to define flexural and torsional stiffnesses. For braced frames, relative values of stiffness are important. Two usual assumptions are to use gross *EI* values for all members or, to use half the gross *EI* of the beam stem for beams and the gross *EI* for the columns. For frames that are free to sway, a realistic estimate of *EI is* desirable and should be used if second-order analyses are carried out. Guidance for the choice of *EI* for this case is given in R10.11.1.

Two conditions determine whether it is necessary to consider torsional stiffness in the analysis of a given structure: (1) the relative magnitude of the torsional and flexural stiffnesses, and (2) whether torsion is required for equilibrium of the structure (equilibrium torsion) or is due to members twisting to maintain deformation compatibility (compatibility torsion). In the case of compatibility torsion, the torsional stiffness may be neglected. For cases involving equilibrium torsion, torsional stiffness should be considered.

R8.6.2 Stiffness and fixed-end moment coefficients for haunched members may be obtained from Reference 8.7.

SECTION R8.7
SPAN LENGTH

Beam moments calculated at support centers may be reduced to the moments at support faces for design of beams. Reference 8.8 provides an acceptable method of reducing moments at support centers to those at support faces.

SECTION R8.8
COLUMNS

Section 8.8 has been developed with the intent of making certain that the most demanding combinations of axial load and moments be identified for design.
Section 8.8.4 has been included to make certain that moments in columns are recognized in the design if the girders have been proportioned using 8.3.3. The moment in 8.8.4 refers to the difference between the moments in a given vertical plane, exerted at column centerline by members framing into that column.

SECTION R8.9
ARRANGEMENT OF LIVE LOAD

For determining column, wall, and beam moments and shears caused by gravity loads, the SBC 304 permits the use of a model limited to the beams in the level considered and the columns above and below that level. Far ends of columns are to be considered as fixed for the purpose of analysis under gravity loads. This assumption does not apply to lateral load analysis. However in analysis for lateral loads, simplified methods (such as the portal method) may be used to obtain the moments, shears, and reactions for structures that are symmetrical and satisfy the assumptions used for such simplified methods. For unsymmetrical and high-rise structures, rigorous methods recognizing all structural displacements should be used.

The engineer is expected to establish the most demanding sets of design forces by investigating the effects of live load placed in various critical patterns.
Most approximate methods of analysis neglect effects of deflections on geometry and axial flexibility. Therefore, beam and column moments may have to be amplified for column slenderness in accordance with 10.11, 10.12, and 10.13.

SECTION R8.10
T- BEAM CONSTRUCTION

Special provisions related to T-beams and other flanged members are stated in 11.6.1 with regard to torsion.

SECTION R8.11
JOIST CONSTRUCTION

The size and spacing limitations for concrete joist construction meeting the limitations of 8.11.1 through 8.11.3 are based on practice.

R8.11.3 A limit on the maximum spacing of ribs is required because of the special provisions permitting higher shear strengths and less concrete protection for the reinforcement for these relatively small, repetitive members.

R8.11.8 The increase in shear strength permitted by 8.11.8 is justified on the basis of:

(1) satisfactory performance of joist construction with higher shear strengths and,

(2) redistribution of local overloads to adjacent joists.

SECTION R8.12
SEPARATE FLOOR FINISH

The SBC 304 does not specify an additional thickness for wearing surfaces subjected to unusual conditions of wear. The need for added thickness for unusual wear is left to the discretion of the designer.

A floor finish may be considered for strength purposes only if it is cast monolithically with the slab. Permission is given to include a separate finish in the structural thickness if composite action is provided for in accordance with Chapter 17.

All floor finishes may be considered for nonstructural purposes such as cover for reinforcement, fire protection, etc. Provisions should be made, however, to ensure that the finish will not spall off, thus causing decreased cover. Furthermore, development of reinforcement considerations require minimum monolithic concrete cover according to 7.7.

CHAPTER 9
STRENGTH AND SERVICEABILITY REQUIREMENTS

SECTION R9.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

The definition of net tensile strain in 2.1 excludes strains due to effective prestress, creep, shrinkage, and temperature.

SECTION R 9.1
GENERAL

Chapter 9 defines the basic strength and serviceability conditions for proportioning structural concrete members. The basic requirement for strength design may be expressed as follows:

$$\text{Design Strength} \geq \text{Required Strength}$$
$$\phi \text{ (Nominal Strength)} \geq U$$

In the strength design procedure, the margin of safety is provided by multiplying the service load by a load factor and the nominal strength by a strength reduction factor.

SECTION R9.2
REQUIRED STRENGTH

The required strength U is expressed in terms of factored loads, or related internal moments and forces. Factored loads are the loads specified in the Saudi building SBC 304 for loading multiplied by appropriate load factors.

The factor assigned to each load is influenced by the degree of accuracy to which the load effect usually can be calculated and the variation that might be expected in the load during the lifetime of the structure. Dead loads, because they are more accurately determined and less variable, are assigned a lower load factor than live loads. Load factors also account for variability in the structural analysis used to compute moments and shears.

The SBC 304 gives load factors for specific combinations of loads. In assigning factors to combinations of loading, some consideration is given to the probability of simultaneous occurrence. While most of the usual combinations of loadings are included, the designer should not assume that all cases are covered.

Due regard is to be given to sign in determining U for combinations of loadings, as one type of loading may produce effects of opposite sense to that produced by another type. The load combinations with $0.9D$ are specifically included for the case where a higher dead load reduces the effects of other loads. The loading case may also be critical for tension-controlled column sections. In such a case, a reduction in axial load and an increase in moment may result in a critical load combination.

Consideration should be given to various combinations of loading to determine the most critical design condition. This is particularly true when strength is dependent on more than one load effect, such as strength for combined flexure and axial load or shear strength in members with axial load.

If special circumstances require greater reliance on the strength of particular members than encountered in usual practice, some reduction in the stipulated strength reduction factors ϕ or increase in the stipulated load factors U may be appropriate for such members.

R9.2.2 If the live load is applied rapidly, as may be the case for parking structures, loading docks, warehouse floors, elevator shafts, etc., impact effects should be considered. In all equations, substitute (L + impact) for L when impact should be considered.

R9.2.3 The designer should consider the effects of differential settlement, creep, shrinkage, temperature, and shrinkage-compensating concrete. The term realistic assessment is used to indicate that the most probable values rather than the upper bound values of the variables should be used.

R9.2.4 The load factor of 1.2 applied to the maximum tendon jacking force results in a design load of about 113 percent of the specified prestressing steel yield strength but not more than 96 percent of the nominal ultimate strength of the prestressing steel. This compares well with the maximum attainable jacking force, which is limited by the anchor efficiency factor.

SECTION R9.3
DESIGN STRENGTH

R9.3.1 The design strength of a member refers to the nominal strength calculated in accordance with the requirements stipulated in the SBC 304 multiplied by a strength reduction factor ϕ, which is always less than one.

The purposes of the strength reduction factor ϕ are (1) to allow for the probability of understrength members due to variations in material strengths and dimensions, (2) to allow for inaccuracies in the design equations, (3) to reflect the degree of ductility and required reliability of the member under the load effects being considered, and (4) to reflect the importance of the member in the structure.[9.1,9.2]

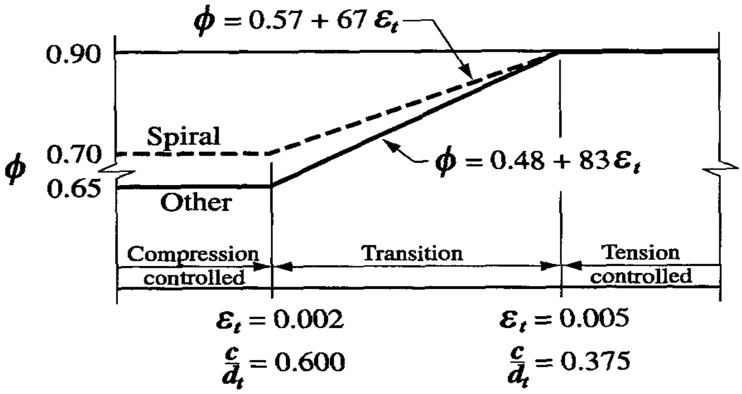
R9.3.2.1 In applying 9.3.2.1 and 9.3.2.2, the axial tensions and compressions to be considered are those caused by external forces. Effects of prestressing forces are not included.

R9.3.2.2 The magnitude of the ϕ factor is determined by the strain conditions at a cross section, at nominal strength.

A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections. Members with spiral reinforcement are assigned a higher ϕ than tied columns since they have greater ductility or toughness.

For sections subjected to axial load with flexure, design strengths are determined by multiplying both P_n and M_n by the appropriate single value of ϕ. Compression-controlled and tension-controlled sections are defined in 10.3.3 and

10.3.4 as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than 0.005, respectively. For sections with net tensile strain ε_t in the extreme tension steel at nominal strength between the above limits, the value of ϕ may be determined by linear interpolation, as shown in Fig. R9.3.2. The concept of net tensile strain ε_t is discussed in R10.3.3.



Interpolation on c/d_t: Spiral $\phi = 0.37 + 0.20/(c/d_t)$
Other $\phi = 0.23 + 0.25/(c/d_t)$

Fig. R9.3.2 - Variation of ϕ with net tensile ε_t and c/d_t for Grade 420 reinforcement and for prestressing steel.

Since the compressive strain in the concrete at nominal strength is assumed in 10.2.3 to be 0.003, the net tensile strain limits for compression-controlled members may also be stated in terms of the ratio c/d_t, where c is the depth of the neutral axis at nominal strength, and d_t is the distance from the extreme compression fiber to the extreme tension steel. The c/d_t limits for compression-controlled and tension-controlled sections are 0.6 and 0.375, respectively. The 0.6 limit applies to sections reinforced with Grade 420 steel and to prestressed sections. Fig. R9.3.2 also gives equations for ϕ as a function of c/d_t.

The net tensile strain limit for tension-controlled sections may also be stated in terms of the ρ/ρ_b. The net tensile strain limit of 0.005 corresponds to a ρ/ρ_b ratio of 0.63 for rectangular sections with Grade 420 reinforcement.

R9.3.2.5 The ϕ factor of 0.85 reflects the wide scatter of results of experimental anchorage zone studies. Since 18.13.4.2, limits the nominal compressive strength of unconfined concrete: in the general zone to $0.7\lambda f'_{ci}$ the effective design strength for unconfined concrete is $0.85 \times 0.7\lambda f'_{ci} = 0.6\lambda f'_{ci}$.

R9.3.2.6 The ϕ factor used in strut-and-tie models is taken equal to the ϕ factor for shear. The value of 4, for strut and-tie models is applied to struts, ties, and bearing areas in such models.

R9.3.2.7 If a critical section occurs in a region where strand is not fully developed, failure may be by bond slip. Such a failure resembles a brittle shear failure; hence, the requirements for a reduced ϕ.

R9.3.4 Strength reduction factors in 9.3.4 are intended to compensate for uncertainties in estimation of strength of structural members in buildings. They are based primarily on experience with constant or steadily increasing applied load. For construction in regions of high seismic risk, some of the strength reduction factors have been modified in 9.3.4 to account for the effects of displacement reversals into the nonlinear range of response on strength.

Section 9.3.4(a) refers to brittle members such as low-rise walls, portions of walls between openings, or diaphragms that are impractical to reinforce to raise their nominal shear strength above nominal flexural strength for the pertinent loading conditions.

Short structural walls were the primary vertical elements of the lateral-force-resisting system in many of the parking structures that sustained damage during the 1994 Northridge earthquake. Section 9.3.4(b) requires the shear strength reduction factor for diaphragms to be 0.60 if the shear strength reduction factor for the walls is 0.60.

R9.3.5 The strength reduction factor ϕ for structural plain concrete design is the same for all strength conditions. Since both flexural tension strength and shear strength for plain concrete depend on the tensile strength characteristics of the concrete, with no reserve strength or ductility possible due to the absence of reinforcement, equal strength reduction factors for both bending and shear are considered appropriate.

SECTION R9.4
DESIGN STRENGTH FOR REINFORCEMENT

In addition to the upper limit of 550 MPa for yield strength of non-prestressed reinforcement, there are limitations on yield strength in other sections of the SBC 304.

In 11.5.2, 11.6.3.4, and 11.7.6, the maximum f_y that may be used in design for shear and torsion reinforcement is 420 MPa, except that f_y up to 550 MPa may be used for shear reinforcement meeting the requirements of ASTM A 497.

In 19.3.2 and 21.2.5, the maximum specified f_y is 420 MPa in shells, folded plates, and structures governed by the special seismic provisions of Chapter 21.

The deflection provisions of 9.5 and the limitations on distribution of flexural reinforcement of 10.6 become increasingly critical as f_y increases.

SECTION R9.5
CONTROL OF DEFLECTIONS

R9.5.1 The provisions of 9.5 are concerned only with deflections or deformations that may occur at service load levels. When long-term deflections are computed, only the dead load and that portion of the live load that is sustained need be considered.

Two methods are given for controlling deflections.[9.3] For nonprestressed beams and one-way slabs, and for composite members, provision of a minimum overall thickness as required by Table 9.5(a) will satisfy the requirements of the SBC 304 for members not supporting or attached to partitions or other construction likely to be damaged by large deflections.

For nonprestressed two-way construction, minimum thickness as required by 9.5.3.1, 9.5.3.2, and 9.5.3.3 will satisfy the requirements of the SBC 304.

For nonprestressed members that do not meet these minimum thickness requirements, or that support or are attached to partitions or other construction likely to be damaged by large deflections, and for all prestressed concrete flexural members, deflections should be calculated by the procedures described or referred to in the appropriate sections of the SBC 304, and are limited to the values in Table 9.5(b).

TABLE 9.5(a)-MINIMUM THICKNESS OF NON-PRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED

		Minimum thickness, h		
	Simply supported	**One end continuous**	**Both ends continuous**	**Cantilever**
Member	**Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.**			
Solid one-way slabs	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$

Notes:
1) Span length ℓ is in mm.
2) Values given shall be used directly for members with normal weight concrete (w_c=2300 kg/m^3) and Grade 420 reinforcement For other conditions, the values shall be modified as follows:
   a) For structural lightweight concrete having unit weight in the range 1500-2000 kg/m³ , the values shall be multiplied by (1.65 - 0.0003w_c) but not less than 1.09, where w_c is the unit weight in kg/m^3 .
   b) For f_y other than 420 MPa, the values shall be multiplied by (0.4 + f_y/700).

R9.5.2 One-way construction (nonprestressed)

R9.5.2.1 The minimum thicknesses of Table 9.5(a) apply for nonprestressed beams and one-way slabs (see 9.5.2), and for composite members (see 9.5.5). These minimum thicknesses apply only to members not supporting or attached to partitions and other construction likely to be damaged by deflection.

Values of minimum thickness should be modified if other than normal weight concrete and Grade 420 reinforcement are used. The notes beneath the table are essential to its use for reinforced concrete members constructed with structural lightweight concrete or with reinforcement having a yield strength other than 420 MPa. If both of these conditions exist, the corrections in footnotes (a) and (b) should both be applied.

The modification for lightweight concrete in footnote (a) is based on studies of the results and discussions in Reference 9.14. No correction is given for concretes weighing between 1900 and 2300 kg/m^3 because the correction term would be close to unity in this range.

The modification for yield strength in footnote (b) is approximate but should yield conservative results for the type of members considered in the table, for typical reinforcement ratios, and for values of f_y between 300 and 550 MPa.

R9.5.2.2 For calculation of immediate deflections of uncracked prismatic members, the usual methods or formulas for elastic deflections may be used with a constant value of $E_c I_g$ along the length of the member. However, if the member is cracked at one or more sections, or if its depth varies along the span, a more exact calculation becomes necessary.

R9.5.2.3 The effective moment of inertia procedure described in the SBC 304 and developed in Reference 9.5 was selected as. being sufficiently accurate for use to control deflections.[9.6-9.8] The effective I_e was developed to provide a transition between the upper and lower bounds of I_g and I_{cr} as a function of the ratio M_{cr} / M_a. For most cases I_e will be less than I_g.

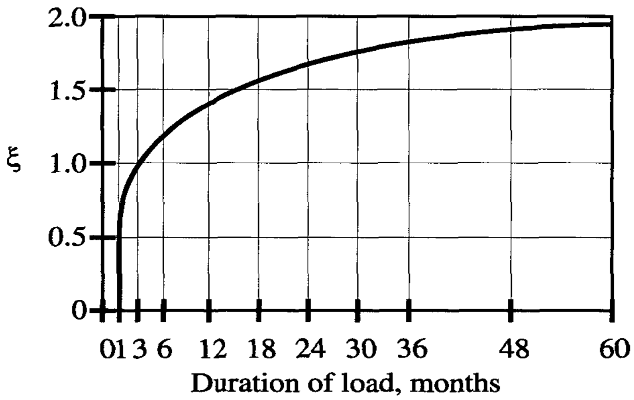


Fig. R9.5.2.5-Multipliers for long-term deflections

R9.5.2.4 For continuous members, the SBC 304 procedure suggests a simple averaging of I_e values for the positive and negative moment sections. The use of the midspan section properties for continuous prismatic members is considered satisfactory in approximate calculations primarily because the midspan rigidity (including the effect of cracking) has the dominant effect on deflections, as shown in References 9.3, 9.9 and 9.10.

R9.5.2.5 Shrinkage and creep due to sustained loads cause additional long-term deflections over and above those which occur when loads are first placed on the structure. Such deflections are influenced by temperature, humidity, curing conditions, age at time of loading, quantity of compression reinforcement, and magnitude of the sustained load. The expression given in this section is considered satisfactory for use with the SBC 304 procedures for the calculation of immediate deflections, and with the limits given in Table 9.5(b). The deflection computed in accordance with this section is the additional long-term deflection due to the dead load and that portion of the live load that will be sustained for a sufficient period to cause significant time-dependent deflections.

Eq. (9-11) was developed in Reference 9.19. In Eq. (9-11) the multiplier on ζ accounts for the effect of compression reinforcement in reducing long-term deflections. $\zeta = 2.0$ represents a nominal time-dependent factor for 5 years duration of loading. The curve in Fig. R9.5.2.5 may be used to estimate values of ζ for loading periods less than five years.

If it is desired to consider creep and shrinkage separately, approximate equations provided in References 9.5, 9.6, 9.11, and 9.12 may be used.

R9.5.2.6 It should be noted that the limitations given in this table relate only to supported or attached nonstructural elements. For those structures in which structural members are likely to be affected by deflection or deformation of members to which they are attached in such a manner as to affect adversely the strength of the structure, these deflections and the resulting forces should be considered explicitly in the analysis and design of the structures as required by 9.5.1. (See Reference 9.8.)

Where long-term deflections are computed, the portion of the deflection before attachment of the nonstructural elements may be deducted. In making this correction use may be made of the curve in Fig. R9.5.2.5 for members of usual sizes and shapes.

R9.5.3 Two-way construction (nonprestressed)

R9.5.3.2 The minimum thicknesses in Table 9.5(c) are those that have been developed through the years. Slabs conforming to those limits have not resulted in systematic problems related to stiffness for short- and long-term loads. These limits apply to only the domain of previous experience in loads, environment, materials, boundary conditions, and spans.

R9.5.3.3 For panels having a ratio of long to short span greater than 2, the use of Eq. (9-12) and (9-13), which express the minimum thickness as a fraction of the long span, may give unreasonable results. For such panels, the rules applying to one-way construction in 9.5.2 should be used.

R9.5.3.4 The calculation of deflections for slabs is complicated even if linear elastic behavior can be assumed. For immediate deflections, the values of E_c and I_g specified in 9.5.2.3 may be used.[9.16] However, other procedures and other values of the stiffness EI may be used if they result in predictions of deflection in reasonable agreement with the results of comprehensive tests.

Since available data on long-term deflections of slabs are too limited to justify more elaborate procedures, the additional long-term deflection for two-way construction is required to be computed using the multipliers given in 9.5.2.5.

R9.5.4 **Prestressed concrete construction.** The SBC 304 requires deflections for all prestressed concrete flexural members to be computed and compared with the allowable values in Table 9.5(b).

R9.5.4.1 Immediate deflections of Class U prestressed concrete members may be calculated by the usual methods or formulas for elastic deflections using the moment of inertia of the gross (uncracked) concrete section and the modulus of elasticity for concrete specified in 8.5.1.

TABLE 9.5(b) - MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\ell/180$*
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\ell/360$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)**	$\ell/480$‡
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\ell/240$§

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

** Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.3, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

‡ Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

§ Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

TABLE 9.5(c)-MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS

Yield strength, f_y, MPa*	Without drop panels†			With drop panels†		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams‡		Without edge beams	With edge beams‡	
300	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{40}$	$\frac{\ell_n}{40}$
420	$\frac{\ell_n}{30}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$
520	$\frac{\ell_n}{28}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{34}$	$\frac{\ell_n}{34}$

* For values of reinforcement yield strength between the values given in the table, minimum thickness shall be determined by linear interpolation.

† Drop panel is defined in 13.3.7.1 and 13.3.7.2.

‡ Slabs with beams between columns along exterior edges. The value of α for the edge beam shall not be less than 0.8.

R9.5.4.2 Class C and Class T prestressed flexural members are defined in 18.3.3. Reference 9.13 gives information on deflection calculations using a bilinear moment-deflection relationship and using an effective moment of inertia. Reference 9.14 gives additional information on deflection of cracked prestressed concrete members.

Reference 9.15 shows that the I_e method can be used to compute deflections of Class T prestressed members loaded above the cracking load. For this case, the cracking moment should take into account the effect of prestress. A method for predicting the effect of nonprestressed tension steel in reducing creep camber is also given in Reference 9.15, with approximate forms given in References 9.8 and 9.16.

R9.5.4.3 Calculation of long-term deflections of prestressed concrete flexural members is complicated. The calculations should consider not only the increased deflections due to flexural stresses, but also the additional long-term deflections resulting from time-dependent shortening of the flexural member.

Prestressed concrete members shorten more with time than similar nonprestressed members due to the precompression in the slab or beam which causes axial creep. This creep together with concrete shrinkage results in significant shortening of the flexural members that continues for several years after construction and should be considered in design. The shortening tends to reduce the tension in the prestressing steel, reducing the precompression in the member and thereby causing increased long-term deflections.

Another factor that can influence long-term deflections of prestressed flexural members is adjacent concrete or masonry that is nonprestressed in the direction of the prestressed member. This can be a slab nonprestressed in the beam direction adjacent to a prestressed beam or a nonprestressed slab system. As the prestressed member tends to shrink and creep more than the adjacent nonprestressed concrete, the structure will tend to reach a compatibility of the shortening effects. This results in a reduction of the precompression in the prestressed member as the adjacent concrete absorbs the compression. This reduction in precompression of the prestressed member can occur over a period of years and will result in additional long-term deflections and in increase tensile stresses in the prestressed member.

Any suitable method for calculating long-term deflections of prestressed members may be used, provided all effects are considered. Guidance may be found in References 9.8, 9.17, 9.18, and 9.19.

R9.5.5 **Composite construction.** A composite construction is defined as a type of construction using members produced by combining different (e.g., concrete and structural steel), members produced by combining cast-in-place and precast concrete, or cast-in-place concrete elements constructed in separate placements but so interconnected that the combined components act together as a single member and respond to loads as a unit.

Since few tests have been made to study the immediate and long-term deflections of composite members, the rules given in 9.5.5.1 and 9.5.2 are based on experience.

If any portion of a composite member is prestressed or if the member is prestressed after the components have been cast, the provisions of 9.5.4 apply and deflections are to be calculated. For nonprestressed composite members, deflections need to be calculated and compared with the limiting values in Table 9.5(b) only when the thickness of the member or the precast part of the member is less than the minimum thickness given in Table 9.5(a). In unshored construction the thickness of concern depends on whether the deflection before or after the attainment of effective composite action is being considered. (In Chapter 17, it is stated that distinction need not be made between shored and unshored members. This refers to strength calculations, not to deflections.)

CHAPTER 10
FLEXURAL AND AXIAL LOADS

SECTION R10.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

d_t = distance from extreme compression fiber to extreme tension steel, mm

The definition of net tensile strain in 2.1 excludes strains due to effective prestress, creep, shrinkage, and temperature.

SECTION R10.2
DESIGN ASSUMPTIONS

R10.2.1 The strength of a member computed by the strength design method of the SBC 304 requires that two basic conditions be satisfied: (1) static equilibrium, and (2) compatibility of strains. Equilibrium between the compressive and tensile forces acting on the cross section at nominal strength should be satisfied. Compatibility between the stress and strain for the concrete and the reinforcement at nominal strength conditions should also be established within the design assumptions allowed by 10.2.

R10.2.2 Many tests have confirmed that the distribution of strain is essentially linear across a reinforced concrete cross section, even near ultimate strength.
Both the strains in reinforcement and in concrete are assumed to be directly proportional to the distance from the neutral axis. This assumption is of primary importance in design for determining the strain and corresponding stress in the reinforcement.

R10.2.3 The maximum concrete compressive strain at crushing of the concrete has been observed in tests of various kinds to vary from 0.003 to higher than 0.008 under special conditions. However, the strain at which ultimate moments are developed is usually about 0.003 to 0.004 for members of normal proportions and materials.

R10.2.4 For deformed reinforcement, it is reasonably accurate to assume that the stress in reinforcement is proportional to strain below the yield strength f_y. The increase in strength due to the effect of strain hardening of the reinforcement is neglected for strength computations. In strength computations, the force developed in tensile or compressive reinforcement is computed as,
when $\varepsilon_s < \varepsilon_y$ (yield strain)

$$A_s f_s = A_s E_s \varepsilon_s$$

when $\varepsilon_s \geq \varepsilon_y$

$$A_s f_s = A_s f_y$$

where ε_s is the value from the strain diagram at the location of the reinforcement. For design, the modulus of elasticity of steel reinforcement E_s may be taken as 200,000 MPa (see 8.5.2).

R10.2.5 The tensile strength of concrete in flexure (modulus of rupture) is a more variable property than the compressive strength and is about 10 to 15% of the compressive strength. Tensile strength of concrete in flexure is neglected in strength design. For members with normal percentages of reinforcement, this assumption is in good agreement with tests. For very small percentages of reinforcement, neglect of the tensile strength at ultimate is usually correct. The strength of concrete in tension, however, is important in cracking and deflection considerations at service loads.

R10.2.6 This assumption recognizes the inelastic stress distribution of concrete at high stress. As maximum stress is approached, the stress-strain relationship for concrete is not a straight line but some form of a curve (stress is not proportional to strain). The general shape of a stress-strain curve is primarily a function of concrete strength and consists of a rising curve from zero to a maximum at a compressive strain between 0.0015 and 0.002 followed by a descending curve to an ultimate strain (crushing of the concrete) from 0.003 to higher than 0.008. As discussed under R10.2.3. The SBC 304 sets the maximum usable strain at 0.003 for design.

The actual distribution of concrete compressive stress is complex and usually not known explicitly. Research has shown that the important properties of the concrete stress distribution can be approximated closely using any one of several different assumptions as to the form of stress distribution. The SBC 304 permits any particular stress distribution to be assumed in design if shown to result in predictions of ultimate strength in reasonable agreement with the results of comprehensive tests. Many stress distributions have been proposed. The three most common are the parabola, trapezoid, and rectangle.

R10.2.7 For design, the SBC 304 allows the use of an equivalent rectangular compressive stress distribution (stress block) to replace the more exact concrete stress distribution. In the equivalent rectangular stress block, an average stress of $0.85 f_c^{'}$ is used with a rectangle of depth $a = \beta_1 c$. The β_1 of 0.85 for concrete with $f_c^{'} \leq$ 30 MPa and 0.05 less for each 7 MPa of $f_c^{'}$ in excess of 30 was determined experimentally.

Research data from tests with high strength concretes[10.1,10.2] supported the equivalent rectangular stress block for concrete strengths exceeding 55 MPa, with a β_1 equal to 0.65.

The equivalent rectangular stress distribution does not represent the actual stress distribution in the compression zone at ultimate, but does provide essentially the same results as those obtained in tests.[10. 3]

SECTION R10.3
GENERAL PRINCIPLES AND REQUIREMENTS

R10.3.1 For members subject to flexure or combined flexure and axial load, derivations of design strength equations are given in Reference 10.3 for rectangular cross section as well as for other cross sections.

R10.3.2 A balanced strain condition exists at a cross section when the maximum strain at the extreme compression fiber just reaches 0.003 simultaneously with the first yield strain f_y / E_s in the tension reinforcement. The reinforcement ratio ρ_b which produces balanced conditions under flexure, depends on the shape of the cross section and the location of the reinforcement.

R10.3.3 The nominal flexural strength of a member is reached when the strain in the extreme compression fiber reaches the assumed strain limit 0.003. The net tensile strain ε_t is the tensile strain in the extreme tension steel at nominal strength, exclusive of strains due to prestress, creep, shrinkage, and temperature. The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength, shown in Fig. R10.3.3, using similar triangles.

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, whereas compression members are usually compression-controlled. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections. Section 9.3.2 specifies the appropriate strength reduction factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

Before the development of these provisions, the limiting tensile strain for flexural members was not stated, but was implicit in the maximum tension reinforcement ratio that was given as a fraction of ρ_b, which was dependent on the yield strength of the reinforcement. The net tensile strain limit of 0.005 for tension-controlled sections was chosen to be a single value that applies to all types of steel (prestressed and nonprestressed) permitted by this SBC 304.

Unless unusual amounts of ductility are required, the 0.005 limit will provide ductile behavior for most designs. One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames. Section 8.4 permits redistribution of negative moments. Since moment redistribution is dependent on adequate ductility in hinge regions, moment redistribution is limited to sections that have a net tensile strain of at least 0.0075.

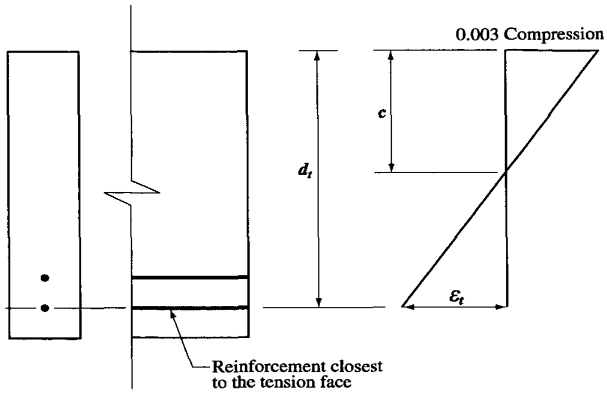


Fig. R10.3.3 - Strain distribution and net tensile strain.

For beams with compression reinforcement, or T-beams, the effects of compression reinforcement and flanges are automatically accounted for in the computation of net tensile strain ε_t.

R10.3.5 The effect of this limitation is to restrict the reinforcement ratio in nonprestressed beams. The reinforcement limit of $0.75\rho_b$ results in a net tensile strain at nominal strength of 0.00376. The proposed limit of 0.005 is more conservative. This limitation does not apply to prestressed members.

R10.3.6 and R10.3.7 The specified minimum eccentricities are intended to serve as a means of reducing the axial load design strength of a section in pure compression to account for accidental eccentricities not considered in the analysis that may exist in a compression member, and to recognize that concrete strength may be less than $f_c^{'}$ under sustained high loads. The primary purpose of the minimum eccentricity requirement is to limit the maximum design axial load strength of a compression member. This is accomplished directly in 10.3.6 by limiting the design axial load strength of a section in pure compression to 85 or 80 percent of the nominal strength. These percentage values approximate the axial load strengths at e/h ratios of 0.05 and 0.10, for the spirally reinforced and tied members, respectively. The same axial load limitation applies to both cast-in-place and precast compression members. Design aids and computer programs based on the minimum eccentricity requirement are equally applicable.

For prestressed members, the design axial load strength in pure compression is computed by the strength design methods of Chapter 10, including the effect of the prestressing force.

Compression member end moments should be considered in the design of adjacent flexural members. In nonsway frames, the effects of magnifying the end moments need not be considered in the design of the adjacent beams. In sway frames, the magnified end moments should be considered in designing the flexural members, as required in 10.13.7.

Corner and other columns exposed to known moments about each axis simultaneously should be designed for biaxial bending and axial load. Satisfactory methods are available in References 10.4 and 10.5. The reciprocal load method (Reference 10.6) and the load contour method (Reference 10.7) are the methods used in those two hand-books. Research [10.8, 10.9] indicates that using the equivalent rectangular stress block provisions of 10.2.7 produces satisfactory strength estimates for doubly symmetric sections. A simple and somewhat conservative estimate of nominal strength P_{ni} can be obtained from the reciprocal load relationship.[10.6]

$$\frac{1}{P_{ni}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_o}$$

where:

P_{ni} = nominal axial load strength at given eccentricity along both axes

P_o = nominal axial load strength at zero eccentricity

P_{nx} = nominal axial load strength at given eccentricity along x-axis

P_{ny} = nominal axial load strength at given eccentricity along y-axis

This relationship is most suitable when values P_{nx} and P_{ny} are greater than the balanced axial force P_b for the particular axis.

SECTION R10.4
DISTANCE BETWEEN LATERAL SUPPORTS OF FLEXURAL MEMBERS

Tests[10.10,10.11] have shown that laterally unbraced reinforced concrete beams of any reasonable dimensions, even when very deep and narrow, will not fail prematurely by lateral buckling provided the beams are loaded without lateral eccentricity that causes torsion.

Laterally unbraced beams are frequently loaded off center (lateral eccentricity) or with slight inclination. Stresses and deformations set up by such loading become detrimental for narrow, deep beams, the more so as the unsupported length increases. Lateral supports spaced closer than $50b$ may be required by loading conditions.

SECTION R10.5
MINIMUM REINFORCEMENT OF FLEXURAL MEMBERS

The provision for a minimum amount of reinforcement applies to flexural members, which for architectural or other reasons, are larger in cross section than required for strength. With a very small amount of tensile reinforcement, the computed moment strength as a reinforced concrete section using cracked section analysis becomes less than that of the corresponding unreinforced concrete section computed from its modulus of rupture. Failure in such a case can be sudden.
To prevent such a failure, a minimum amount of tensile reinforcement is required by 10.5.1 in both positive and negative moment regions.

R10.5.2 When the flange of a section is in tension, the amount of tensile reinforcement needed to make the strength of the reinforced section equal that of the unreinforced section is about twice that for a rectangular section or that of a flanged section with the flange in compression. A higher amount of minimum tensile reinforcement is particularly necessary in cantilevers and other statically determinate members where there is no possibility for redistribution of moments.

R10.5.3 The minimum reinforcement required for slabs should be equal to the same amount as that required by 7.12 for shrinkage and temperature reinforcement.

Soil-supported slabs such as slabs on grade are not considered to be structural slabs in the context of this section, unless they transmit vertical loads from other parts of the structure to the soil. Reinforcement, if any, in soil-supported slabs should be proportioned with due consideration of all design forces. Mat foundations and other slabs that help support the structure vertically should meet the requirements of this section.

In reevaluating the overall treatment of 10.5, the maximum spacing for reinforcement in structural slabs (including footings) was reduced from the $4h$ for temperature and shrinkage reinforcement to the compromise value of $3h$, which is somewhat larger than the $2h$ limit of 13.3.2 for two-way slab systems.

SECTION R10.6
DISTRIBUTION OF FLEXURAL REINFORCEMENT IN BEAMS AND ONE-WAY SLABS

R10.6.1 Many structures designed by working stress methods and with low steel stress served their intended functions with very limited flexural cracking. When high strength reinforcing steels are used at high service load stresses, however, visible cracks should be expected, and steps should be taken in detailing of the reinforcement to control cracking. For protection of reinforcement against corrosion, and for aesthetic reasons, many fine hairline cracks are preferable to a few wide cracks.

Control of cracking is particularly important when reinforcement with a yield strength in excess of 300 MPa is used. Current good detailing practices will usually lead to adequate crack control even when reinforcement of 420 MPa yield is used.

Extensive laboratory work[10.12-10.14] involving deformed bars has confirmed that crack width at service loads is proportional to steel stress. The significant variables reflecting steel detailing were found to be thickness of concrete cover and the area of concrete in the zone of maximum tension surrounding each individual reinforcing bar.

Crack width is inherently subject to wide scatter even in careful laboratory work and is influenced by shrinkage and other time-dependent effects. The better crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

R10.6.3 Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

R10.6.4 To control flexural cracks, the maximum bar spacing is specified in Reference 10.15, 10.16, and 10.17. For the usual case of beams with Grade 420 reinforcement and 50 mm clear cover to the main reinforcement, with f_s = 250 MPa, the maximum bar spacing is 300 mm

Crack widths in structures are highly variable. In previous SBC 304s, provisions were given for distribution of reinforcement that were based on empirical equations using a calculated maximum crack width of 0.4 mm. The current provisions for spacing are intended to limit surface cracks to a width that is generally acceptable in practice but may vary widely in a given structure.

The role of cracks in the corrosion of reinforcement is controversial. Research (Reference 10.18 and 10.19) shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels. For this reason, no distinction between interior and exterior exposure is made.

R10.6.5 Although a number of studies have been conducted, clear experimental evidence is not available regarding the crack width beyond which a corrosion danger exists. Exposure tests indicate that concrete quality, adequate consolidation, and ample concrete cover may be of greater importance for corrosion protection than crack width at the concrete surface.

R10.6.6 In major T-beams, distribution of the negative reinforcement for control of cracking should take into account two considerations: (1) wide spacing of the reinforcement across the full effective width of flange may cause some wide cracks to form in the slab near the web and, (2) close spacing near the web leaves the outer regions of the flange unprotected. The one-tenth limitation is to guard against too wide a spacing, with some additional reinforcement required to protect the outer portions of the flange.

R10.6.7 For relatively deep flexural members, some reinforcement should be placed near the vertical faces of the tension zone to control cracking in the web.[10.16] (See Fig. R10.6.7.) Without such auxiliary steel, the width of the cracks in the web may exceed the crack widths at the level of the flexural tension reinforcement.
Where the provisions for deep beams, walls, or precast panels require more steel, those provisions (along with their spacing requirements) will govern.

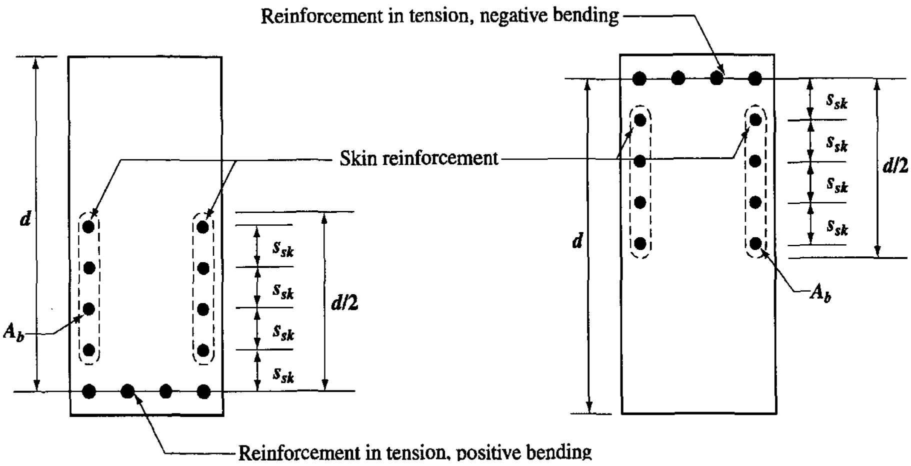


Fig. R10.6.7 - Skin reinforcement of beams and joists with d > 900mm

SECTION R10.7
DEEP BEAMS

Are based on D-region behavior (see Appendix A in SBC 304).
SBC 304 does not contain detailed requirements for designing deep beams for flexure except that nonlinearity of strain distribution and lateral buckling is to be considered. Suggestions for the design of deep beams for flexure are given in References 10.20, 10.21, and 10.22.

SECTION R10.8
DESIGN DIMENSIONS FOR COMPRESSION MEMBERS

The engineer should recognize the need for careful workmanship, as well as the increased significance of shrinkage stresses with small sections.

R10.8.2-
R10.8.4 For column design,[10.23] the SBC 304 provisions for quantity of reinforcement, both vertical and spiral, are based on the gross column area and core area, and the design strength of the column is based on the gross area of the column section. In some cases, however, the gross area is larger than necessary to carry the factored

load. The basis of 10.8.2, 10.8.3, and 10.8.4 is that it is satisfactory to design a column of sufficient size to carry the factored load and then simply add concrete around the designed section without increasing the reinforcement to meet the minimum percentages required by 10.9.1. The additional concrete should not be considered as carrying load; however, the effects of the additional concrete on member stiffness should be included in the structural analysis. The effects of the additional concrete also should be considered in design of the other parts of the structure that interact with the oversize member.

SECTION R10.9
LIMITS FOR REINFORCEMENT OF COMPRESSION MEMBERS

R10.9.1 This section prescribes the limits on the amount of longitudinal reinforcement for noncomposite compression members. If the use of high reinforcement ratios would involve practical difficulties in the placing of concrete, a lower percentage and hence a larger column, or higher strength concrete or reinforcement (see R9.4) should be considered. The percentage of reinforcement in columns should usually not exceed 4 percent if the column bars are required to be lap spliced.

Minimum reinforcement - Since the design methods for columns incorporate separate terms for the load carried by concrete and by reinforcement, it is necessary to specify some minimum amount of reinforcement to ensure that only reinforced concrete columns are designed by these procedures. Reinforcement is necessary to provide resistance to bending, which may exist whether or not computations show that bending exists, and to reduce the effects of creep and shrinkage of the concrete under sustained compressive stresses. Tests have shown that creep and shrinkage tend to transfer load from the concrete to the reinforcement, with a consequent increase in stress in the reinforcement, and that this increase is greater as the ratio of reinforcement decreases. Unless a lower limit is placed on this ratio, the stress in the reinforcement may increase to the yield level under sustained service loads. This phenomenon was emphasized in Reference 10.24 and minimum reinforcement ratios of 0.01 and 0.005 were recommended for spiral and tied columns, respectively. However, in this SBC 304, the minimum ratio is 0.01 for both types of laterally reinforced columns.

Maximum reinforcement - Extensive tests of column investigation (Reference 10.24) included reinforcement ratios no greater than 0.06. Although other tests with as much as 17 percent reinforcement in the form of bars produced results similar to those obtained previously, it is necessary to note that the loads in these tests were applied through bearing plates on the ends of the columns and the problem of transferring a proportional amount of the load to the bars was thus minimized or avoided. Maximum ratios of 0.08 and 0.03 were recommended by Reference 10.24 for spiral and tied columns, respectively. In this SBC 304, it is required that bending be considered in the design of all columns, and the maximum ratio of 0.08 is applied to both types of columns. This limit can be considered a practical maximum for reinforcement in terms of economy and requirements for placing.

When the number of bars in a circular arrangement is less than eight, the orientation of the bars will affect the moment strength of eccentrically loaded columns and should be considered in design.

R10.9.2 For compression members, a minimum of four longitudinal bars are required when bars are enclosed by rectangular or circular ties. For other shapes, one bar should be provided at each apex or corner and proper lateral reinforcement provided. For example, tied triangular columns require three longitudinal bars, one at each apex of the triangular ties. For bars enclosed by spirals, six bars are required.

R10.9.3 The effect of spiral reinforcement in increasing the load-carrying strength of the concrete within the core is not realized until the column has been subjected to a load and deformation sufficient to cause the concrete shell outside the core to spa11 off. The amount of spiral reinforcement required by Eq. (10-5) is intended to provide additional load-carrying strength for concentrically loaded columns equal to or slightly greater than the strength lost when the shell spalls off. This principle was recommended by Reference 10.24 and is adopted in this SBC 304. The derivation of Eq. (10-5) is given in Reference 10.24. Tests and experience show that columns containing the amount of spiral reinforcement required by this section exhibit considerable toughness and ductility.

SECTION R10.10
SLENDERNESS EFFECTS IN COMPRESSION MEMBERS

Provisions for slenderness effects in compression members and frames are discussed in Reference 10.25 to better recognize the use of second-order analyses and to improve the arrangement of the provisions dealing with sway (unbraced) and nonsway (braced) frames. The use of a refined nonlinear second-order analysis is permitted in 10.10.1. Sections 10.11, 10.12, and 10.13 present an approximate design method based on the moment magnifier method. For sway frames, the magnified sway moment $\delta_s M_s$ may be calculated using a second-order elastic analysis, by an approximation to such an analysis, or by the traditional sway moment magnifier.

R10.10.1 Two limits are placed on the use of the refined second-order analysis. First, the structure that is analyzed should have members similar to those in the final structure. If the members in the final structure have cross-sectional dimensions more than 10 percent different from those assumed in the analysis, new member properties should be computed and the analysis repeated. Second, the refined second-order analysis procedure should have been shown to predict ultimate loads within 15 percent of those reported in tests of indeterminate reinforced concrete structures. At the very least, the comparison should include tests of columns in planar nonsway frames, sway frames, and frames with varying column stiffnesses. To allow for variability in the actual member properties and in the analysis, the member properties used in analysis should be multiplied by a stiffness reduction factor ϕ_k less than one. For consistency with the second-order analysis in 10.13.4.1, the stiffness reduction factor ϕ_k can be taken as 0.80. The concept of a stiffness reduction factor ϕ_k is discussed in R10.12.3.

R10.10.2 As an alternate to the refined second-order analysis of 10.10.1, design may be based on elastic analyses and the moment magnifier approach.[10.26,10.27] For sway frames the magnified sway moments may be calculated using a second-order elastic analysis based on realistic stiffness values. See R10.13.4.1.

SECTION R10.11
MAGNIFIED MOMENTS - GENERAL

This section describes an approximate design procedure that uses the moment magnifier concept to account for slenderness effects. Moments computed using an ordinary first-order frame analysis are multiplied by a moment magnifier that is a function of the factored axial load P_u and the critical buckling load P_c for the column. Nonsway and sway frames are treated separately in 10.12 and 10.13. Provisions applicable to both nonsway and sway columns are given in 10.11. A first-order frame analysis is an elastic analysis that does not include the internal force effects resulting from deflections.

R10.11.1 The stiffnesses EI used in an elastic analysis used for strength design should represent the stiffnesses of the members immediately prior to failure. This is particularly true for a second-order analysis that should predict the lateral deflections at loads approaching ultimate. The EI values should not be based totally on the moment-curvature relationship for the most highly loaded section along the length of each member. Instead, they should correspond to the moment-end rotation relationship for a complete member.

The alternative values of E_c, I_g, and A_g given in 10.11.1 have been chosen from the results of frame tests and analyses and include an allowance for the variability of the computed deflections. The modulus of elasticity E_c is based on the specified concrete strength while the sway deflections are a function of the average concrete strength, which is higher. The moments of inertia were taken as 0.875 times those in Reference 10.28. These two effects result in an overestimation of the second-order deflections in the order of 20 to 25 percent, corresponding to an implicit stiffness reduction factor ϕ_k of 0.80 to 0.85 on the stability calculation. The concept of a stiffness reduction factor ϕ_k is discussed in R10.12.3

The moment of inertia of T-beams should be based on the effective flange width defined in 8.10. It is generally sufficiently accurate to take I_g of a T-beam as two times the I_g for the web, $2\left(b_w h^3/12\right)$.

If the factored moments and shears from an analysis based on the moment of inertia of a wall taken equal to $0.70 I_g$ indicate that the wall will crack in flexure, based on the modulus of rupture, the analysis should be repeated with $I = 0.35 I_g$ in those stories where cracking is predicted at factored loads.

The alternative values of the moments of inertia given in 10.11.1 were derived for nonprestressed members. For prestressed members, the moments of inertia may differ from the values in 10.11.1 depending on the amount, location, and type of the reinforcement and the degree of cracking prior to ultimate. The stiffness values for prestressed concrete members should include an allowance for the variability of the stiffnesses.

Sections 10.11 through 10.13 provide requirements for strength and assume frame analyses will be carried out using factored loads. Analyses of deflections, vibrations, and building periods are needed at various service (unfactored) load levels[10.29,10.30] to determine the serviceability of the structure and to estimate the wind forces in wind tunnel laboratories. The seismic base shear is also based on the service load periods of vibration. The magnified service loads and deflections by a second-order analysis should also be computed using service loads. The moments of inertia of the structural members in the service load analyses should, therefore, be representative of the degree of cracking at the various service load levels investigated. Unless a more accurate estimate of the degree of cracking at design service load level is available, it is satisfactory to use *1/0.70 = 1.43* times the moments of inertia given in 10.11.1 for service load analyses.

Item (d) in 10.11.1 refers to the unusual case of sustained lateral loads. Such a case might exist, for example, if there were permanent lateral loads resulting from unequal earth pressures on two sides of a building.

R10.11.4 The moment magnifier design method requires the designer to distinguish between nonsway frames, which are designed according to 10.12, and sway frames, which are designed according to 10.13. Frequently this can be done by inspection by comparing the total lateral stiffness of the columns in a story to that of the bracing elements. A compression member may be assumed nonsway by inspection if it is located in a story in which the bracing elements (shearwalls, shear trusses, or other types of lateral bracing) have such substantial lateral stiffness to resist the lateral deflections of the story that any resulting lateral deflection is not large enough to affect the column strength substantially. If not readily apparent by inspection, 10.11.4.1 and 10.11.4.2 give two possible ways of doing this. In 10.11.4.1, a story in a frame is said to be nonsway if the increase in the lateral load moments resulting from $P\Delta$ effects does not exceed 5 percent of the first-order moments.[10.28] Section 10.11.4.2 gives an alternative method of determining this based on the stability index for a story Q. In computing Q, $\sum P_u$ should correspond to the lateral loading case for which $\sum P_u$ is greatest. A frame may contain both nonsway and sway stories. This test would not be suitable if V_u is zero.

If the lateral load deflections of the frame have been computed using service loads and the service load moments of inertia given in 10.11.1, it is permissible to compute Q in Eq. (10-6) using 1.2 times the sum of the service gravity loads, the service load story shear, and 1.43 times the first-order service load story deflections.

R10.11.5 An upper limit is imposed on the slenderness ratio of columns designed by the moment magnifier method of 10.11 to 10.13. No similar limit is imposed if design is carried out according to 10.10.1. The limit of $k\ell_u / r = 100$ represents the upper range of actual tests of slender compression members in frames.

R10.11.6 When biaxial bending occurs in a compression member, the computed moments about each principal axes should be magnified. The magnification factors δ are computed considering the buckling load P_c about each axis separately based on the appropriate effective length kl_u and the stiffness EI. If the buckling capacities are different about the two axes, different magnification factors will result.

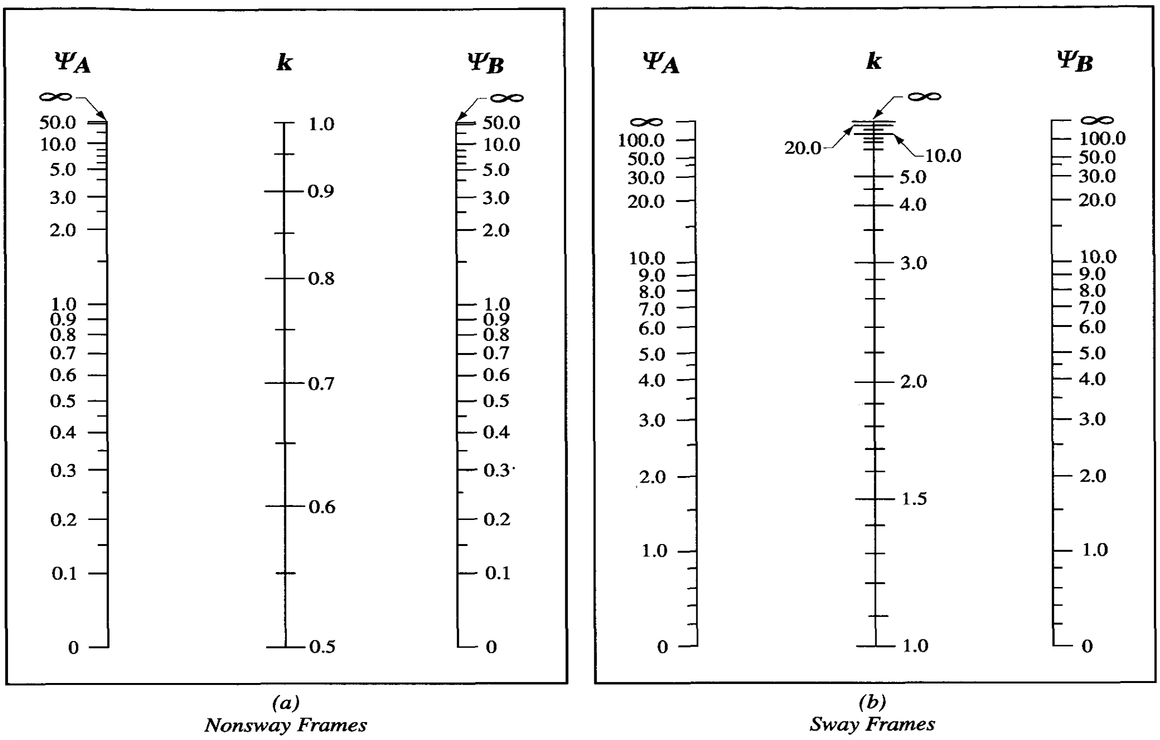
SECTION R10.12
MAGNIFIED MOMENTS – NONSWAY FRAMES

R10.12.1 The moment magnifier equations were derived for hinged end columns and should be modified to account for the effect of end restraints. This is done by using an effective length $k\ell_u$ in the computation of P_c.

The primary design aid to estimate the effective length factor k is the Jackson and Moreland Alignment Charts (Fig. R10.12.1), which allow a graphical determination of k for a column of constant cross section in a multibay frame.[10.31,10.32]

The effective length is a function of the relative stiffness at each end of the compression member. Studies have indicated that the effects of varying beam and column reinforcement percentages and beam cracking should be considered in determining the relative end stiffnesses. In determining ψ for use in evaluating the effective length factor k, the rigidity of the flexural members may be calculated on the basis of $0.35I_g$ for flexural members to account for the effect of cracking and reinforcement on relative stiffness, and $0.70I_g$ for compression members.

The simplified equations (A-E), listed below for computing the effective length factors for nonsway and sway members, may be used. Eq. (A), (B), and (E) are taken from Reference 10.33 and 10.34. Eq. (C) and (D) for sway members were developed in Reference 10.32.



Ψ = ratio of $\Sigma(EI/\ell_c)$ of compression members to $\Sigma(EI/\ell)$ of flexural members in a plane at one end of a compression member

ℓ = span length of flexural member measured center to center of joints

Fig. R10.12.1 - Effective length factors, k.

For compression members in a nonsway frame, an upper bound to the effective length factor may be taken as the smaller of the following two expressions:

$$k = 0.7 + 0.05(\psi_A + \psi_B) \leq 1.0 \qquad \text{(A)}$$

$$k = 0.85 + 0.05\psi_{\min} \leq 1.0 \qquad \text{(B)}$$

where ψ_A and ψ_B are the values of ψ at the two ends of the column and $\psi_{\min}$ is the smaller of the two values.

For compression members in a sway frame, restrained at both ends, the effective length factor may be taken as:

For $\Psi_m < 2$

$$k = \frac{20 - \Psi_m}{20}\sqrt{1 + \Psi_m} \qquad \text{(C)}$$

For $\Psi_m \geq 2$

$$k = 0.9\sqrt{1 + \Psi_m} \qquad \text{(D)}$$

where ψ_m is the average of the ψ-values at the two ends of the compression member.

For compression members in a sway frame, hinged at one end, the effective length factor may be taken as:

$$k = 2.0 + 0.3\psi \qquad \text{(E)}$$

Where ψ is the value at the restrained end.

The use of the charts in Fig. R10.12.1, or the equations in this section, may be considered as satisfying the requirements of the SBC 304 to justify k less than 1.0.

R10.12.2 Eq. (10-7) is derived from Eq. (10-9) assuming that a 5 percent increase in moments due to slenderness is acceptable.[10.26] The derivation did not include ϕ in the calculation of the moment magnifier. As a first approximation, k may be taken equal to 1.0 in Eq. (10-7).

R10.12.3 Studies reported in Reference 10.35 indicate that the stiffness reduction factor ϕ_k, and the cross-sectional strength reduction ϕ-factors do not have the same values. These studies suggest the stiffness reduction factor ϕ_k for an isolated column should be 0.75 for both tied and spiral columns. The 0.75 factors in Eq. (10-9) and (10-18) are stiffness reduction factors. This is done to avoid confusion between a stiffness reduction factor ϕ_k in Eq. (10-9) and (10-18), and the cross-sectional strength reduction ϕ-factors.

In defining the critical load, the main problem is the choice of a stiffness EI that reasonably approximates the variations in stiffness due to cracking, creep, and the nonlinearity of the the concrete stress-strain curve. Eq. (10-11) was derived for small eccentricity ratios and high levels of axial load where the slenderness effects are most pronounced.

Creep due to sustained load will increase the lateral deflections of a column and hence the moment magnification. This is approximated for design by reducing the stiffness EI used to compute P_c and hence δ_{ns} by dividing EI by $(1 + \beta_d)$. Both the concrete and steel terms in Eq. (10-11) are divided by $(1+\beta_d)$. This reflects the premature yielding of steel in columns subjected to sustained load.

Either Eq. (10-11) or (10-12) may be used to compute EI. Eq. (10-12) is a simplified approximation to Eq. (10-11). It is less accurate than Eq. (10-11).[10.36] Eq. (10-12) may be simplified further by assuming β_d = 0.6. When this is done Eq. (10-12) becomes

$$EI = 0.25 E_c I_g \qquad \text{(F)}$$

The term β_d is defined differently for nonsway and sway frames. See 10.0. For nonsway frames, β_d is the ratio of the maximum factored axial sustained load to the maximum factored axial load.

R10.12.3.1 The factor C_m is an equivalent moment correction factor. The derivation of the moment magnifier assumes that the maximum moment is at or near midheight of the column. If the maximum moment occurs at one end of the column, design should be based on an equivalent uniform moment C_m / M_2 that would lead to the same maximum moment when magnified. [10.26]

In the case of compression members that are subjected to transverse loading between supports, it is possible that the maximum moment will occur at a section away from the end of the member. If this occurs, the value of the largest calculated moment occurring anywhere along the member should be used for the value of M_2 in Eq. (10-8). In accordance with the last sentence of 10.12.3.1, C_m is to be taken as 1.0 for this case.

R10.12.3.2 In the SBC 304, slenderness is accounted for by magnifying the column end moments. If the factored column moments are very small or zero, the design of slender columns should be based on the minimum eccentricity given in this section. It is not intended that the minimum eccentricity be applied about both axes simultaneously.

The factored column end moments from the structural analysis are used in Eq. (10-13) in determining the ratio M_1 / M_2 for the column when the design should be based on minimum eccentricity. This eliminates what would otherwise be a discontinuity between columns with computed eccentricities less than the minimum eccentricity and columns with computed eccentricities equal to or greater than the minimum eccentricity.

SECTION R10.13
MAGNIFIED MOMENTS – SWAY FRAMES

The design procedure of sway frames for slenderness consists of three steps:

(1) The magnified sway moments $\delta_s M_s$ are computed. This should be done in one of three ways. First, a second-order elastic frame analysis may be used (10.13.4.1).

Second, an approximation to such analysis (10.13.4.2) may be used. The third option is to use the sway magnifier δ_s from previous editions of the SBC 304 (10.13.4.3);

(2) The magnified sway moments $\delta_s M_s$ are added to the unmagnified nonsway moment M_{ns} at each end of each column (10.13.3). The nonsway moments may be computed using a first-order elastic analysis;

(3) If the column is slender and heavily loaded, it is checked to see whether the moments at points between the ends of the column exceed those at the ends of the column. As specified in 10.13.5 this is done using the nonsway frame magnifier δ_{ns} with P_c computed assuming $k = 1.0$ or less.

R10.13.1 See R10.12.1.

R10.13.3 The analysis described in this section deals only with plane frames subjected to loads causing deflections in that plane. If torsional displacements are significant, a three-dimensional second-order analysis should be used.

R10.13.4 **Calculation of $\delta_s M_s$**

R10.13.4.1 A second-order analysis is a frame analysis that includes the internal force effects resulting from deflections. When a second-order elastic analysis is used to compute $\delta_s M_s$, the deflections should be representative of the stage immediately prior to the ultimate load. For this reason the reduced $E_c I_g$ values given in 10.11.1 should be used in the second-order analysis.

The term β_d is defined differently for nonsway and sway frames. See 10.0. Sway deflections due to short-term loads such as wind or earthquake are a function of the short-term stiffness of the columns following a period of sustained gravity load. For this case the definition of β_d in 10.0 gives β_d = 0. In the unusual case of a sway frame where the lateral loads are sustained, β_d will not be zero. This might occur if a building on a sloping site is subjected to earth pressure on one side but not on the other.

In a second-order analysis the axial loads in all columns that are not part of the lateral load resisting elements and depend on these elements for stability should be included.

The second-order analysis method is based on the values of E and I from 10.11.1. These lead to a 20 to 25 percent overestimation of the lateral deflections that corresponds to a stiffness reduction factor ϕ_k between 0.80 and 0.85 on the $P\Delta$ moments. No additional ϕ-factor is needed in the stability calculation. Once the moments are established, selection of the cross sections of the columns involves the strength reduction factors ϕ from 9.3.2.2.

R10.13.4.2 The iterative $P\Delta$ analysis for second-order moments can be represented by an infinite series. The solution of this series is given by Eq. (10-17).[10.28] Reference 10.37 shows that Eq. (10-17) closely predicts the second order moments in a sway frame until δ_s exceeds 1.5.

The $P\Delta$ moment diagrams for deflected columns are curved, with Δ related to the deflected shape of the columns. Eq. (10-17) and most commercially available second-order frame analyses have been derived assuming that the $P\Delta$ moments result from equal and opposite forces of $P\Delta / \ell_C$ applied at the bottom and top of the story. These forces give a straight line $P\Delta$ moment diagram. The curved $P\Delta$

moment diagrams lead to lateral displacements in the order of 15 percent larger than those from the straight line $P\Delta$ moment diagrams. This effect can be included in Eq. (10-17) by writing the denominator as $(1-1.15Q)$ rather than $(1-Q)$. The 1.15 factor has been left out of Eq. (10-17) to maintain consistency with available computer programs.

If deflections have been calculated using service loads, Q in Eq. (10-17) should be calculated in the manner explained in R10.11.4.

The Q factor analysis is based on deflections calculated using the values of E_c and I_g from 10.11.1, which include the equivalent of a stiffness reduction factor ϕ_k as explained in R10.13.4.1. As a result, no additional ϕ-factor is needed in the stability calculation. Once the moments are established using Eq. (10-17), selection of the cross sections of the columns involves the strength reduction factors ϕ from 9.3.2.2.

R10.13.4.3 To check the effects of story stability, δ_s is computed as an averaged value for the entire story based on use of $\sum P_u / \sum P_c$. This reflects the interaction of all sway resisting columns in the story in the $P\Delta$ effects since the lateral deflection of all columns in the story should be equal in the absence of torsional displacements about a vertical axis.

In addition, it is possible that a particularly slender individual column in a sway frame could have substantial midheight deflections even if adequately braced against lateral end deflections by other columns in the story. Such a column will have ℓ_u / r greater than the value given in Eq. (10-19) and should be checked using 10.13.5.

If the lateral load deflections involve a significant torsional displacement, the moment magnification in the columns farthest from the center of twist may be underestimated by the moment magnifier procedure. In such cases, a three-dimensional second-order analysis should be considered.

The 0.75 in the denominator of Eq. (10-18) is a stiffness reduction factor ϕ_k as explained in R10.12.3.

In the calculation of EI, β_d will normally be zero for a sway frame because the lateral loads are generally of short duration. (See R10.13.4.1).

R10.13.5 The unmagnified nonsway moments at the ends of the columns are added to the magnified sway moments at the same points. Generally, one of the resulting end moments is the maximum moment in the column. However, for slender columns with high axial loads the point of maximum moment may be between the ends of the column so that the end moments are no longer the maximum moments. If ℓ_u / r is less than the value given by Eq. (10-19) the maximum moment at any point along the height of such a column will be less than 1.05 times the maximum end moment. When ℓ_u / r exceeds the value given by Eq. (10-19), the maximum moment will occur at a point between the ends of the column and will exceed the maximum end moment by more than 5 percent.[10.25] In such a case the maximum moment is calculated by magnifying the end moments using Eq. (10-8).

R10.13.6 The possibility of sidesway instability under gravity loads alone should be investigated. When using second-order analyses to compile $\delta_s M_s$ (10.13.4.1), the frame should be analyzed twice for the case of factored gravity loads plus a lateral load applied to the frame. This load may be the lateral load used in design or it may be a single lateral load applied to the top of the frame. The first analysis should be a first-order analysis, the second analysis should be a second-order analysis. The deflection from the second-order analysis should not exceed 2.5 times the deflection from the first-order analysis. If one story is much more flexible than the others, the deflection ratio should be computed in that story. The lateral load should be large enough to give deflections of a magnitude that can be compared accurately. In unsymmetrical frames that deflect laterally under gravity loads alone, the lateral load should act in the direction for which it will increase the lateral deflections.

When using 10.13.4.2 to compute $\delta_s M_s$ the value of Q evaluated using factored gravity loads should not exceed 0.60. This is equivalent to δ_s = 2.5. The values of V_u and Δ_o used to compute Q can result from assuming any real or arbitrary set of lateral loads provided that V_u and Δ_o are both from the same loading. If Q as computed in 10.11.4.2 is 0.2 or less, the stability check in 10.13.6 is satisfied.

When $\delta_s M_s$ is computed using Eq. (10-18), an upper limit of 2.5 is placed on δ_s. For higher δ_s values, the frame will be very susceptible to variations in EI and foundation rotations. If δ_s exceeds 2.5 the frame should be stiffened to reduce $\delta_s . \sum P_u$ shall include the axial load in all columns and walls including columns that are not part of the lateral load resisting system. The value δ_s = 2.5 is a very high magnifier. It has been chosen to offset the conservatism inherent in the moment magnifier procedure.

For nonsway frames, β_d is the ratio of the maximum factored axial sustained load to the maximum factored axial load.

R10.13.7 The strength of a sway frame is governed by the stability of the columns and by the degree of end restraint provided by the beams in the frame. If plastic hinges form in the restraining beam, the structure approaches a failure mechanism and its axial load capacity is drastically reduced. Section 10.13.7 provides that the designer make certain that the restraining flexural members have the capacity to resist the magnified column moments.

SECTION R10.15
TRANSMISSION OF COLUMN LOADS THROUGH FLOOR SYSTEM

The requirements of this section are based on a paper on the effect of floor concrete strength on column strength.[10.38] The provisions mean that when the column concrete strength does not exceed the floor concrete strength by more than 40 percent, no special precautions need be taken. For higher column concrete strengths, methods in 10.15.1 or 10.15.2 should be used for corner or edge columns. Methods in 10.15.1, 10.15.2, or 10.15.3 should be used for interior columns with adequate restraint on all four sides.

R10.15.1 Application of the concrete placement procedure described in 10.15.1 requires the placing of two different concrete mixtures in the floor system. The lower strength mixture should be placed while the higher strength concrete is still plastic and should be adequately vibrated to ensure the concretes are well integrated. This requires careful coordination of the concrete deliveries and the possible use of retarders. In some cases, additional inspection services will be required when this procedure is used. It is important that the higher strength concrete in the floor in the region of the column be placed before the lower strength concrete in the remainder of the floor to prevent accidental placing of the low strength concrete in the column area. It is the designer's responsibility to indicate on the drawings where the high and low strength concretes are to be placed.

Since the concrete placement requirement should be carried out in the field, it is now expressed in a way that is directly evident to workers. The new requirement will also locate the interface between column and floor concrete farther out into the floor, away from regions of very high shear. The amount of column concrete to be placed within the floor is expressed as a simple 600-mm extension from face of the column, to be directly evident to workers.

R10.15.3 Research[10.39] has shown that heavily loaded slabs do not provide as much confinement as lightly loaded slabs when ratios of column concrete strength to slab concrete strength exceed about 2.5. Consequently, a limit is placed on the concrete strength ratio assumed in design.

SECTION R10.16
COMPOSITE COMPRESSION MEMBERS

R10.16.1 Composite columns are defined without reference to classifications of combination, composite or concrete-filled pipe column. Reference to other metals used for reinforcement has been omitted because they are seldom used in concrete construction.

R10.16.2 The same rules used for computing the load-moment interaction strength for reinforced concrete sections can be applied to composite sections. Interaction charts for concrete-filled tubing would have a form identical to those of Reference 10.40 and Reference 10.32 but with γ slightly greater than 1.0.

R10.16.3 and
R10.16.4 Direct bearing or direct connection for transfer of forces between steel and concrete can be developed through lugs, plates, or reinforcing bars welded to the structural shape or tubing before the concrete is cast. Flexural compressive stress need not be considered a part of direct compression load to be developed by bearing. A concrete encasement around a structural steel shape may stiffen the shape, but it would not necessarily increase its strength.

R10.16.5 Eq. (10-20) is given because the rules of 10.11.2 for estimating the radius of gyration are overly conservative for concrete filled tubing and are not applicable for members with enclosed structural shapes.

In reinforced concrete columns subject to sustained loads, creep transfers some of the load from the concrete to the steel, increasing the steel stresses. In the case of lightly reinforced columns, this load transfer may cause the compression steel to yield prematurely, resulting in a loss in the effective EI. Accordingly, both the concrete and steel terms in Eq. (10-11) are reduced to account for creep. For heavily reinforced columns or for composite columns in which the pipe or structural shape makes up a large percentage of the cross section, the load transfer due to creep is not significant. Accordingly, Eq. (10-21) will be used so that only the EI of the concrete is reduced for sustained load effects.

R10.16.6 **Structural steel encased concrete core**. Steel encased concrete sections should have a metal wall thickness large enough to attain longitudinal yield stress before buckling outward.

R10.16.7 **Spiral reinforcement around structural steel core.** Concrete that is laterally confined by a spiral has increased load-carrying strength, and the size of the spiral required can be regulated on the basis of the strength of the concrete outside the spiral the same reasoning that applies for columns reinforced only with longitudinal bars. The radial pressure provided by the spiral ensures interaction between concrete, reinforcing bars, and steel core such that longitudinal bars will both stiffen and strengthen the cross section.

R10.16.8 **Tie reinforcement around structural steel core.** Concrete that is laterally confined by tie bars is likely to be rather thin along at least one face of a steel core section. Therefore, complete interaction between the core, the concrete, and any longitudinal reinforcement should not be assumed. Concrete will probably separate from smooth faces of the steel core. To maintain the concrete around the structural steel core, it is reasonable to require more lateral ties than needed for ordinary reinforced concrete columns. Because of probable separation at high strains between the steel core and the concrete, longitudinal bars will be ineffective in stiffening cross sections even though they would be useful in sustaining compression forces. The yield strength of the steel core should be limited to that which exists at strains below those that can be sustained without spalling of the concrete. It has been assumed that axially compressed concrete will not spall at strains less than 0.0018. The yield strength of 0.0018 x 200,000, or 360 MPa, represents an upper limit of the useful maximum steel stress.

SECTION R10.17 BEARING STRENGTH

R10.17.1 This section deals with bearing strength of concrete supports. The permissible bearing stress of $0.85 f_c^{'}$ is based on tests reported in Reference 10.41. (See also 15.8).

When the supporting area is wider than the loaded area on all sides, the surrounding concrete confines the bearing area, resulting in an increase in bearing strength. No minimum depth is given for a supporting member. The minimum depth of support will be controlled by the shear requirements of 11.12.

When the top of the support is sloped or stepped, advantage may still be taken of the condition that the supporting member is larger than the loaded area, provided the supporting member does not slope at too great an angle. Fig. R10.17 illustrates the application of the frustum to find A_2. The frustum should not be confused with the path by which a load spreads out as it travels downward through the support.

Such a load path would have steeper sides. However, the frustum described has somewhat flat side slopes to ensure that there is concrete immediately surrounding the zone of high stress at the bearing. A_1 is the loaded area but not greater than the bearing plate or bearing cross-sectional area.

R10.17.2 Post-tensioning anchorages are usually laterally reinforced, in accordance with 18.13.

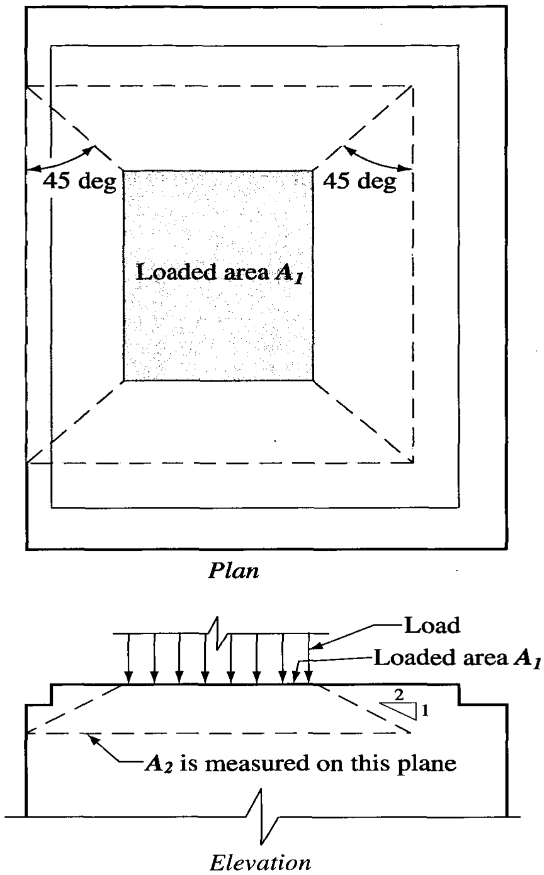


Fig. R10.17 - Application of frustum to find A_2 in stepped or sloped supports

CHAPTER 11
SHEAR AND TORSION

This chapter includes shear and torsion provisions for both nonprestressed and prestressed concrete members. The shear-friction concept (11.7) is particularly applicable to design of reinforcement details in precast structures. Special provisions are included for deep flexural members (11.8), brackets and corbels (11.9), and shear walls (11.10). Shear provisions for slabs and footings are given in 11.12.

SECTION R11.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

Tests[11.1] have indicated that the average shear stress over the full effective section also may be applicable for circular sections. Note the special definition of d for such sections.

Although the value of d may vary along the span of a prestressed beam, studies[11.2] showed that, for prestressed concrete members, d need not be taken less than $0.80h$. The beams considered had some straight tendons or reinforcing bars at the bottom of the section and had stirrups that enclosed that steel.

SECTION R11.1
SHEAR STRENGTH

The shear strength is based on an average shear stress on the full effective cross section $b_w d$. In a member without shear reinforcement, shear is assumed to be carried by the concrete web. In a member with shear reinforcement, a portion of the shear strength is assumed to be provided by the concrete and the remainder by the shear reinforcement.

The shear strength provided by concrete V_c is assumed to be the same for beams with and without shear reinforcement and is taken as the shear causing significant inclined cracking. These assumptions are discussed in References 11.1, 11.2, and 11.3.

Appendix A allows the use of strut-and-tie models in the shear design of disturbed regions. The traditional shear design procedures, which ignore D-regions, are acceptable in shear spans that include B-regions.

R11.1.1.1 Openings in the web of a member can reduce its shear strength. The effects of openings are discussed in Section 4.7 of Reference 11.1 and in References 11.4 and 11.5.

R11.1.1.2 In a member of variable depth, the internal shear at any section is increased or decreased by the vertical component of the inclined flexural stresses. Computation methods are outlined in various textbooks and in Reference 11.6

R11.1.2 Because of a lack of test data and practical experience with concretes having compressive strengths greater than 70 MPa, a maximum value of 25/3 MPa is imposed on $\sqrt{f_c'}$ for use in the calculation of shear strength of concrete beams, joists, and slabs. Exceptions to this limit were permitted in beams and joists when the transverse reinforcement satisfied an increased value for the minimum amount of web reinforcement. There are limited test data on the two-way shear strength of high-strength concrete slabs. Until more experience is obtained for two-way slabs built with concretes that have strengths greater than 70 MPa, it is prudent to limit $\sqrt{f_c'}$ to 25/3 MPa for the calculation of shear strength.

R11.1.2.1 Based on the test results in References 11.7, 11.8, 11.9, 11.10, and 11.11, an increase in the minimum amount of transverse reinforcement is required for high-strength concrete. These tests indicated a reduction in the reserve shear strength as f_c' increased in beams reinforced with the specified minimum amount of transverse reinforcement, which is equivalent to an effective shear stress of 0.34 MPa.

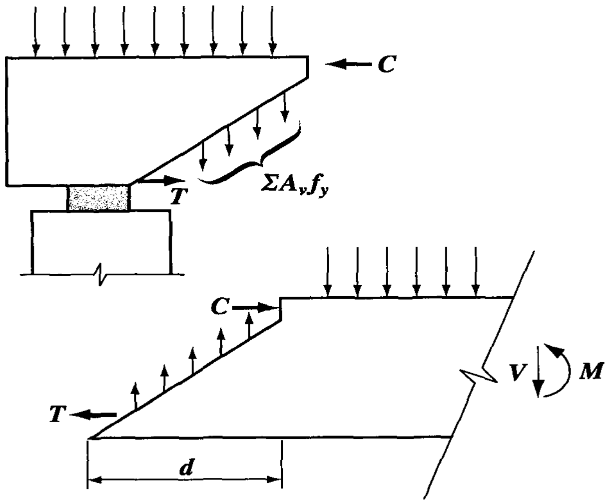


Fig. R11.1.3.1(a) - Free body diagrams of the end of a beam

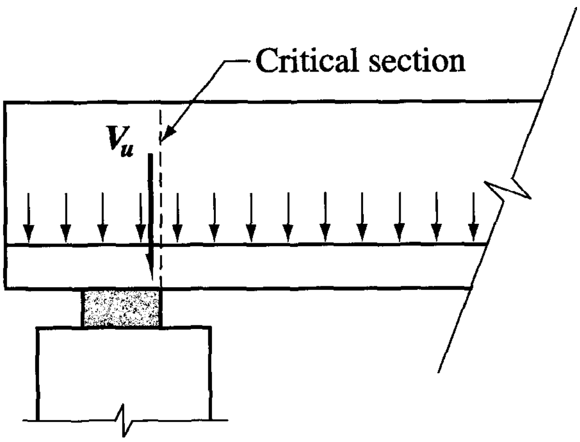


Fig. R11.1.3.1(b) - Location of critical section for shear in a member loaded near bottom

R11.1.3.1 The closest inclined crack to the support of the beam in Fig. R11.1.3.1(a) will extend upwards from the face of the support reaching the compression zone about d from the face of the support. If loads are applied to the top of this beam, the stirrups across this crack are stressed by loads acting on the lower freebody in Fig. R11.1.3.1(a). The loads applied to the beam between the face of the column and the point d away from the face are transferred directly to the support by compression in the web above the crack. Accordingly, the SBC 304 permits design for a maximum factored shear force V_u at a distance d from the support for nonprestressed members, and at a distance $h/2$ for prestressed members. Two things are emphasized: first, stirrups are required across the potential crack designed for the shear at d from the support, and second, a tension force exists in the longitudinal reinforcement at the face of the support.

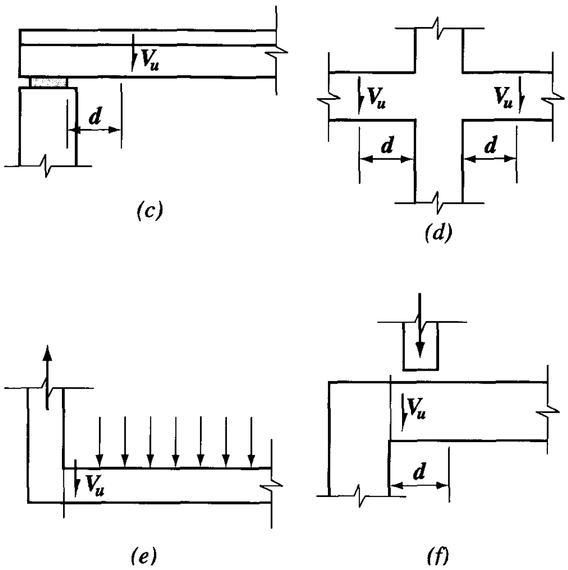


Fig. R11.1.3.1(c), (d), (e), (f) - Typical support conditions for locating factored shear force V_u

In Fig. R11.1.3.1(b), loads are shown acting near the bottom of a beam. In this case, the critical section is taken at the face of the support. Loads acting near the support should be transferred across the inclined crack extending upward from the support face. The shear force acting on the critical section should include all loads applied below the potential inclined crack.

Typical support conditions where the shear force at a distance d from the support may be used include: (1) members supported by bearing at the bottom of the member, such as shown in Fig. R11.1.3.1(c); and (2) members framing monolithically into another member as illustrated in Fig. R11.1.3.1(d).

Support conditions where this provision should not be applied include: (1) Members framing into a supporting member in tension, such as shown in Fig. R11.1.3.1(e). For this case, the critical section for shear should be taken at the face of the support. Shear within the connection should also be investigated and special corner reinforcement should be provided. (2) Members for which loads are not applied at or near the top of the member. This is the condition referred to in Fig. 11.1.3.1(b). For such cases the critical section is taken at the face of the support. Loads acting near the support should be transferred across the inclined crack extending upward from the support face. The shear force acting on the critical section should include all loads applied below the potential inclined crack. (3) Members loaded such that the shear at sections between the support and a distance d from the support differs radically from the shear at distance d. This commonly occurs in brackets and in beams where a concentrated load is located close to the support, as shown in Fig. R11.1.3.1(f) or in footings supported on piles. In this case the shear at the face of' the support should be used.

R11.1.3.2 Because d frequently varies in prestressed members, the location of the critical section has arbitrarily been taken as $h/2$ from the face of the support.

SECTION R11.2
LIGHTWEIGHT CONCRETE

Two alternative procedures are provided to modify the provi sions for shear and torsion when lightweight aggregate concrete is used. The lightweight concrete modification applies only to the terms containing $\sqrt{f_c^{'}}$ in the equations of Chapter 11 SBC 304.

R11.2.1.1 The first alternative bases the modification on laboratory tests to determine the relationship between splitting tensile strength f_{ct} and the compressive strength $f_c^{'}$ for the lightweight concrete being used. For normal weight concrete, the splitting tensile strength f_{ct} is approximately equal to, $\sqrt{f_c^{'}}$ /1.8.[11.10,11.11]

R11.2.1.2 The second alternative bases the modification on the assumption that the tensile strength of lightweight concrete is a fixed fraction of the tensile strength of normalweight concrete[11.12]. The multipliers are based on data from tests[11.13] on many types of structural lightweight aggregate concrete.

SECTION R11.3
SHEAR STRENGTH PROVIDED BY CONCRETE FOR NONPRESTRESSED MEMBERS

R11.3.1.1 See R11.3.2.1

R11.3.1.2 and
R11.3.1.3 See R11.3.2.2.

R11.3.2.1 Eq. (11-5) is the basic expression for shear strength of members without shear reinforcement.[11.3] Designers should recognize that the three variables in Eq. (11-5), $\sqrt{f_c^{'}}$ (as a measure of concrete tensile strength), ρ_w and $V_u d/M_u$, are known to affect shear strength, although some research data [11.1.1,1.14] indicate that Eq. (11-5) overestimates the influence of $f_c^{'}$ and underestimates the influence of ρ_w and $V_u d/M_u$. Further information[11.15] has indicated that shear strength decreases as the overall depth of the member increases.

The minimum value of M_u equal to $V_u d$ in Eq. (11-5) is to limit V_c near points of inflection.

For most designs, it is convenient to assume that the second term of Eq. (11-5) equals $0.17\sqrt{f_c^{'}}$ and use V_c equal to $(1/6)\sqrt{f_c^{'}}b_w d$ as permitted in 11.3.1.1.

R11.3.2.2 Eq. (11-6) and (11-7), for members subject to axial compression in addition to shear and flexure are derived in Reference 11.3. As N_u is increased, the value of

V_c computed from Eq. (11-5) and (11-6) will exceed the upper limit given by Eq. (11-7) before the value of M_m given by Eq. (11-6) becomes negative. The value of V_c obtained from Eq. (11-5) has no physical significance if a negative value of M_m is substituted. For this condition, Eq. (11-7) or Eq. (11-4) should be used to calculate V_c. Values of V_c for members subject to shear and axial load are illustrated in Fig. R11.3.2.2. The background for these equations is discussed and comparisons are made with test data in Reference 11.2.

Because of the complexity of Eq. (11-5) and (11-6), an alternative design provision, Eq. (11-4), is permitted.

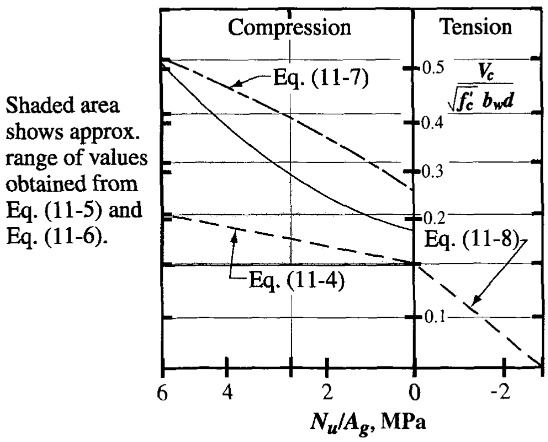


Fig. R11.3.2.2 - Comparison of shear strength equations for members subject to axial load

R11.3.2.3 Eq. (11-8) may be used to compute V_c for members subject to significant axial tension. Shear reinforcement may then be designed for $V_n - V_c$. The term significant is used to recognize that a designer must use judgment in deciding whether axial tension needs to be considered. Low levels of axial tension often occur due to volume changes, but are not important in structures with adequate expansion joints and minimum reinforcement. It may be desirable to design shear reinforcement to carry total shear if there is uncertainty about the magnitude of axial tension.

R11.3.3 Shear tests of members with circular sections indicate that the effective area can be taken as the gross area of the section or as an equivalent rectangular area.[11.1,11.16,11.17]

SECTION 11.4
SHEAR STRENGTH PROVIDED BY CONCRETE FOR PRESTRESSED MEMBERS

R11.4.1 Eq. (11-9) offers a simple means of computing V_c for prestressed concrete beams.[11.2] It may be applied to beams having prestressed reinforcement only, or to members reinforced with a combination of prestressed reinforcement and nonprestressed deformed bars. Eq. (11-9) is most applicable to members subject to uniform loading and may give conservative results when applied to composite girders for bridges.

In applying Eq. (11-9) to simply supported members subject to uniform loads $V_u d / M_u$ can be expressed as

$$\frac{V_u d}{M_u} = \frac{d(\ell - 2x)}{x(\ell - x)}$$

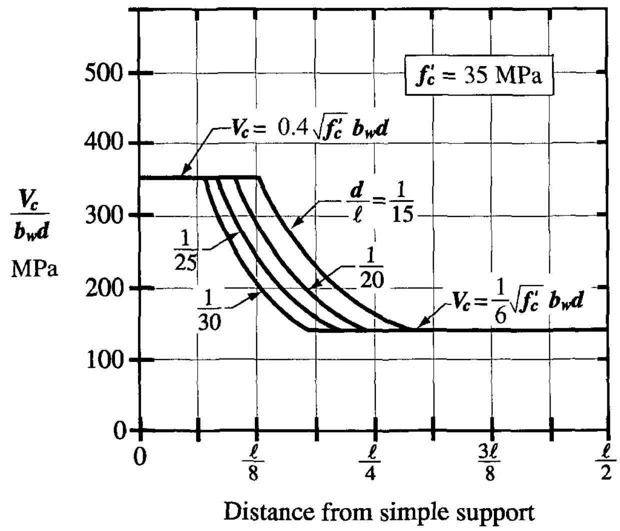


Fig. R11.4.1 - Application of Eq. (11-9) to uniformly loaded prestressed members

where ℓ is the span length and x is the distance from the section being investigated to the support. For concrete with f_c' equal to 35 MPa, V_c from 11.4.1 varies as shown in Fig. R11.4.1. Design aids based on this equation are given in Reference 11.18.

R11.4.2 Two types of inclined cracking occur in concrete beams: web-shear cracking and flexure-shear cracking. These two types of inclined cracking are illustrated in Fig. R11.4.2.

Web-shear cracking begins from an interior point in a member when the principal tensile stresses exceed the tensile strength of the concrete. Flexure-shear cracking is initiated by flexural cracking. When flexural cracking occurs, the shear stresses in the concrete above the crack are increased. The flexure-shear crack develops when the combined shear and tensile stress exceeds the tensile strength of the concrete.

Eqn. (11-11) Eq. (11-10) and (11-12) may be used to determine the shear forces causing flexure-shear and web-shear cracking, respectively. The shear strength provided by the concrete V_c is assumed equal to the lesser of V_{ci} and V_{cw}. The derivations of Eq. (11-10) and (11-12) are summarized in Reference 11.19.

In deriving Eq. (11-10) it was assumed that V_{ci} is the sum of the shear required to cause a flexural crack at the point in question given by:

$$V = \frac{V_i M_{cr}}{M_{\max}}$$

plus an additional increment of shear required to change the flexural crack to a flexure-shear crack. The externally applied factored loads, from which V_i and M_{max} are determined, include superimposed dead load, earth pressure, and live load. In computing M_{cr} for substitution into Eq. (11-10), I and y_t, are the properties of the section resisting the externally applied loads.

For a composite member, where part of the dead load is resisted by only a part of the section, appropriate section properties should be used to compute f_d. The shear due to dead loads, V_d and that due to other loads V_i are separated in this case.

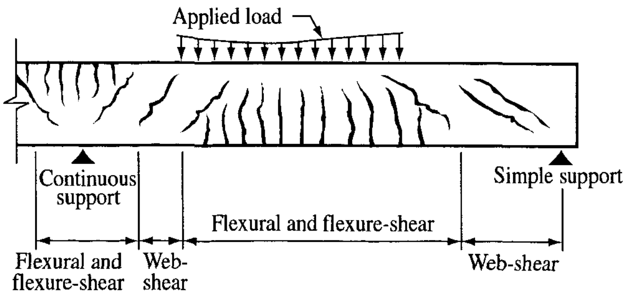


Fig. R11.4.2 - Types of cracking in concrete beams

V_d is then the total shear force due to unfactored dead load acting on that part of the section carrying the dead loads acting prior to composite action plus the unfactored superimposed dead load acting on the composite member. The terms V_i and M_{max} may be taken as:

$$V_i = V_u - V_d$$
$$M_{max} = M_u - M_d$$

where V_u and M_u are the factored shear and moment due to the total factored loads, and M_d is the moment due to unfactored dead load (the moment corresponding to f_d.

For noncomposite, uniformly loaded beams, the total cross section resists all the shear and the live and dead load shear force diagrams are similar. In this case Eq. (11-10) reduces to:

$$V_{ci} = \frac{\sqrt{f_c'}}{20} b_w d + \frac{V_u M_{ct}}{M_u}$$

where:

$$M_{ct} = (I / y_t)\left(\sqrt{f_c'} + f_{pe}\right)$$

The symbol M_{ct} in the two preceding equations represents the total moment, including dead load, required to cause cracking at the extreme fiber in tension. This is not the same as M_{cr}, in SBC 304 Eq. (11-10) where the cracking moment is that due to all loads except the dead load. In Eq. (11-10) the dead load shear is added as a separate term.

M_u is the factored moment on the beam at the section under consideration, and V_u is the factored shear force occurring simultaneously with M_u. Since the same section properties apply to both dead and live load stresses, there is no need to compute dead load stresses and shears separately. The cracking moment M_{ct} reflects the total stress change from effective prestress to a tension of $\sqrt{f_c^{'}}/2$, assumed to cause flexural cracking.

Eq. (11-12) is based on the assumption that web-shear cracking occurs due to the shear causing a principal tensile stress of approximately $(1/3)\sqrt{f_c^{'}}$ at the centroidal axis of the cross section. V_p is calculated from the effective prestress force without load factors.

R11.4.3 and R11.4.4 The effect of the reduced prestress near the ends of pretensioned beams on the shear strength should be taken into account. Section 11.4.3 relates to the shear strength at sections within the transfer length of prestressing steel when bonding of prestressing steel extends to the end of the member.

Section 11.4.4 relates to the shear strength at sections.within the length over which some of the prestressing steel is not bonded to the concrete, or within the transfer length of the prestressing steel for which bonding does not extend to the end of the beam.

SECTION R11.5
SHEAR STRENGTH PROVIDED BY SHEAR REINFORCEMENT

R11.5.2 Limiting the design yield strength of shear reinforcement to 420 MPa provides a control on diagonal crack width. Research[11.20,11.21,11.22] has indicated that the performance of higher strength steels as shear reinforcement has been satisfactory. In particular**,** full-scale beam tests described in Reference 11.21 indicated that the widths of inclined shear cracks at service load levels were less for beams reinforced with smaller diameter deformed welded wire fabric cages designed on the basis of a yield strength of 520 MPa than beams reinforced with deformed Grade 420 stirrups.

R11.5.3 It is essential that shear (and torsion) reinforcement be adequately anchored at both ends to be fully effective on either side of any potential inclined crack. This generally requires a hook or bend at the end of the reinforcement as provided by 12.13.

R11.5.5 Minimum shear reinforcement

R11.5.5.1 Shear reinforcement restrains the growth of inclined cracking. Ductility is increased and a warning of failure is provided. In an unreinforced web, the sudden formation of inclined cracking might lead directly to failure without warning. Such reinforcement is of great value if a member is subjected to an unexpected tensile force or an overload. Accordingly, a minimum area of shear reinforcement not less than that given by Eq. (11-13) or (11-14) is required wherever the total factored shear force V_u is greater than one-half the shear strength provided by concrete ϕV_c Slabs, footings and joists are excluded from the minimum shear reinforcement requirement because there is a possibility of load sharing between weak and strong areas. However, research results[11.23] have shown that deep, lightly reinforced one-way slabs, particularly if constructed with high-strength concrete, may fail at shear loads less than V_u, calculated from Eq. (11-3).

Even when the total factored shear strength V_u is less than one-half of the shear strength provided by the concrete ϕV_c the use of some web reinforcement is recommended in all: thin-web post-tensioned prestressed concrete members (joists, waffle slabs, beams, and T-beams) to reinforce against tensile forces in webs resulting from local deviations from the design tendon profile, and to provide a means of supporting the tendons in the design profile during construction. If sufficient support is not provided, lateral wobble and local deviations from the smooth parabolic tendon profile assumed in design may result during placement of the concrete. In such cases, the deviations in the tendons tend to straighten out when the tendons are stressed. This process may impose large tensile stresses in webs, and severe cracking may develop if no web reinforcement is provided. Unintended curvature of the tendons, and the resulting tensile stresses in webs, may be minimized by securely tying tendons to stirrups that are rigidly held in place by other elements of the reinforcing cage and held down in the forms. The maximum spacing of stirrups used for this purpose should not exceed the smaller of $1.5h$ or 1.2 m. When applicable, the shear reinforcement provisions of 11.5.4 and 11.5.5 will require closer stirrup spacings.

For repeated loading of flexural members, the possibility of inclined diagonal tension cracks forming at stresses appreciably smaller than under static loading should be taken into account in the design. In these instances, it would be prudent to use at least the minimum shear reinforcement expressed by Eq. (11-13) or (11-14), even though tests or calculations based on static loads show that shear reinforcement is not required.

R11.5.5.2 When a member is tested to demonstrate that its shear and flexural strengths are adequate, the actual member dimensions and material strengths are known. The strength used as a basis for comparison should therefore be that corresponding to a strength reduction factor of unity ($\phi = 1.0$), i.e. the required nominal strength V_n and M_n. This ensures that if the actual material strengths in the field were less than specified, or the member dimensions were in error such as to result in a reduced member strength, a satisfactory margin of safety will be retained.

R11.5.5.3 Tests[11.9] have indicated the need to increase the minimum area of shear reinforcement as concrete strength increases to prevent sudden shear failures when inclined cracking occurs. Equation (11-13) provides for a gradual increase in the minimum area of transverse reinforcement, while maintaining the previous minimum value.

R11.5.5.4 Tests[11.24] of prestressed beams with minimum web reinforcement based on Eq. (11-13) and (11-14) indicated that the smaller A_v from these two equations was sufficient to develop ductile behavior.

Eq. (11-14) may be used only for prestressed members meeting the minimum prestress force requirements given in 11.5.5.4. This equation is discussed in Reference 11.24.

R11.5.6 Design of shear reinforcement.

Design of shear reinforcement is based on a modified truss analogy. The truss analogy assumes that the total shear is carried by shear reinforcement. However, considerable research on both non-prestressed and prestressed members has indicated that shear reinforcement needs to be designed to carry only the shear exceeding that which causes inclined cracking, provided the diagonal members in the truss are assumed to be inclined at 45 deg.

Eq. (11-15), (11-16), and (11-17) are presented in terms of shear strength V_s attributed to the shear reinforcement. When shear reinforcement perpendicular to axis of member is used, the required area of shear reinforcement A_v and its spacing s are computed by

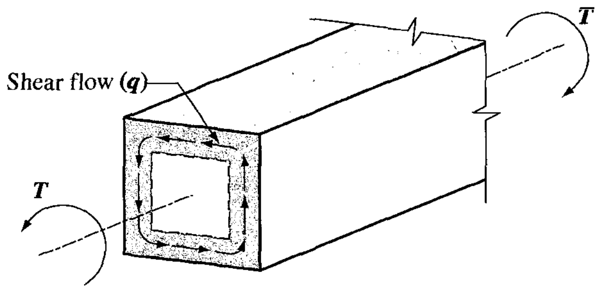
$$\frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_y d}$$

Research[11.25,11.26] has shown that shear behavior of wide beams with substantial flexural reinforcement is improved if the transverse spacing of stirrup legs across the section is reduced.

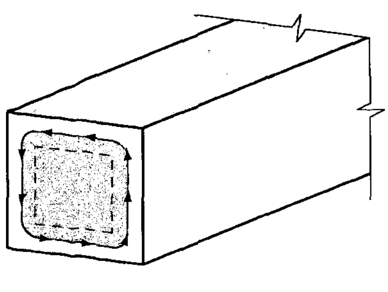
R11.5.6.3 Although the transverse reinforcement in a circular section may not consist of straight legs, tests indicate that Eq. (11-15) is conservative if d *is* taken as defined in 11.3.3.[11.16, 11.17]

SECTION R11.6
DESIGN FOR TORSION

The design for torsion is based on a thin-walled tube, space truss analogy. A beam subjected to torsion is idealized as a thin-walled tube with the core concrete cross section in a solid beam neglected as shown in Fig. R11.6(a). Once a reinforced concrete beam has cracked in torsion, its torsional resistance is provided primarily by closed stirrups and longitudinal bars located near the surface of the member. In the thin-walled tube analogy the resistance is assumed to be provided by the outer skin of the cross section roughly centered on the closed stirrups. Both hollow and solid sections are idealized as thin-walled tubes both before and after cracking.



(a) Thin-walled tube



(b) Area enclosed by shear flow path

Fig. R11.6 - (a) Thin-walled tube; (b) area enclosed by shear flow path

In a closed thin-walled tube, the product of the shear stress τ and the wall thickness t at any point in the perimeter is known as the shear flow, $q = \tau t$. The shear flow q due to torsion acts as shown in Fig. R11.6(a) and is constant at all points around the perimeter of the tube. The path along which it acts extends around the tube at midthickness of the walls of the tube. At any point along the perimeter of the tube the shear stress due to torsion is $\tau = T/(2A_o t)$ where A_o is the gross area enclosed by the shear flow path, shown shaded in Fig. R11.6(b), and t is the thickness of the wall at the point where τ is being computed. The shear flow follows the midthickness of the walls of the tube and A_o is the area enclosed by the path of the shearflow. For a hollow member with continuous walls, A_o includes the area of the hole.

The elliptical interaction between the shear carried by the concrete, V_c and the torsion carried by the concrete is not considered. V_c remains constant at the value it has when there is no torsion, and the torsion carried by the concrete is always taken as zero.

The design procedure is derived and compared with test results in References 11.27 and 11.28.

R11.6.1 Threshold torsion.

Torques that do not exceed approximately one-quarter of the cracking torque T_{cr} will not cause a structurally significant reduction in either the flexural or shear strength and can be ignored. The cracking torsion under pure torsion T_{cr} is derived by replacing the actual section with an equivalent thin-walled tube with a wall thickness t prior to cracking of $0.75A_{cp}/P_{cp}$ and an area enclosed by the wall centerline A_o equal to $2A_{cp}/3$. Cracking is assumed to occur when the principal tensile stress reaches $(1/3)\sqrt{f_c'}$. In a nonprestressed beam loaded with torsion alone, the principal tensile stress is equal to the torsional shear stress, $\tau = T/(2A_o t)$. Thus, cracking occurs when τ reaches $(1/3)\sqrt{f_c'}$ giving the cracking torque T_{cr} as:

$$T_{cr} = \frac{1}{3}\sqrt{f_c'}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

For solid members, the interaction between the cracking torsion and the inclined cracking shear is approximately circular or elliptical. For such a relationship, a torque of $0.25T_{cr}$ as used in 11.6.1, corresponds to a reduction of 3% in the inclined cracking shear. This reduction in the inclined cracking shear was considered negligible. The stress at cracking $(1/3)\sqrt{f_c'}$ has purposely been taken as a lower bound value for prestressed members, the torsional cracking load is increased by the prestress. A Mohr's Circle analysis based on average stresses indicates the torque required to cause a principal tensile stress equal to $(1/3)\sqrt{f_c'}$ is $\sqrt{1+3f_{pc}/\sqrt{f'_c}}$ times the corresponding torque in a nonprestressed beam. A similar modification is made in part (c) of 11.6.1 for members subjected to axial load and torsion.

For torsion, a hollow member is defined as having one or more longitudinal voids, such as a single-cell or multiple-cell box girder. Small longitudinal voids, such as un-grouted post-tensioning ducts that result in A_g / A_{cp} greater than or equal to 0.95, can be ignored when computing the threshold torque in 11.6.1. The interaction between torsional cracking and shear cracking for hollow sections is assumed to vary from the elliptical relationship for members with small voids, to a straight-line relationship for thin-walled sections with large voids. For a straight-line interaction, a torque of $0.25T_{cr}$ would cause a reduction in the inclined cracking shear of about 25%. This reduction was judged to be excessive.

Two changes are made to modify 11.6.1 to apply to hollow sections. First, the minimum torque limits are multiplied by A_g / A_{cp} because tests of solid and hollow beams[11.29] indicate that the cracking torque of a hollow section is approximately A_g / A_{cp} times the cracking torque of a solid section with the same outside dimensions. The second change is to multiply the cracking torque by A_g / A_{cp} a second time to reflect the transition from the circular interaction between the inclined cracking loads in shear and torsion for solid members, to the approximately linear interaction for thin-walled hollow sections.

R11.6.2 Calculation of factored torsional moment T_u

R11.6.2.1 and
R11.6.2.2 In designing for torsion in reinforced concrete structures, two conditions may be identified:[11.30,11.31]

(a) The torsional moment cannot be reduced by redistribution of internal forces (11.6.2.1). This is referred to as equilibrium torsion, since the torsional moment is required for the structure to be in equilibrium.

For this condition, illustrated in Fig. R11.6.2.1, torsion reinforcement designed according to 11.6.3 through 11.6.6 must be provided to resist the total design torsional moments.

(b) The torsional moment can be reduced by redistribution of internal forces after cracking (11.6.2.2) if the torsion arises from the member twisting to maintain compatibility of deformations. This type of torsion is referred to as compatibility torsion.

For this condition, illustrated in Fig. R11.6.2.2, the torsional stiffness before cracking corresponds to that of the uncracked section according to St. Venant's theory. At torsional cracking, however, a large twist occurs under an essentially constant torque, resulting in a large redistribution of forces in the structure.[11.30,11.31] The cracking torque under combined shear, flexure, and torsion corresponds to a principal tensile stress somewhat less than the $(1/3)\sqrt{f_c'}$ quoted in R11.6.1.

When the torsional moment exceeds the cracking torque, a maximum factored torsional moment equal to the cracking torque may be assumed to occur at the critical sections near the faces of the supports. This limit has been established to control the width of torsional cracks. The replacement of A_{cp} with A_g, as in the calculation of the threshold torque for hollow sections in 11.6.1, is not applied here. Thus, the torque after redistribution is larger and hence more conservative.

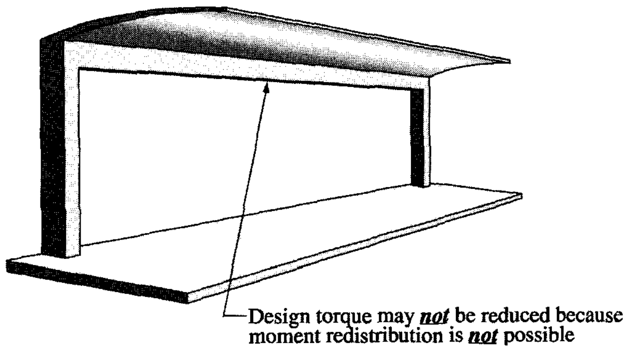


Fig. R11.6.2.1 - Design torque may not be reduced (11.6.2.1)

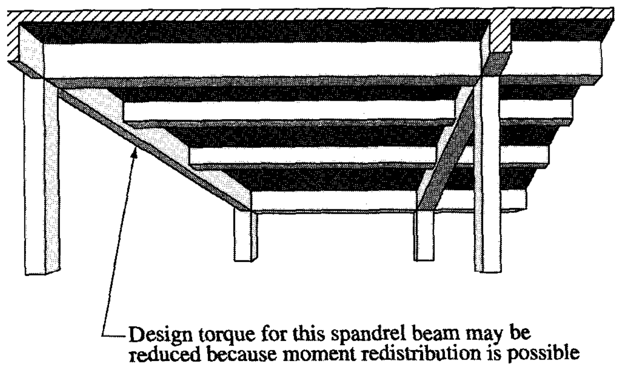


Fig. R11.6.2.2 - Design torque may be reduced (11.6.2.2)

Section 11.6.2.2 applies to typical and regular framing conditions. With layouts that impose significant torsional rotations within a limited length of the member, such as a heavy in 11.6.2.2. Torque loading located close to a stiff column, or a column that rotates in the reverse directions because of other loading, a more exact analysis is advisable.

When the factored torsional moment from an elastic analysis based on uncracked section properties is between the values in 11.6.1 and the values given in this section, torsion reinforcement should be designed to resist the computed torsional moments.

R11.6.2.4 and
R11.6.2.5 It is not uncommon for a beam to frame into one side of a girder near the support of the girder. In such a case a concentrated shear and torque are applied to the girder.

R11.6.3 Torsional moment strength

R11.6.3.1 The size of a cross section is limited for two reasons, first to reduce unsightly cracking and second to prevent crushing of the surface concrete due to inclined compressive stresses due to shear and torsion. In Eq. (11-18) and (11-19), the two terms on the left hand side are the shear stresses due to shear and torsion. The sum of these stresses may not exceed the stress causing shear cracking plus $2/3\sqrt{f_c'}$ similar to the limiting strength given in 11.5.6.9 for shear without torsion. The limit is expressed in terms of V_c to allow its use for nonprestressed or prestressed concrete. It was originally derived on the basis of crack control. It is not necessary to check against crushing of the web since this happens at higher shear stresses.

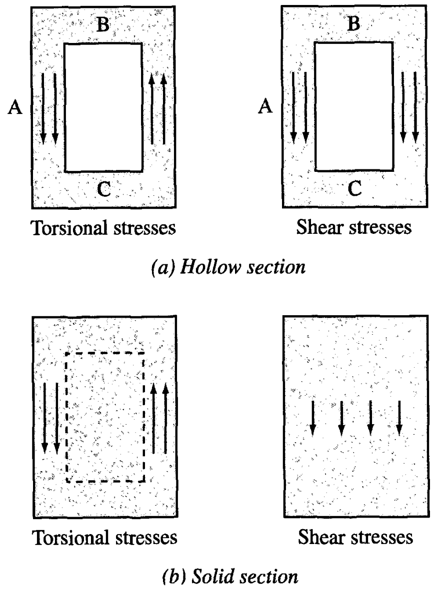


Fig. R11.6.3.1 - Addition of torsional and shear stresses

In a hollow section, the shear stresses due to shear and torsion both occur in the walls of the box as shown in Fig. 11.6.3.1(a) and hence are directly additive at point A as given in Eq. (11-19). In a solid section the shear stresses due to torsion act in the "tubular" outside section while the shear stresses due to V_u. are spread across the width of the section as shown in Fig. R11.6.3.1(b). For this reason stresses are combined in Eq. (11-18) using the square root of the sum of the squares rather than by direct addition.

R11.6.3.2 Generally, the maximum will be on the wall where the torsional and shearing stresses are additive [Point A in Fig. R11.6.3.1(a)]. If the top or bottom flanges are thinner than the vertical webs, it may be necessary to evaluate Eq. (11-19) at points B and C in Fig. R11.6.3.1(a). At these points the stresses due to the shear force are usually negligible.

R11.6.3.4 Limiting the design yield strength of torsion reinforcement to 420 MPa provides a control on diagonal crack width.

R11.6.3.5 The factored torsional resistance ϕT_n must equal or exceed the torsion T_n due to the factored loads. In the calculation of T_n all the torque is assumed to be resisted by stirrups and longitudinal steel with $T_c = 0$. At the same time, the shear resisted by concrete V_c is assumed to be unchanged by the presence of torsion. For beams with V_u greater than about $0.8\phi V_c$ the resulting amount of combined shear and torsional reinforcement is essentially the same. For smaller values of V_u, more shear and torsion reinforcement will be required.

R11.6.3.6 Eq. (11-21) is based on the space truss analogy shown in Fig. R11.6.3.6(a) with compression diagonals at an angle θ, assuming the concrete carries no tension and the reinforcement yields. After torsional cracking develops, the torsional resistance is provided mainly by closed stirrups, longitudinal bars, and compression diagonals. The concrete outside these stirrups is relatively ineffective. For this reason A_o , the area enclosed by the shear flow path around the perimeter of the tube, is defined after cracking in terms of A_{oh} , the area enclosed by the centerline of the outermost closed hoops. The area A_{oh} is shown in Fig. R11.6.3.6(b) for various cross sections. In an I-, T-, or L-shaped section, A_{oh} is taken as that area enclosed by the outermost legs of interlocking stirrups as shown in Fig. R11.6.3.6(b). The expression for A_o given in Reference 11.32 may be used if greater accuracy is desired.

The shear flow q in the walls of the tube, discussed in R11.6, can be resolved into the shear forces V_1 to V_4 acting in the individual sides of the tube or space truss, as shown in Fig. R11.6.3.6(a).

The angle θ can be obtained by analysis[11.32] or may be taken to be equal to the values given in 11.6.3.6 (a) or (b). The same value of θ should be used in both Eq. (11-21) and (11-22). As θ gets smaller, the amount of stirrups required by Eq. (11-21) decreases. At the same time the amount of longitudinal steel required by Eq. (11-22) increases.

R11.6.3.7 Fig. R11.6.3.6(a) shows the shear forces V_1 to V_4 resulting from the shear flow around the walls of the tube. On a given wall of the tube, the shear flow V_i is resisted tube by a diagonal compression component, $D_i = V_i / \sin\theta$, in the concrete. An axial tension force, $N_i = V_i(\cot\theta)$, is needed in he longitudinal steel to complete the resolution of V_i.

Fig. R11.6.3.7 shows the diagonal compressive stresses and the axial tension force, N_i, acting on a short segment along one wall of the tube. Because the shear flow due to torsion is constant at all points around the perimeter of the tube, the resultants of D_i and N_i act through the midheight of side i. As a result, half of N_i can be assumed to be resisted by each of the top and bottom chords as shown. Longitudinal reinforcement with a capacity $A_e f_{ye}$ should be provided to resist the sum of the N_i forces, $\sum N_i$, acting in all of the walls of the tube.

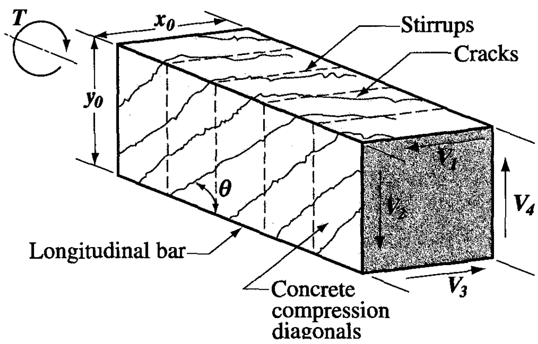


Fig. R11.6.3.6(a) - Space truss analogy

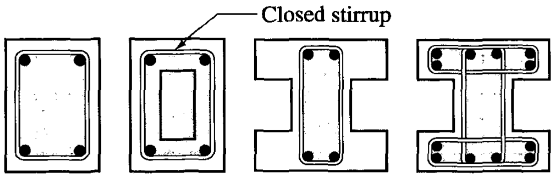


Fig. R11.6.3.6(b) - Definition of A_{oh}

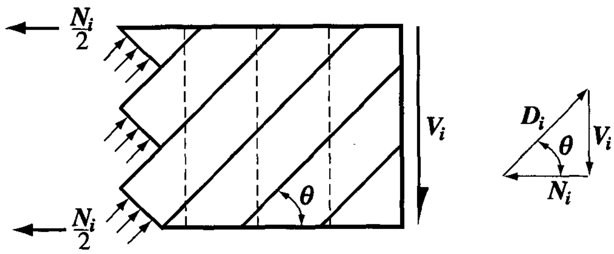


Fig. R11.6.3.7 - Resolution of shear force V_i; into diagonal compression force D_i; and axial tension force N_i in one wall of the tube

In the derivation of Eq. (11-22), axial tension forces are summed along the sides of the area A_o. These sides form a perimeter length, p_o, approximately equal to the length of the line joining the centers of the bars in the corners of the tube. For ease in computation this has been replaced with the perimeter of the closed stirrups, p_h.

Frequently, the maximum allowable stirrup spacing governs the amount of stirrups provided. Furthermore, when combined shear and torsion act, the total stirrup area is the sum of the amounts provided for shear and torsion. To avoid the need to provide excessive amounts of longitudinal reinforcement, 11.6.3.7 states that the A_t / s used in calculating A_l at any given section should be taken as the A_t / s calculated at that section using Eq. (11-21).

R11.6.3.8 The stirrup requirements for torsion and shear are added and stirrups are provided to supply at least the total amount required. Since the stirrup area A_v for shear is defined in terms of all the legs of a given stirrup while the stirrup area A_t for torsion is defined in terms of one leg only, the addition of stirrups is carried out as follows:

$$Total\left(\frac{A_v + t}{s}\right) = \frac{A_v}{s} + 2\frac{A_t}{s}$$

If a stirrup group had four legs for shear, only the legs adjacent to the sides of the beam would be included in this summation since the inner legs would be ineffective for torsion.

The longitudinal reinforcement required for torsion is added at each section to the longitudinal reinforcement required for bending moment that acts at the same time as the torsion. The longitudinal reinforcement is then chosen for this sum, but should not be less than the amount required for the maximum bending moment at that section if this exceeds the moment acting at the same time as the torsion. If the maximum bending moment occurs at one section, such as the midspan, while the maximum torsional moment occurs at another, such as the support, the total longitudinal steel required may be less than that obtained by adding the maximum flexural steel plus the maximum torsional steel. In such a case the required longitudinal steel is evaluated at several locations.

The most restrictive requirements for spacing, cut-off points, and placement for flexural, shear, and torsional steel should be satisfied. The flexural steel should be extended a distance d, but not less than $12d_b$, past where it is no longer needed for flexure as required in 12.10.3.

R11.6.3.9 The longitudinal tension due to torsion is offset in part by the compression in the flexural compression zone, allowing a reduction in the longitudinal torsion steel required in the compression zone.

R11.6.3.10 As explained in R11.6.3.7, torsion causes an axial tension force. In a nonprestressed beam this force is resisted by longitudinal reinforcement having an axial tensile capacity of $A_\ell f_{y\ell}$. This steel is in addition to the flexural reinforcement and is distributed uniformly around the sides of the perimeter so that the resultant of $A_\ell f_{y\ell}$ acts along the axis of the member.

In a prestressed beam, the same technique (providing additional reinforcing bars with capacity $A_\ell f_{y\ell}$) can be followed, or the designer can use any overcapacity of the prestressing steel to resist some of the axial force $A_\ell f_{y\ell}$ outlined in the next paragraph.

In a prestressed beam, the prestressing steel stress at ultimate at the section of the maximum moment is f_{ps}. At other sections, the prestressing steel stress at ultimate will be between f_{se} and f_{ps}. A portion of the $A_\ell f_{y\ell}$ force acting on the sides of the perimeter where the prestressing steel is located can be resisted by a force $A_{ps}\Delta f_p$ in the prestressing steel, where Δf_p is f_{ps} minus the prestressing steel stress due to flexure at the ultimate load at the section in question. This can be taken as M_u at the section, divided by $(\phi 0.9 d_p A_{ps})$, but Δf_p should not be more than 420 MPa.

Longitudinal reinforcing bars will be required on the other sides of the member to provide the remainder of the $A_\ell f_{y\ell}$ force, or to satisfy the spacing requirements given in 11.6.6.2, or both.

R11.6.4 Details of torsional reinforcement

R11.6.4.1 Both longitudinal and closed transverse reinforcement are required to resist the diagonal tension stresses due to torsion. The stirrups must be closed, since inclined cracking due to torsion may occur on all faces of a member.

In the case of sections subjected primarily to torsion, the concrete side cover over the stirrups spoils off at high torques.[11.33] This renders lapped-spliced stirrups ineffective, leading to a premature torsional failure.[11.34] In such cases, closed stirrups should not be made up of pairs of U-stirrups lapping one another.

R11.6.4.2 When a rectangular beam fails in torsion, the corners of the beam tend to spoil off due to the inclined compressive stresses in the concrete diagonals of the space truss changing direction at the corner as shown in Fig. R11.6.4.2 (a). In tests,[11.33] closed stirrups anchored by 90 deg hooks failed when this occurred. For this reason, 135 deg hooks are preferable for torsional stirrups in all cases. In regions where this spalling is prevented by an adjacent slab or flange, 11.6.4.2(b) relaxes this and allows 90 deg hooks.

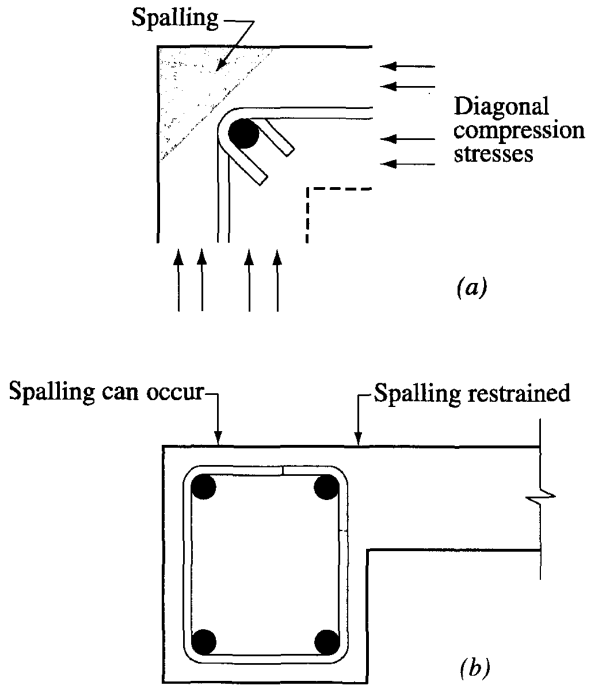


Fig. R11.6.4.2 - Spalling of corners of beams loaded in torsion

R11.6.4.3 If high torsion acts near the end of a beam, the longitudinal torsion reinforcement should be adequately anchored. Sufficient development length should be provided outside the inner face of the support to develop the needed tension force in the bars or tendons. In the case of bars, this may require hooks or horizontal U-shaped bars lapped with the longitudinal torsion reinforcement.

R11.6.4.4 The closed stirrups provided for torsion in a hollow section should be located in the outer half of the wall thickness effective for torsion where the wall thickness can be taken as A_{oh} / p_h.

R11.6.5 Minimum torsion reinforcement

R11.6.5.1 and
R11.6.5.2 If a member is subject to a factored torsional moment T_u greater than the values specified in 11.6.1, the minimum amount of transverse web reinforcement for combined shear and torsion is $0.35 b_w s / f_y$. The differences in the definition of A_v, and the symbol A_t should be noted; A_v is the area of two legs of a closed stirrup while A_t is the area of only one leg of a closed stirrup.

Tests[11.9] of high-strength reinforced concrete beams have indicated the need to increase the minimum area of shear reinforcement to prevent shear failures when inclined cracking occurs. Although there are a limited number of tests of high-strength concrete beams in torsion, the equation for the minimum area of transverse closed stirrups has been changed for consistency with calculations required for minimum shear reinforcement.

R11.6.5.3 Reinforced concrete beam specimens with less than 1 percent torsional reinforcement by volume have failed in pure torsion at torsional cracking.[11.27]

R11.6.6 Spacing of torsion reinforcement

R11.6.6.1 The spacing of the stirrups is limited to ensure the development of the ultimate torsional strength of the beam, to prevent excessive loss of torsional stiffness after cracking, and to control crack widths. For a square cross section the $p_h/8$ limitation requires stirrups at $d/2$, which corresponds to 11.5.4.1.

R11.6.6.2 In R11.6.3.7 it was shown that longitudinal reinforcement is needed to resist the sum of the longitudinal tensile forces due to torsion in the walls of the thin-walled tube. Since the force acts along the centroidal axis of the section, the centroid of the additional longitudinal reinforcement for torsion should approximately coincide with the centroid of the section. The code accomplishes this by requiring the longitudinal torsional reinforcement to be distributed around the perimeter of the closed stirrups. Longitudinal bars or tendons are required in each corner of the stirrups to provide anchorage for the legs of the stirrups. Corner bars have also been found to be very effective in developing torsional strength and in controlling cracks.

R11.6.6.3 The distance $(b_t + d)$ beyond the point theoretically required for torsional reinforcement is larger than that used for shear and flexural reinforcement because torsional diagonal tension cracks develop in a helical form.

SECTION R11.7
SHEAR-FRICTION

R11.7.1 With the exception of 11.7, virtually all provisions regarding shear are intended to prevent diagonal tension failures rather than direct shear transfer failures. The purpose of 11.7 is to provide design methods for conditions where shear transfer should be considered: an interface between concretes cast at different times, an interface between concrete and steel, reinforcement details for precast concrete structures, and other situations where it is considered appropriate to investigate shear transfer across a plane in structural concrete. (See References 11.35 and 11.36).

R11.7.3 Although untracked concrete is relatively strong in direct shear there is always the possibility that a crack will form in an unfavorable location. The shear-friction concept assumes that such a crack will form, and that reinforcement must be provided across the crack to resist relative displacement along it. When shear acts along a crack, one crack face slips relative to the other. If the crack faces are rough and irregular, this slip is accompanied by separation of the crack faces. At ultimate, the separation is sufficient to stress the reinforcement crossing the crack to its yield point. The reinforcement provides a clamping force $A_{vf} f_y$ across the crack faces. The applied shear is then resisted by friction between the crack faces, by resistance to the shearing off of protrusions on the crack faces, and by dowel action of the reinforcement crossing the crack. Successful application of 11.7 depends on proper selection of the location of an assumed crack.[11.18.11.35]

The relationship between shear-transfer strength and the reinforcement crossing the shear plane can be expressed in various ways. Eq. (11-25) and (11-26) of 11.7.4 are based on the shear-friction model. This gives a conservative prediction of shear-transfer strength. Other relationships that give a closer estimate of shear-transfer strength[11.18,11.37,11.38] can be used under the provisions of 11.7.3. For example when the shear-friction reinforcement is perpendicular to the shear plane, the shear strength V_n is given by[11.37, 11.38]

$$V_n = 0.8 A_{vf} f_v + A_c K_1$$

where A_c is the area of concrete section resisting shear transfer (mm^2) and K_1 = 2.8 MPa for normalweight concrete, 1.4 MPa for all-lightweight concrete, and 1.7 MPa for sand-lightweight concrete. These values of K_1 apply to both monolithically cast concrete and to concrete cast against hardened concrete with a rough surface, as defined in 11.7.9.

In this equation, the first term represents the contribution of friction to shear-transfer resistance (0.8 representing the coefficient of friction). The second term represents the sum of the resistance to shearing of protrusions on the crack faces and the dowel action of the reinforcement.

When the shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in that reinforcement, the shear strength V_n is given by

$$V_n = A_{vf} f_y \left(0.8 \sin\alpha_f + \cos\alpha_f\right) + A_c K_1 \sin^2\alpha_f$$

where α_f is the angle between the shear-friction reinforcement and the shear plane, (i.e. $0 < \alpha_f <$ 90 deg).

When using the modified shear-friction method, the terms $\left(A_{vf} f_y / A_c\right)$ or $\left(A_{vf} f_y \sin\alpha_f / A_c\right)$ should not be less than 1.4 MPa for the design equations to be valid.

R11.7.4 Shear-friction design method

R11.7.4.1 The required area of shear-transfer reinforcement A_{vf} is computed using

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$

The specified upper limit on shear strength should also be observed.

R11.7.4.2 When the shear-friction reinforcement is inclined to the shear plane, such that the component of the shear force parallel to the reinforcement tends to produce tension in the reinforcement, as shown in Fig. R11.7.4, part of the shear is resisted by the component parallel to the shear plane of the tension force in the reinforcement. Eq. (11-26) should be used only when the shear force component parallel to the reinforcement produces tension in the reinforcement, as shown in Fig. R11.7.4. When α_f is greater than 90 deg, the relative movement of the surfaces tends to compress the bar and Eq. (11-26) is not valid.

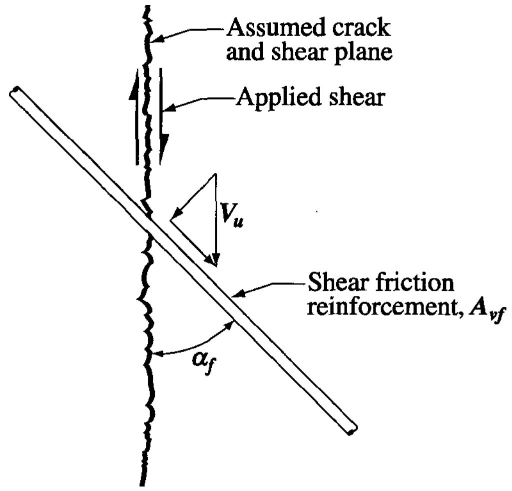


Fig. R11.7.4 - Shear-friction reinforcement at an angle to assumed crack

R11.7.4.3 In the shear-friction method of calculation, it is assumed that all the shear resistance is due to the friction between the crack faces. It is, therefore, necessary to use artificially high values of the coefficient of friction in the shear-friction equations so that the calculated shear strength will be in reasonable agreement with test results. For concrete cast against hardened concrete not roughened in accordance with 11.7.9, shear resistance is primarily due to dowel action of the reinforcement and tests[1139] indicate that reduced value of $\mu = 0.6\lambda$ specified for this case is appropriate.

The value of for concrete placed against as-rolled structural steel relates μ specified to the design of connections between precast concrete members, or between structural steel members and structural concrete members. The shear-transfer reinforcement may be either reinforcing bars or headed stud; shear connectors; also, field welding to steel plates after casting of concrete is common. The design of shear connectors for composite action of concrete slabs and steel beams is not covered by these provisions, but should be in accordance with Reference 11.40.

R11.7.5 This upper limit on shear strength is specified because Eq. (11-25) and (11-26) become un-conservative if V_n has a greater value.

R11.7.7 If a resultant tensile force acts across a shear plane, reinforcement to carry that tension should be provided in addition to that provided for shear transfer. Tension may be caused by restraint of deformations due to temperature change, creep, and shrinkage. Such tensile forces have caused failures, particularly in beam bearings.

When moment acts on a shear plane, the flexural tension stresses and flexural compression stresses are in equilibrium. There is no change in the resultant compression $A_{vf} f_y$ acting across the shear plane and the shear-transfer strength is not changed. It is therefore not necessary to provide additional reinforcement to resist the flexural tension stresses, unless the required flexural tension reinforcement exceeds the amount of shear-transfer reinforcement provided in the flexural tension zone. This has been demonstrated experimentally.[11.41]

It has also been demonstrated experimentally [11.36] that if a resultant compressive force acts across a shear plane, the shear-transfer strength is a function of the sum of the resultant compressive force and the force $A_{vf} f_y$ in the shear-friction reinforcement. In design, advantage should be taken of the existence of a compressive force across the shear plane to reduce the amount of shear-friction reinforcement required, only if it is certain that the compressive force is permanent.

R11.7.8 If no moment acts across the shear plane, reinforcement should be uniformly distributed along the shear plane to minimize crack widths. If a moment acts across the shear plane, it is desirable to distribute the shear-transfer reinforcement primarily in the flexural tension zone.I

Since the shear-friction reinforcement acts in tension, it should have full tensile anchorage on both sides of the shear plane. Further, the shear-friction reinforcement anchorage should engage the primary reinforcement, otherwise a potential crack may pass between the shear-friction reinforcement and the body of the concrete. This requirement applies particularly to welded headed studs used with steel inserts for connections in precast and cast-in-place concrete. Anchorage may be developed by bond, by a welded mechanical anchorage, or by threaded dowels and screw inserts. Space limitations often require a welded mechanical anchorage. For anchorage of headed studs in concrete see Reference 11.18.

SECTION R11.8
DEEP BEAMS

R11.8.1 The behavior of a deep beam is discussed in References 11.5 and 11.38. For a deep beam supporting gravity loads, this section applies if the loads are applied on the top of the beam and the beam is supported on its bottom face. If the loads are applied through the sides or bottom of such a member, the design for shear should be the same as for ordinary beams.

The longitudinal reinforcement in deep beams should be extended to the supports and adequately anchored by embedment, hooks, or welding to special devices. Bent-up bars are not recommended.

R11.8.2 Deep beams can be designed using strut-and-tie models, regardless of how they are loaded and supported. Section 10.7.1 allows the use of nonlinear stress fields when proportioning deep beams. Such analyses should consider the effects of cracking on the stress distribution.

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R11.8.5 The relative amounts of horizontal and vertical shear reinforcement are set based on tests[11.42,11.43,11.44] that have shown that vertical shear reinforcement is more effective than horizontal shear reinforcement. The maximum spacing of bars is limited to 300 mm to restrain the width of the cracks.

SECTION R11.9
SPECIAL PROVISIONS FOR BRACKETS AND CORBELS

Brackets and corbels are cantilevers having shear span-to-depth ratios not greater than unity, which tend to act as simple trusses or deep beams, rather than flexural members designed for shear according to 11.3.

The corbel shown in Fig. R11.9.1 may fail by shearing along the interface between the column and the corbel, by yielding of the tension tie, by crushing or splitting of the compression strut, or by localized bearing or shearing failure under the loading plate. These failure modes are illustrated and are discussed more fully in Reference 11.1. The notation used in 11.9 is illustrated in Fig. R11.9.2.

R11.9.1 An upper limit of 1.0 for a/d is imposed for design by 11.9.3 and 11.9.4 for two reasons. First, for shear span-to-depth ratios exceeding unity, the diagonal tension cracks are less steeply inclined and the use of horizontal stirrups alone as specified in 11.9.4 is not appropriate. Second, this method of design has only been validated experimentally for a/d of unity or less. An upper limit is provided for N_{uc} because this method of design has only been validated experimentally for N_{uc} less than or equal to V_u including N_{uc}, equal to zero.

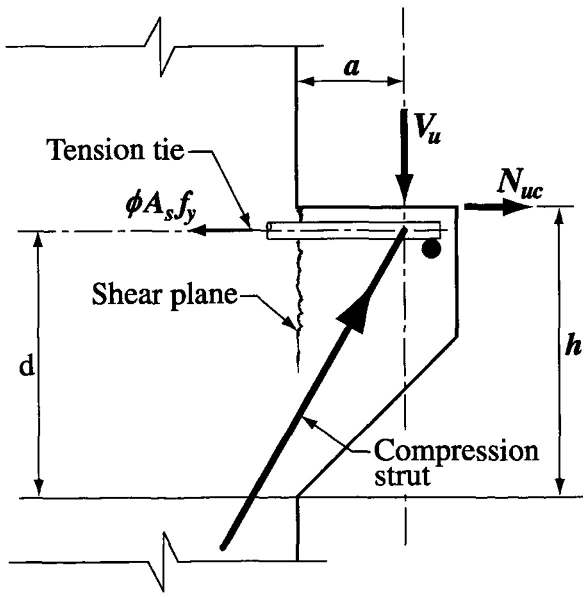


Fig. R11.9.1 - Structural action of a corbel

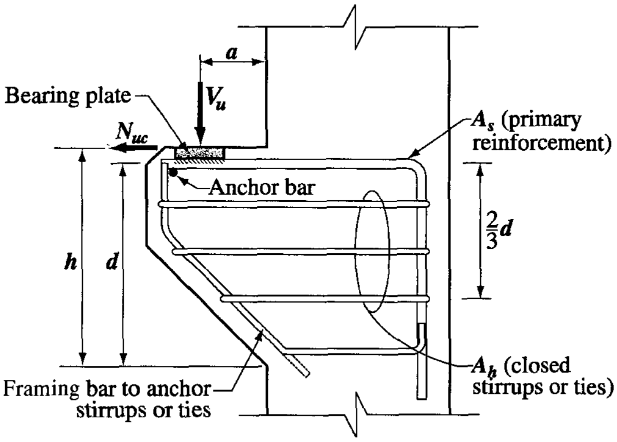


Fig. R11.9.2 - Notation used in Section 11.9

R11.9.2 A minimum depth is required at the outside edge of the bearing area so that a premature failure will not occur due to a major diagonal tension crack propagating from below the bearing area to the outer sloping face of the corbel or bracket. Failures of this type have been observed [11.45] in corbels having depths at the outside edge of the bearing area less than required in this section of the code.

R11.9.3.1 Corbel and bracket behavior is predominantly controlled by shear; therefore, a single value of $\phi = 0.75$ is required for all design conditions.

R11.9.3.2.2 Tests [11.46] have shown that the maximum shear strength of lightweight concrete corbels or brackets is a function of both f_c' and a/d. No data are available for corbels or brackets made of sand-lightweight concrete. As a result, the same limitations have been placed on both all-lightweight and sand-lightweight brackets and corbels.

R11.9.3.3 Reinforcement required to resist moment can be calculated using flexural theory. The factored moment is calculated by summing moments about the flexural reinforcement at the face of support.

R11.9.3.4 Because the magnitude of horizontal forces acting on corbels or brackets cannot usually be determined with great accuracy, it is required that N_{uc} be regarded as a live load.

R11.9.3.5 Tests [11.46] suggest that the total amount of reinforcement $(A_s + A_h)$ required to cross the face of support should be the greater of:

a) The sum of A_{vf} calculated according to 11.9.3.2 and A_n calculated according to 11.9.3.4;

b) The sum of 1.5 times A_f calculated according to 11.9.3.3 and A_n calculated according to 11.9.3.4.

If (a) controls, $A_s = (2A_{vf}/3 + A_n)$ is required as primary tensile reinforcement, and the remaining $A_{vf}/3$ should be provided as closed stirrups parallel to A_s and distributed within $2d/3$, adjacent to A_s. Section 11.9.4 satisfies this by requiring $A_h = 0.5(2A_{vf}/3)$.

If (b) controls, $A_s = (A_f + A_n)$ is required as primary tension reinforcement, and the remaining $A_f/2$ should be provided as closed stirrups parallel to A_s and distributed within $2d/3$, adjacent to A_s. Again 11.9.4 satisfies this requirement.

R11.9.4 Closed stirrups parallel to the primary tension reinforcement are necessary to prevent a premature diagonal tension failure of the corbel or bracket. The required area of closed stirrups $A_h = 0.5(A_s - A_n)$ automatically yields the appropriate amounts, as discussed in R11.9.3.5 above.

R11.9.5 A minimum amount of reinforcement is required to prevent the possibility of sudden failure should the bracket or corbel concrete crack under the action of flexural moment and outward tensile force N_{uc}.

R11.9.6 Because the horizontal component of the inclined concrete compression strut (see Fig. R11.9.1) is transferred to the primary tension reinforcement at the location of the vertical load, the reinforcement A_s is essentially uniformly stressed from the face of the support to the point where the vertical load is applied. It should, therefore, be anchored at its outer end and in the supporting column, so as to be able to develop its yield strength from the face of support to the vertical load. Satisfactory anchorage at the outer end can be obtained by bending the A_s bars in a horizontal loop as specified in (b), or by welding a bar of equal diameter or a suitably sized angle across the ends of the A_s bars. The

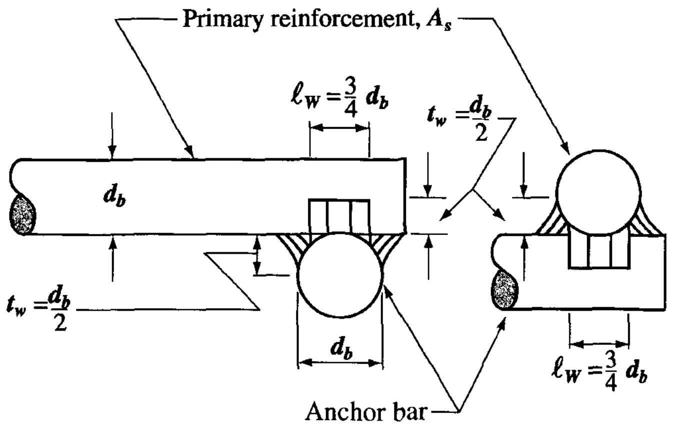


Fig. R11.9.6 - Weld details used in tests of Reference 11.45

welds should be designed to develop the yield strength of the reinforcement A_s. The weld detail used successfully in the corbel tests reported in Reference 11.45 is shown in Fig. R11.9.6. The reinforcement A_s should be anchored within the supporting column in accordance with the requirements of Chapter 12. See additional discussion on end anchorage in R12.10.6.

R11.9.7 The restriction on the location of the bearing area is necessary to ensure development of the yield strength of the reinforcement A_s near the load. When corbels are designed to resist horizontal forces, the bearing plate should be welded to the tension reinforcement A_s.

SECTION R11.10
SPECIAL PROVISIONS FOR WALLS

R11.10.1 Shear in the plane of the wall is primarily of importance for shear walls with a small height-to-length ratio. The design of higher walls, particularly walls with uniformly distributed reinforcement will probably be controlled by flexural considerations.

R11.10.3 Although the width-to-depth ratio of shear-walls is less than that for ordinary beams, tests [11.47] on shear-walls with a thickness equal to $\ell_w/25$ have indicated that ultimate shear stresses in excess of $(5/6)\sqrt{f_c'}$; can be obtained.

R11.10.5 and
R11.10.6 Eq. (11-29) and (11-30) may be used to determine the inclined cracking strength at any section through a shear wall. Eq. (11-29) corresponds to the occurrence of a principal tensile stress of approximately $(1/3)\sqrt{f_c'}$ at the centroid of the shear wall cross section. Eq. (11-30) corresponds approximately to the occurrence of a flexural tensile stress of $(1/2)\sqrt{f_c'}$ at a section $\ell_w/2$ above the section being investigated. As the term

$$\left(\frac{M_u}{V_u} - \frac{\ell_w}{2}\right)$$

decreases, Eq. (11-29) will control before this term becomes negative. When this term becomes negative Eq. (11-29) should be used.

R11.10.7 The values of V_c computed from Eq. (11-29) and (11-30) at a section located a lesser distance of $\ell_w/2$ or $h_w/2$ above the base apply to that and all sections between this section and the base. However, the maximum factored shear force V_u at any section, including the base of the wall, is limited to ϕV_n in accordance with 11.10.3.

R11.10.9 **Design of shear reinforcement for walls**
Both horizontal and vertical shear reinforcement are required for all walls. For low walls, test data[11.48] indicate that horizontal shear reinforcement becomes less effective with vertical reinforcement becoming more effective. This change in effectiveness of the horizontal versus vertical reinforcement is recognized in Eq. (11-32); when h_w/ℓ_w is less than 0.5, the amount of vertical reinforcement is equal to the amount of horizontal reinforcement.

When h_w / ℓ_w is greater than 2.5, only a minimum amount of vertical reinforcement is required $(0.0025 s_1 h)$.

Eq. (11-31) is presented in terms of shear strength V_s provided by the horizontal shear reinforcement for direct application in Eq. (11-1) and (11-2). Vertical shear reinforcement also should be provided in accordance with 11.10.9.4 within the spacing limitation of 11.10.9.5.

SECTION R11.11
TRANSFER OF MOMENTS TO COLUMNS

R11.11.1 Tests[11.49] have shown that the joint region of a beam-to-column connection in the interior of a building does not require shear reinforcement if the joint is confined on four sides by beams of approximately equal depth. However, joints without lateral confinement, such as at the exterior of a building, need shear reinforcement to prevent deterioration due to shear cracking[11.50].

SECTION R11.12
SPECIAL PROVISIONS FOR SLABS AND FOOTINGS

11.12.1 Differentiation should be made between a long and narrow slab or footing acting as a beam, and a slab or footing subject to two-way action where failure may occur by punching along a truncated cone or pyramid around a concentrated load or reaction area.

R11.12.1.2 The critical section for shear in slabs subjected to bending in two directions follows the perimeter at the edge of the loaded area.[11.3] The shear stress acting on this section at factored loads is a function of $\sqrt{f_c'}$, and the ratio of the side dimension of the column to the effective slab depth. A much simpler design equation results by assuming a pseudocritical section located at a distance $d/2$ from the periphery of the concentrated load. When this is done, the shear strength is almost independent of the ratio of column size to slab depth. For rectangular columns, this critical section was defined by straight lines drawn parallel to and at a distance $d/2$ from the edges of the loaded area. Section 11.12.1.3 allows the use of a rectangular critical section.

For slabs of uniform thickness, it is sufficient to check shear on one section. For slabs with changes in thickness, such as the edge of drop panels, it is necessary to check shear at several sections.

For edge columns at points where the slab cantilevers beyond the column, the critical perimeter will either be three-sided or four-sided.

R11.12.2.1 For square columns, the shear stress due to ultimate loads in slabs subjected to bending in two directions is limited to $(1/3)\sqrt{f_c'}$. However, tests (Ref. 11.51) have indicated that the value of $(1/3)\sqrt{f_c'}$ is unconservative when the ratio β_c of the lengths of the long and short sides of a rectangular column or loaded area is larger than 2.0. In such cases, the actual shear stress on the critical section at punching shear failure varies from a maximum of about $(1/3)\sqrt{f_c'}$ around the corners of the column or loaded area, down to $(1/6)\sqrt{f_c'}$ or less along the long sides between the two end sections. Other tests (Ref. 11.52) indicates that v_c, decreases as the ratio b_o/d increases. Eq. (11-33) and (11-34) were developed to account for these two effects. The words "interior," "edge," and "corner columns" in 11.12.2.1(b) refer to critical sections with 4, 3, and 2 sides respectively.

For shapes other than rectangular, β_c is taken to be the ratio of the longest overall dimension of the effective loaded area to the largest overall perpendicular dimension of the effective loaded area, as illustrated for an L-shaped reaction area in Fig. R11.12.2. The effective loaded area is that area totally enclosing the actual loaded area, for which the perimeter is a minimum.

R11.12.2.2 For prestressed slabs and footings, a modified form of code Eq. (11-33) and (11-36) is specified for two-way action shear strength. Research[11.53,11.54] indicates that the shear strength of two-way prestressed slabs around interior columns is conservatively predicted by Eq. (11-36). V_c from Eq. (11-36) corresponds to a diagonal tension failure of the concrete initiating at the critical section defined in 11.12.1.2. The mode of failure differs from a punching shear failure of the concrete compression zone around the perimeter of the loaded area predicted by Eq. (11-33). Consequently, the term β_c does not enter into Eq. (11-36). Design values for f_c' and f_{pc} are restricted due to limited test data available for higher values. When computing f_{pc}, loss of prestress due to restraint of the slab by shearwalls and other structural elements should be taken into account.

In a prestressed slab with distributed tendons, the V_p term in Eq. (11-36) contributes only a small amount to the shear strength; therefore, it may be conservatively taken as zero. If V_p is to be included, the tendon profile assumed in the calculations should be noted.

For an exterior column support where the distance from the outside of the column to the edge of the slab is less than four times the slab thickness, the prestress is not fully effective around the total perimeter b_a of the critical section. Shear strength in this case is therefore conservatively taken the same as for a nonprestressed slab.

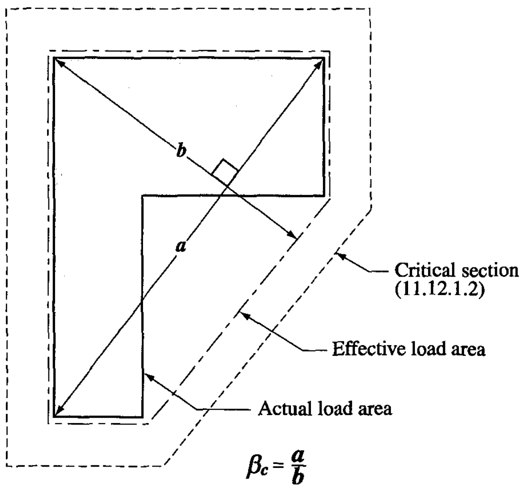


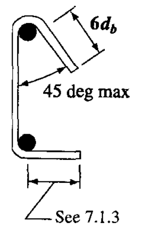
Fig. R11.12.2 - Value of β_c for a nonrectangular loaded area

R11.12.3 Research [11.55-11.59] has shown that shear reinforcement consisting of properly anchored bars or wires and single- or multiple-leg stirrups, or closed stirrups, can increase the punching shear resistance of slabs. The spacing limits given in 11.12.3.3 correspond to slab shear reinforcement details that have been shown to be effective. Sections 12.13.2 and 12.13.3 give anchorage requirements for stirrup-type shear reinforcement that should also be applied for bars or wires used as slab shear reinforcement. It is essential that this shear reinforcement engage longitudinal reinforcement at both the top and bottom of the slab, as shown for typical details in Fig. R11.12.3 (a) to (c). Anchorage of shear reinforcement according to the requirements of 12.13 is difficult in slabs thinner than 250 mm. Shear reinforcement consisting of vertical bars mechanically anchored at each end by a plate or head capable of developing the yield strength of the bars has been used successfully.[11.59]

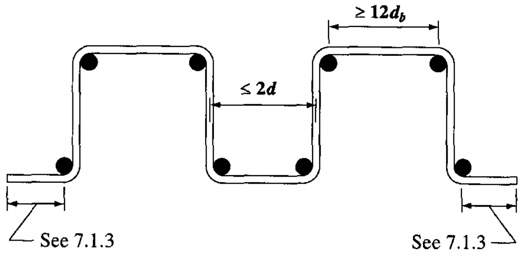
In a slab-column connection for which the moment transfer is negligible, the shear reinforcement should be symmetrical about the centroid of the critical section (Fig. R11.12.3 (d)). Spacing limits defined in 11.12.3.3 are also shown in Fig. R11.12.3 (d) and (e). At edge columns or for interior connections where moment transfer is significant, closed stirrups are recommended in a pattern as symmetrical as possible. Although the average shear stresses on faces *AD* and *BC* of the exterior column in Fig. R11.12.3 (e) are lower than on face *AB,* the closed stirrups extending from faces *AD* and *BC* provide some torsional capacity along the edge of the slab.

R11.12.4 Based on reported test data,[11.60] design procedures are presented for shearhead reinforcement consisting of structural steel shapes. For a column connection transferring moment, the design of shearheads is given in 11.12.6.3.

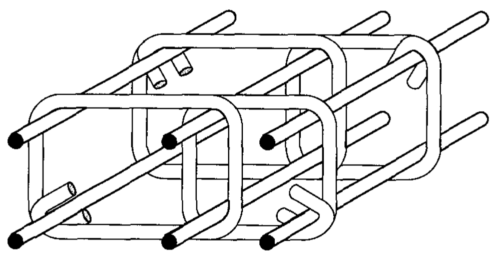
Three basic criteria should be considered in the design of shearhead reinforcement for connections transferring shear due to gravity load. First, a minimum flexural strength should be provided to ensure that the required shear strength of the slab is reached before the flexural strength of the shearhead is exceeded. Second, the shear stress in the slab at the end of the shearhead reinforcement should be limited. Third, after these two requirements are satisfied, the designer can reduce the negative slab reinforcement in proportion to the moment contribution of the shearhead at the design section.



(a) single-leg stirrup or bar



(b) multiple-leg stirrup or bar



(c) closed stirrups

Fig. R11.12.3(a-c) - Single or multiple-leg stirrup type slab shear reinforcement

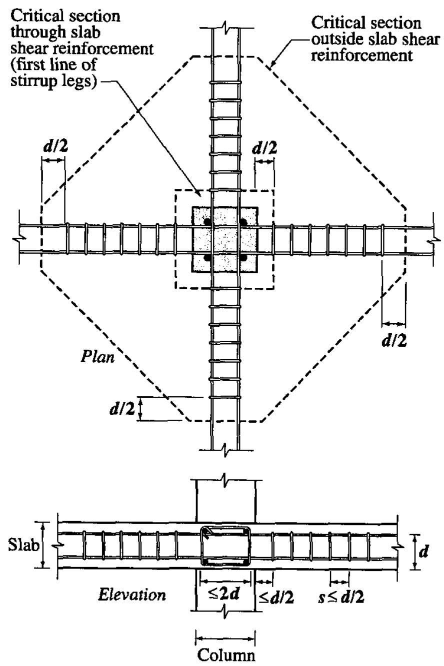


Fig. R11.12.3(d) - Arrangement of stirrup shear reinfor-cement, interior column.

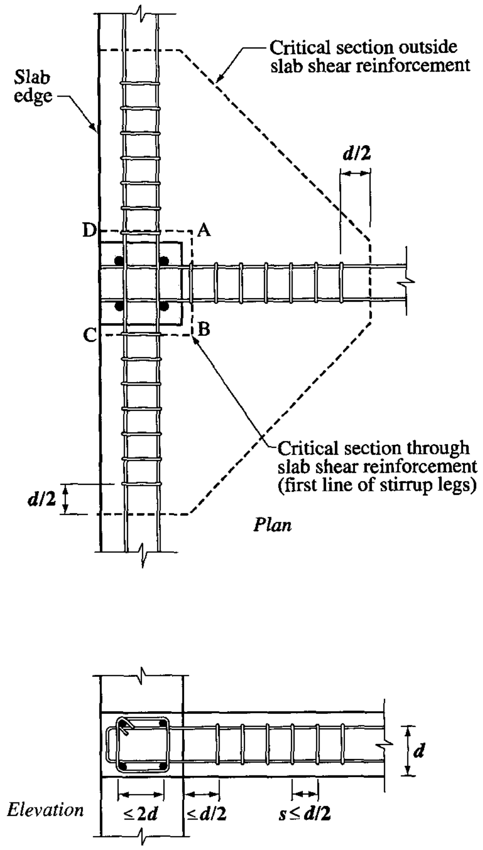


Fig. R11.12.3(e) - Arrangement of stirrup shear reinforcement, edge column

R11.12.4.5 and
R11.12.4.6 The assumed idealized shear distribution along an arm of a shearhead at an interior column is shown in Fig. R11.12.4.5. The shear along each of the arms is taken as $\alpha_v V_c / \eta$, where α_v and η are defined in 11.12.4.5 and 11.12.4.6, and V_c is defined in 11.12.2.1. However, the peak shear at the face of the column is taken as the total shear considered per arm $V_u / \phi\eta$ minus the shear considered carried to the column by the concrete compression zone of the slab. The latter term is expressed as $(V_c / \eta)(1 - \alpha_v)$, so that it approaches zero for a heavy shearhead and approaches $V_u / \phi\eta$ when a light shearhead is used. Equation (11-37) then follows from the assumption that ϕV_c is about one-half the factored shear force V_u. In this equation, M_p is the required plastic moment strength of each shearhead arm necessary to ensure that factored shear V_u is attained as the moment strength of the shearhead is reached. The quantity ℓ_v is the length from the center of the column to the point at which the shearhead is no longer required, and the distance $c_1 / 2$ is one-half the dimension of the column in the direction considered.

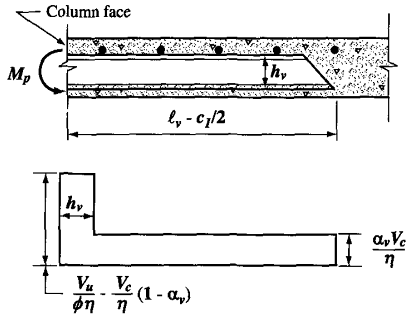


Fig. R11.12.4.5 - Idealized shear acting on shear head.

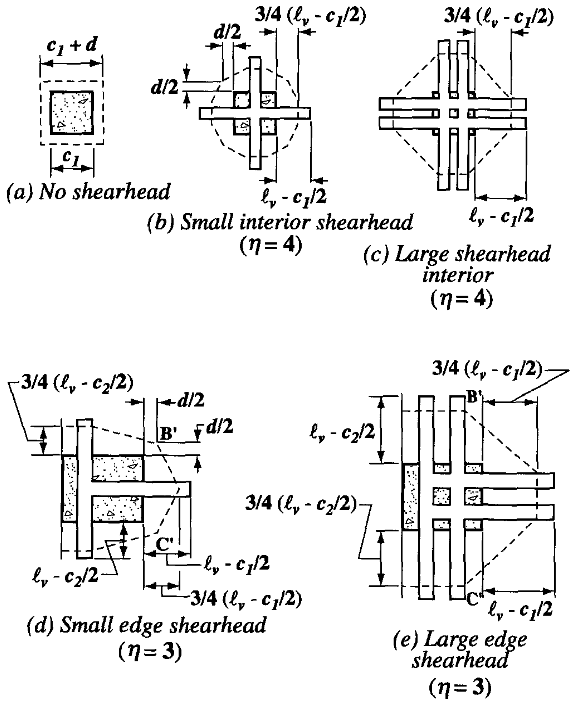


Fig. R11.12.4.7 - Location of critical section defined in 11.12.4.

R11.12.4.7 The test results. 60 indicated that slabs containing under reinforcing shearheads failed at a shear stress on a critical section at the end of the shearhead reinforcement less than $(1/3)\sqrt{f_c'}$. Although the use of over-reinforcing shearheads brought the shear strength back to about the equivalent of $(1/3)\sqrt{f_c'}$, the limited test data suggest that a conservative design is desirable. Therefore, the shear strength is calculated as $(1/3)\sqrt{f_c'}$ on an assumed critical section located inside the end of the shearhead reinforcement.

The critical section is taken through the shearhead arms three-fourths of the distance $[\ell_v - (c_1/2)]$ from the face of the column to the end of the shearhead. However, this assumed critical section need not be taken closer than $d/2$ to the column. See Fig. R11.12.4.7.

R11.12.4.9 If the peak shear at the face of the column is neglected, and ϕV_c, is again assumed to be about one-half of V_u, the moment contribution of the shearhead M_v can be conservatively computed from Eq. (11-38), in which ϕ is the factor for flexure.

R11.12.4.10 See R11.12.6.3.

R11.12.5 Openings in slabs

Provisions for design of openings in slabs (and footings) were developed in Reference 11.3. The locations of the effective portions of the critical section near typical openings and free edges are shown by the dashed lines in Fig. R11.12.5. Additional research11.51 has confirmed that these provisions are conservative.

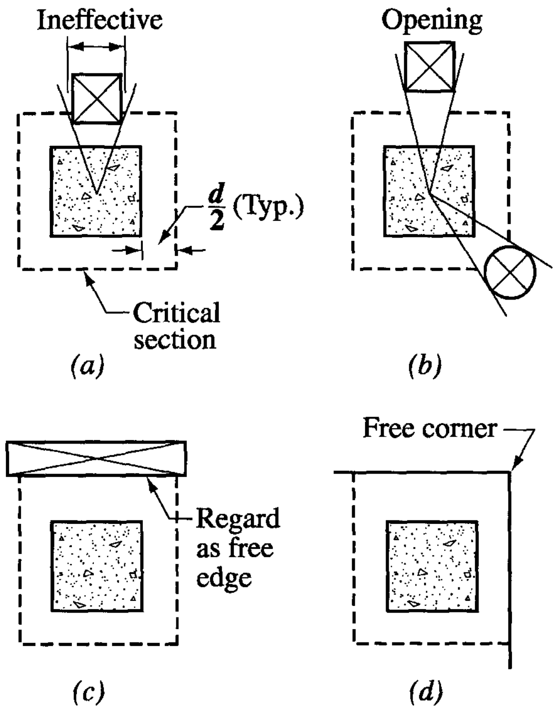


Fig. R11.12.5 - Effect of openings and free edges (effective perimeter shown with dashed lines)

R11.12.6 Transfer of moment in slab-column connections

R11.12.6.1 In Reference 11.61 it was found that where moment is transferred between a column and a slab, 60 percent of the moment should be considered transferred by flexure across the perimeter of the critical section defined in 11.12.1.2, and 40 percent by eccentricity of the shear about the centroid of the critical section. For rectangular columns, the portion of the moment transferred by flexure increases as the width of the face of the critical section resisting the moment increases, as given by Eq. (13-1).

Most of the data in Reference 11.61 were obtained from tests of square columns, and little information is available for round columns. These can be approximated as square columns. Fig. R13.6.2.5 shows square supports having the same area as some nonrectangular members.

R11.12.6.2 The stress distribution is assumed as illustrated in Fig. R11.12.6.2 for an interior or exterior column. The perimeter of the critical section, *ABCD,* is determined in accordance with 11.12.1.2. The factored shear force V_u and unbalanced moment M_u are determined at the centroidal axis c-c of the critical section. The maximum factored shear stress may be calculated from:
The maximum factored shear stress may be calculated from:

$$v_{u(AB)} = \frac{V_u}{A_c} - \frac{\gamma_v M_u c_{AB}}{J_c}$$

or

$$v_{u(CD)} = \frac{V_u}{A_c} - \frac{\gamma_v M_u c_{CD}}{J_c}$$

where γ_v is given by Eq. (11-39). For an interior column, A_c and J_c may be calculated by

A_c = area of concrete of assumed critical section

= $2d(c_1 + c_2 + 2d)$

J_c = property of assumed critical section analogous to polar moment of inertia

$$= \frac{d(c_1+d)^3}{6} + \frac{d(c_1+d)d^3}{6} + \frac{d(c_2+d)(c_1+d)^2}{2}$$

Similar equations may be developed for A_c and J_c for columns located at the edge or corner of a slab.

The fraction of the unbalanced moment between slab and column not transferred by eccentricity of the shear should be transferred by flexure in accordance with 13.5.3. A conservative method assigns the fraction transferred by flexure over an effective slab width defined in 13.5.3.2. Often designers concentrate column strip reinforcement near the column to accommodate this unbalanced moment. Available test data[11.61] seems to indicate that this practice does not increase shear strength but may be desirable to increase the stiffness of the slab-column junction.

Test data[11.62] indicate that the moment transfer capacity of a prestressed slab to column connection can be calculated using the procedures of 11.12.6 and 13.5.3.

Where shear reinforcement has been used, the critical section beyond the shear reinforcement generally has a polygonal shape (Fig. R11.12.3(d) and (e)). Equations for calculating shear stresses on such sections are given in Reference 11.58.

R11.12.6.3 Tests[11.63] indicate that the critical sections are defined in 11.12.1.2(a) and 11.12.1.3 and are appropriate for calculations of shear stresses caused by transfer of moments even when shearheads are used. Then, even though the critical sections for direct shear and shear due to moment transfer differ, they coincide or are in close proximity at the column corners where the failures initiate. Because a shearhead attracts most of the shear as it funnels toward the column, it is conservative to take the maximum shear stress as the sum of the two components.

Section 11.12.4.10 requires the moment M_p to be transferred to the column in shearhead connections transferring unbalanced moments. This may be done by bearing within the column or by mechanical anchorage.

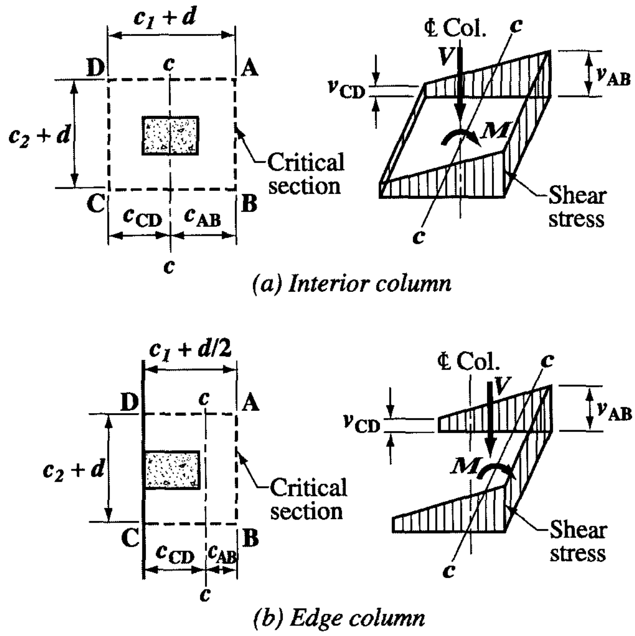


Fig. R11.12.6.2 - Assumed distribution of shear stress

CHAPTER 12
DEVELOPMENT AND SPLICES OF REINFORCEMENT

The development length concept is based on the attainable average bond stress over the length of embedment of the reinforcement. Development lengths are required because of the tendency of highly stressed bars to split relatively thin sections of restraining concrete. A single bar embedded in a mass of concrete should not require as great a development length; although a row of bars, even in mass concrete, can create a weakened plane with longitudinal splitting along the plane of the bars.

In application, the development length concept requires minimum lengths or extensions of reinforcement beyond all points of peak stress in the reinforcement. Such peak stresses generally occur at the points in 12.10.2.

The strength reduction factor ϕ is not used in the development length and lap splice equations. An allowance for strength reduction is already included in the expressions for determining development and splice lengths.

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

SECTION R12.1
DEVELOPMENT OF REINFORCEMENT GENERAL

From a point of peak stress in reinforcement, some length of reinforcement or anchorage is necessary to develop the stress. This development length or anchorage is necessary on both sides of such peak stress points. Often the reinforcement continues for a considerable distance on one side of a critical stress point so that calculations need involve only the other side, for example, the negative moment reinforcement continuing through a support to the middle of the next span.

SECTION R12.2
DEVELOPMENT OF DEFORMED BARS AND DEFORMED WIRE IN TENSION

The general development length equation (Eq. (12-1)) is given in 12.2.3. The equation is based on the expression for development length endorsed in references: 12.2, 12.3. In Eq. (12-1), c is a factor that represents the smallest of the side cover, the cover over the bar or wire (in both cases measured to the center of the bar or wire), or one-half the center-to-center spacing of the bars or wires. k_{tr} is a factor that represents the contribution of confining reinforcement across potential splitting planes. α is the traditional reinforcement location factor to reflect the adverse effects of the top reinforcement casting position. β is a coating factor reflecting the effects of epoxy coating. There is a limit on the product $\alpha\beta$. γ is a reinforcement size factor that reflects the more favorable performance of smaller diameter reinforcement. λ is a factor reflecting the lower tensile strength of lightweight concrete and the resulting reduction of the splitting resistance, which increases the development length in lightweight concrete. A limit of 2.5 is placed on the term $(c+k_{tr})/d_b$. When $(c+k_{tr})/d_b$ is less than 2.5, splitting failures are likely to occur. For values above 2.5, a pullout failure is expected and an increase in cover or transverse reinforcement is unlikely to increase the anchorage capacity.

Equation (12-1) allows the designer to see the effect of all variables controlling the development length. The designer is permitted to disregard terms when such omission results in longer and hence, more conservative, development lengths.

The provisions of 12.2.2 and 12.2.3 give a two-tier approach. The user can either calculate ℓ_d based on the actual $(c + k_{tr})/d_b$ (12.2.3) or calculate ℓ_d using 12.2.2, which is based on two preselected values of $(c + k_{tr})/d_b$.

Section 12.2.2 recognizes that many current practical construction cases utilize spacing and cover values along with confining reinforcement, such as stirrups or ties, that result in a value of $(c + k_{tr})/d_b$ of at least 1.5. Examples include a minimum clear cover of d_b along with either minimum clear spacing of $2d_b$, or a combination of minimum clear spacing of d_b and minimum ties or stirrups. For these frequently occurring cases, the development length for larger bars can be taken as $\ell_d = \left[\left(3/5 f_y \alpha\beta\lambda \;/\; \sqrt{f_c'}\right)\right] d_b$. For Dia 20 mm deformed bars and smaller, as well as for deformed wire, the development lengths could be reduced 20% using γ = 0.80. This is the basis for the middle column of the table in 12.2.2. With less cover and in the absence of minimum ties or stirrups, the minimum clear spacing limits of 7.6.1 and the minimum concrete cover requirements of 7.7 result in minimum values of c *of* d_b. Thus, for "other cases," the values are based on using $(c + k_{tr})/d_b =$ 1.0 in Eq. (12-1).

The user may easily construct simple, useful expressions. For example, in all structures with normal weight concrete (λ = 1.0), uncoated reinforcement (β = 1.0), Dia 22 mm or larger bottom bars (α = 1.0) with f_c' = 30 MPa and Grade 420 reinforcement, the equations reduce to.

$$\ell_d = \frac{(3)(420)\,(1.0)(1.0)(1.0)}{5\,\sqrt{30}}\,d_b \;= 46\,d_b$$

or

$$\ell_d = \frac{9(420)(1.0)(1.0)(1.0)}{10\,\sqrt{30}}\,d_b \;= 69\,d_b$$

Thus, as long as minimum cover of d_b is provided along with a minimum clear spacing of $2d_b$, or a minimum clear cover of d_b and a minimum clear spacing of d_b are provided along with minimum ties or stirrups, a designer knows that $\ell_d = 46d_b$. The penalty for spacing bars closer or providing less cover is the requirement that $\ell_d = 69d_b$.

Many practical combinations of side cover, clear cover, and confining reinforcement can be used with 12.2.3 to produce significantly-shorter, development lengths than allowed by 12.2.2. For example, bars or wires with minimum clear cover not less than $2d_b$ and minimum clear spacing not less than $4d_b$ and without any confining reinforcement would have $a(c + k_{tr})/d_b$ value of 2.5 and would require a development length of only $28d_b$ for the example above.

R12.2.4 The reinforcement location factor α accounts for position of the reinforcement in freshly placed concrete. Section 12.2.4 allows a lower value to be used for factor λ when the splitting tensile strength of the lightweight concrete is specified. See 5.1.4.

Studies[12.3, 12.4, 12.5] of the anchorage of epoxy-coated bars show that bond strength is reduced because the coating prevents adhesion and friction between the bar and the concrete. The factors reflect the type of anchorage failure likely to occur. When the cover or spacing is small, a splitting failure can occur and the anchorage or bond strength is substantially reduced. If the cover and spacing between bars is large, a splitting failure is precluded and the effect of the epoxy coating on anchorage strength is not as large. Studies [12.6] have shown that although the cover or spacing may be small, the anchorage strength may be increased by adding transverse steel crossing the plane of splitting, and restraining the splitting crack.

Because the bond of epoxy-coated bars is already reduced due to the loss of adhesion between the bar and the concrete, an upper limit of 1.7 is established for the product of the top reinforcement and epoxy-coated reinforcement factors.

Although there is no requirement for transverse reinforcement along the tension development or splice length, recent research [12.7, 12.8] indicates that in concrete with very high compressive strength, brittle anchorage failure occurred in bars with inadequate transverse reinforcement. In splice tests of Dia 25 and Dia 36 mm bars in concrete with an $f_c^{'}$ of approximately 100 MPa, transverse reinforcement improved ductile anchorage behavior.

R12.2.5 **Excess reinforcement.** The reduction factor based on area is not to be used in those cases where anchorage development for full f_y , is required. For example, the excess reinforcement factor does not apply for development of positive moment reinforcement at supports according to 12.1 1.2, for development of shrinkage and temperature reinforcement according to 7.12.2.3, or for development of reinforcement provided according to 7.13 and 13.3.8.5.

SECTION R12.3
DEVELOPMENT OF DEFORMED BARS AND DEFORMED WIRE IN COMPRESSION

The weakening effect of flexural tension cracks is not present for bars and wire in compression, and usually end bearing of the bars on the concrete is beneficial. Therefore, shorter development lengths are specified for compression than for tension. The development length may be reduced 25 percent when the reinforcement is enclosed within spirals or ties. A reduction in development length is also permitted if excess reinforcement is provided.

SECTION R12.4
DEVELOPMENT OF BUNDLED BARS

R12.4.1 An increased development length for individual bars is required when three or four bars are bundled together. The extra extension is needed because the grouping makes it more difficult to mobilize bond resistance from the core between the bars.

The designer should also note 7.6.6.4 relating to the cutoff points of individual bars within a bundle and 12.14.2.2 relating to splices of bundled bars. The increases in development length of 12.4 do apply when computing splice lengths of bundled bars in accordance with 12.14.2.2. The development of bundled bars by a standard hook of the bundle is not covered by the provisions of 12.5.

R12.4.2 Although splice and development lengths of bundled bars are based on the diameter of individual bars increased by 20 or 33 percent, as appropriate, it is necessary to use an equivalent diameter of the entire bundle derived from the equivalent total area of bars when determining factors in 12.2, which considers cover and clear spacing and represents the tendency of concrete to split.

SECTION R12.5
DEVELOPMENT OF STANDARD HOOKS IN TENSION

Study of failures of hooked bars indicate that splitting of the concrete cover in the plane of the hook is the primary cause of failure and that splitting originates at the inside of the hook where the local stress concentrations are very high. Thus, hook development is a direct function of bar diameter d_b, which governs the magnitude of compressive stresses on the inside of the hook. Only standard hooks (see 7.1) are considered and the influence of larger radius of bend cannot be evaluated by 12.5.

The hooked bar anchorage provisions give the total hooked bar embedment length as shown in Fig. R12.5. The development length ℓ_{dh} is measured from the critical section to the outside end (or edge) of the hook.

The development length for standard hooks ℓ_{dh} of 12.5.2 can be reduced by all applicable modification factors of 12.5.3. As an example, if the conditions of both 12.5.3(a) and (c) are met, both factors may be applied.

The effects of bar yield strength, excess reinforcement, lightweight concrete, and factors to reflect the resistance to splitting provided from confinement by concrete and transverse ties or stirrups are based on recommendations from References 12.1 and 12.2.

Tests[12.9] indicate that closely spaced ties at or near the bend portion of a hooked bar are most effective in confining the hooked bar. For construction purposes, this is not always practicable. The cases where the modification factor of 12.5.3(b) may be used are illustrated in Fig. R12.5.3(a) and (b). Figure R12.5.3(a) shows placement of ties or stirrups perpendicular to the bar being developed, spaced along the development length, ℓ_{dh} of the hook. Figure R12.5.3(b) shows placement of ties or stirrups parallel to the bar being developed along the length of the tail extension of the hook plus bend. The latter configuration would be typical in a beam column joint.

The factor for excess reinforcement in 12.5.3(d) applies only where anchorage or development for full f_y is not specifically required. Unlike straight bar development, no distinction is made between top bars and other bars; such a distinction is difficult for hooked bars in any case. A minimum value of ℓ_{dh} is specified to prevent failure by direct pullout in cases where a hook may be located very near the critical section. Hooks cannot be considered effective in compression.

Tests[12.10] indicate that the development length for hooked bars should be increased by 20 percent to account for reduced bond when reinforcement is epoxy coated.

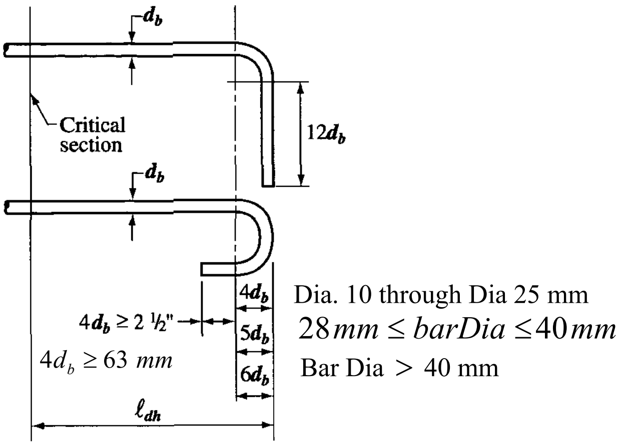


Fig. R12.5 - Hooked bar details for development of standard hooks

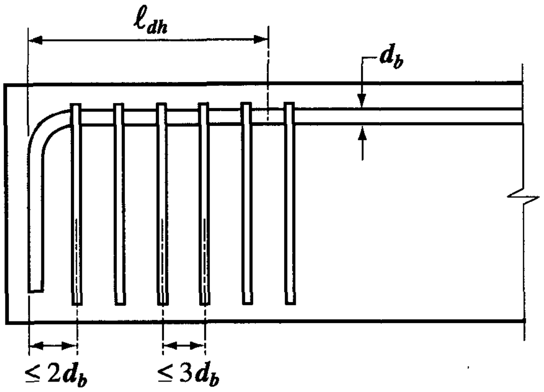


Fig. R12.5.3(a) - Ties or stirrups placed perpendicular to the bar being developed, spaced along the development length ℓ_{dh}.

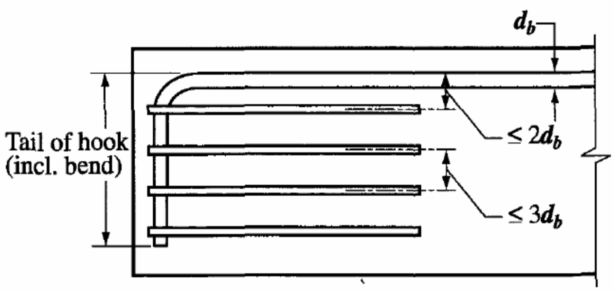


Fig. R12.5.3(b) - Ties or stirrups placed parallel to the bar being developed, spaced along the length of the tail extension of the hook plus bend.

R12.5.4 Bar hooks are especially susceptible to a concrete splitting failure if both side cover (normal to plane of hook) and top or bottom cover (in plane of hook) are small. See Fig. R12.5.4.

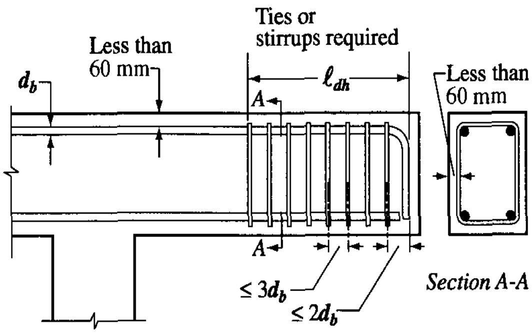


Fig. R12.5.4 - Concrete cover per 12.5.4

With minimum confinement provided by concrete, additional confinement provided by ties or stirrups is essential, especially if full bar strength should be developed by a hooked bar with such small cover. Cases where hooks may require ties or stirrups for confinement are at ends of simply supported beams, at free end of cantilevers, and at ends of members framing into a joint where members do not extend beyond the joint. In contrast, if calculated bar stress is so low that the hook is not needed for bar anchorage, the ties or stirrups are not necessary. Also, provisions of 12.5.4 do not apply for hooked bars at discontinuous ends of slabs with confinement provided by the slab continuous on both sides normal to the plane of the hook.

R12.5.5 In compression, hooks are ineffective and may not be used as anchorage.

SECTION R12.6
MECHANICAL ANCHORAGE

R12.6.1 Mechanical anchorage can be made adequate for strength both for tendons and for bar reinforcement.

R12.6.3 Total development of a bar consists of the sum of all the parts that contribute to anchorage. When a mechanical anchorage is not capable of developing the required design strength of the reinforcement, additional embedment length of reinforcement should be provided between the mechanical anchorage and the critical section.

SECTION R12.7
DEVELOPMENT OF WELDED DEFORMED WIRE FABRIC IN TENSION

Fig. R12.7 shows the development requirements for deformed wire fabric with one cross wire within the development length. ASTM A 497 for deformed wire fabric requires the same strength of the weld as required for plain wire fabric (ASTM A 185). Some of the development is assigned to welds and some assigned to the length of deformed wire. The factors in 12.7.2 are applied to the deformed wire development length computed from 12.2, but with an absolute minimum of 200 mm.

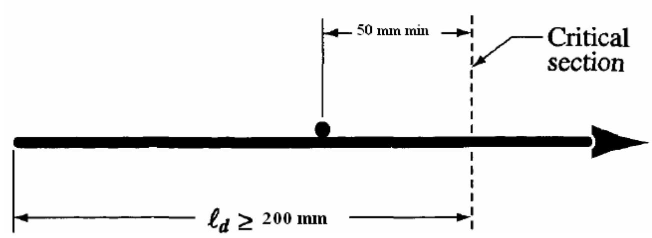


Fig. R12.7 - Development of welded deformed wire fabric

Tests[12.11] have indicated that epoxy-coated welded wire fabric has essentially the same development and splice strengths as uncoated fabric since the cross wires provide the primary anchorage for the wire. Therefore, an epoxy-coating factor of 1.0 is used for development and splice lengths of epoxy-coated welded wire fabric with cross wires within the splice or development length.

SECTION R12.8
DEVELOPMENT OF WELDED PLAIN WIRE FABRIC IN TENSION

Fig. R12.8 shows the development requirements for plain wire fabric with development primarily dependent on the location of cross wires. For fabrics made with the smaller wires, an embedment of at least two cross wires 50 mm or more beyond the point of critical section is adequate to develop the full yield strength of the anchored wires. However, for fabrics made with larger closely spaced wires, a longer embedment is required and a minimum development length is provided for these fabrics.

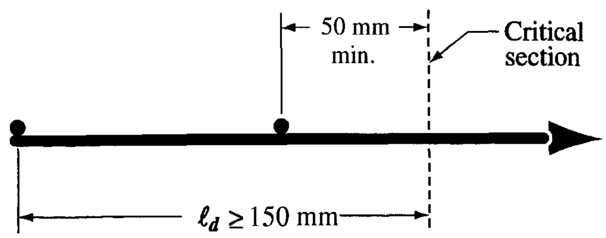


Fig. R12.8 - Development of welded plain wire fabric

SECTION R12.9
DEVELOPMENT OF PRESTRESSING STRAND

The development requirements for prestressing strand are intended to provide bond integrity for the strength of the member. The provisions are based on tests performed on normalweight concrete members with a minimum cover of 50 mm. These tests may not represent the behavior of strand in low water-cementitious materials ratio, no-slump concrete. Fabrication methods should ensure consolidation of concrete around the strand with complete contact between the steel and concrete. Extra precautions should be exercised when low water-cementitious materials ratio, no-slump concrete is used.

The first term in Eq. (12-2) represents the transfer length of the strand, that is, the distance over which the strand should be bonded to the concrete to develop the prestress f_{se} in the strand. The second term represents the additional length over which the strand should be bonded so that a stress f_{ps} may develop in the strand at nominal strength of the member.

The bond of strand is a function of a number of factors, including the configuration and surface condition of the steel, the stress in the steel, the depth of concrete beneath the strand, and the method used to transfer the force in the strand to the concrete. For bonded applications, quality assurance procedures should be used to confirm that the strand is capable of adequate bond.[12.12,12.13] The precast concrete manufacturer may rely on certification from the strand manufacturer that the strand has bond characteristics that comply with this section. Strand with a slightly rusted surface can have an appreciably shorter transfer length than clean strand. Gentle release of the strand will permit a shorter transfer length than abruptly cutting the strands.

The provisions of 12.9 do not apply to plain wires or to end-anchored tendons. The length for smooth wire could be expected to be considerably greater due to the absence of mechanical interlock. Flexural bond failure would occur with plain wire when first slip occurred.

R12.9.1.1 Figure R12.9 shows the relationship between steel stress and the distance over which the strand is bonded to the concrete represented by Eq. (12-2). This idealized variation of strand stress may be used for analyzing sections within the development region.[12.14,12.15] The expressions for transfer length, and for the

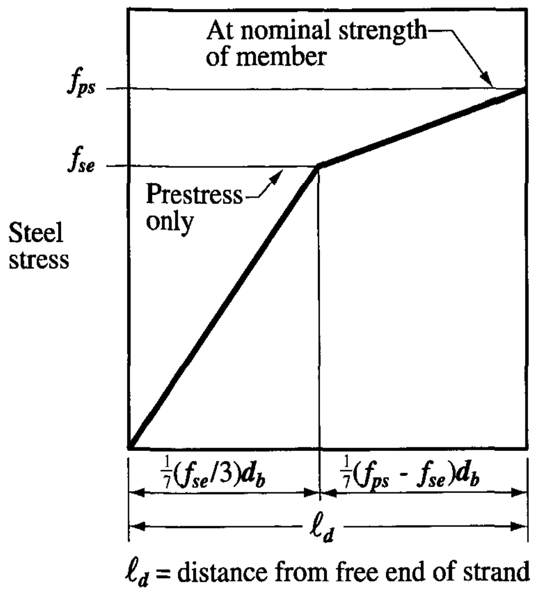


Fig. R12.9 - Idealized bilinear relationship between steel stress and distance from the free end of strand.

additional bonded length necessary to develop an increase in stress of $(f_{ps} - f_{se})$, are based on tests of members prestressed with clean, 6, 9, and 12 mm diameter strands for which the maximum value of f_{ps} was 2600 MPa. See References 12.16, 12.17, and 12.18.

R12.9.2 Where bonding of one or more strands does not extend to the end of the member, critical sections may be at locations other than where full design strength is required to be developed, and detailed analysis may be required. References 12.14

and 12.15 show a method that may be used in the case of strands with different points of full development. Conservatively, only the strands that are fully developed at a section may be considered effective at that section. If critical sections occur in the transfer region, special considerations may be necessary. Some loading conditions, such as where heavy concentrated loads occur within the strand development length, may cause critical sections to occur away from the section that is required to develop full design strength.

R12.9.3 Exploratory tests (Reference: 12.16) that study the effect of debonded strand (bond not permitted to extend to the ends of members) on performance of pretensioned girders indicated that the performance of these girders with embedment lengths twice those required by 12.9.1 closely matched the flexural performance of similar pretensioned girders with strand fully bonded to ends of girders. Accordingly, doubled development length is required for strand not bonded through to the end of a member. Subsequent tests[12.19] indicated that in pretensioned members designed for zero tension in the concrete under service load conditions (see 18.4.2), the development length for debonded strands need not be doubled. For analysis of sections with debonded strands at locations where strand is not fully developed, it is usually assumed that both the transfer length and development length are doubled.

SECTION R12.10
DEVELOPMENT OF FLEXURAL REINFORCEMENT – GENERAL

R12.10.2 Critical sections for a typical continuous beam are indicated with a "c" or an "x" in Fig. R12.10.2. For uniform loading, the positive reinforcement extending into the support is more apt to be governed by the requirements of 12.11.3 rather than by development length measured from a point of maximum moment or bar cutoff.

R12.10.3 The moment diagrams customarily used in design are approximate; some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, lateral loads, or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location of the calculated tensile stress approximately a distance d towards a point of zero moment. When stirrups are provided, this effect is less severe, although still present to some extent.
To provide for shifts in the location of maximum moments, the SBC 304 requires the extension of reinforcement a distance d or $12d_b$ beyond the point at which it is theoretically no longer required to resist flexure, except as noted.

Cutoff points of bars to meet this requirement are illustrated in Fig. R12.10.2.
When bars of different sizes are used, cutoff points of bars to meet this requirement are illustrated in Fig. R12.10.2. The extension should be in accordance with the diameter of bar being terminated. A bar bent to the far face of a beam and continued there may logically be considered effective, in satisfying this section, to the point where the bar crosses the mid-depth of the member.

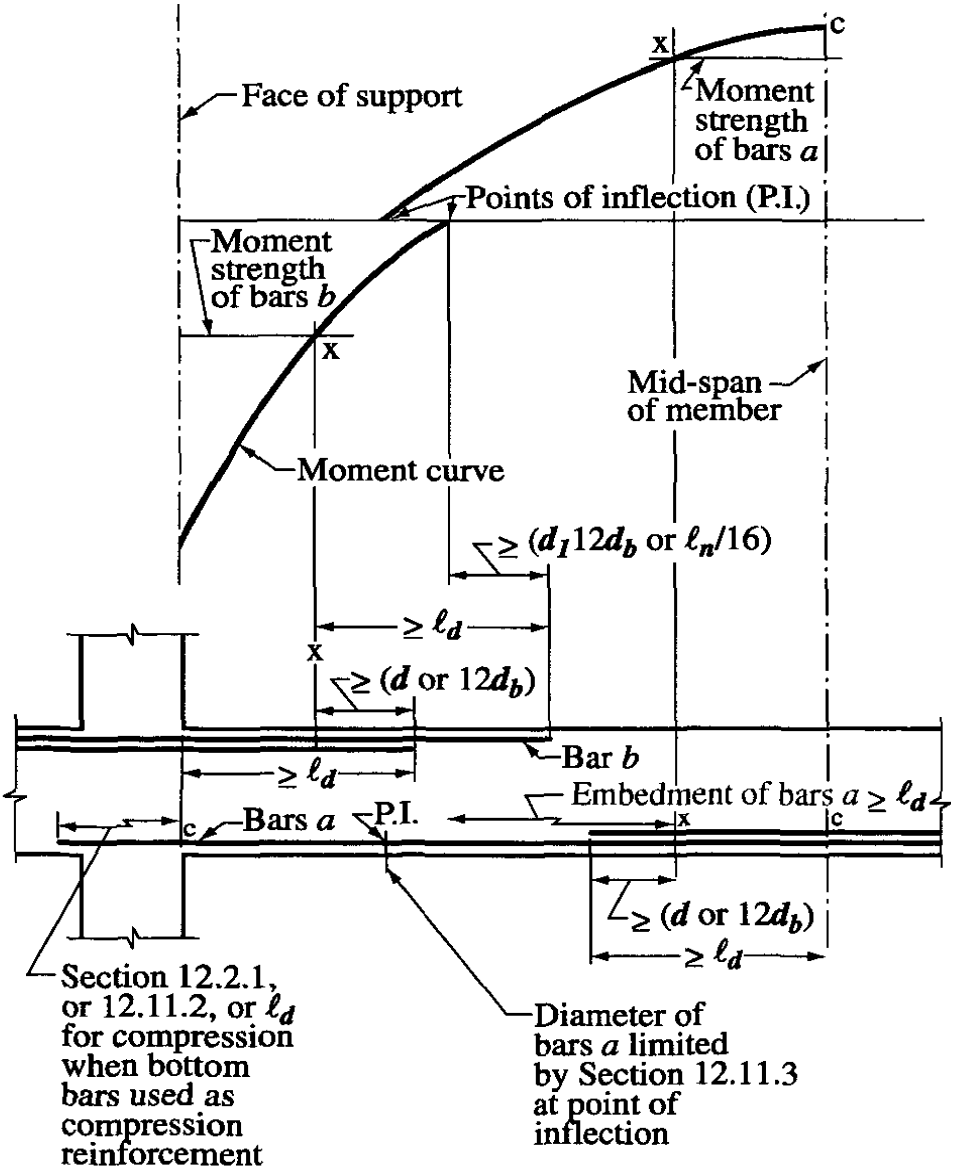


Fig. R12.10.2 - Development of flexural reinforcement in a typical continuous beam

R12.10.4 Peak stresses exist in the remaining bars wherever adjacent bars are cutoff, or bent, in tension regions. In Fig. R12.10.2 an "x" is used to indicate the peak stress points remaining in continuing bars after part of the bars have been cutoff. If bars are cutoff as short as the moment diagrams allow, these peak stresses become the full f_y, which requires a full ℓ_d extension as indicated. This extension may exceed the length required for flexure.

R12.10.5 Reduced shear strength and loss of ductility when bars are cutoff in a tension zone, as in Fig. R12.10.2, have been reported. The SBC 304 does not permit flexural reinforcement to be terminated in a tension zone unless special conditions are satisfied. Flexure cracks tend to open early wherever any reinforcement is terminated in a tension zone. If the steel stress in the continuing reinforcement and the shear strength are each near their limiting values, diagonal tension cracking tends to develop prematurely from these flexure cracks. Diagonal cracks are less likely to form where shear stress is low (see 12.10.5.1). Diagonal cracks can be restrained by closely spaced stirrups (see 12.10.5.2). A lower steel stress reduces the probability of such diagonal cracking (see 12.10.5.3). These requirements are not intended to apply to tension splices which are covered by 12.2, 12.13.5, and the related 12.15.

R12.10.6 Brackets, members of variable depth, and other members where steel stress f_s does not decrease linearly in proportion to a decreasing moment require special consideration for proper development of the flexural reinforcement. For the bracket shown in Fig. R12.10.6, the stress at ultimate in the reinforcement is almost constant at approximately f_y from the face of support to the load point. In such a case, development of the flexural reinforcement depends largely on the end anchorage provided at the loaded end. Reference 12.20 suggests a welded cross bar of equal diameter as a means of providing effective end anchorage. An end hook in the vertical plane, with the minimum diameter bend, is not totally effective because an essentially plain concrete corner will exist near loads applied close to the corner. For wide brackets (perpendicular to the plane of the figure) and loads not applied close to the corners, U-shaped bars in a horizontal plane provide effective end hooks.

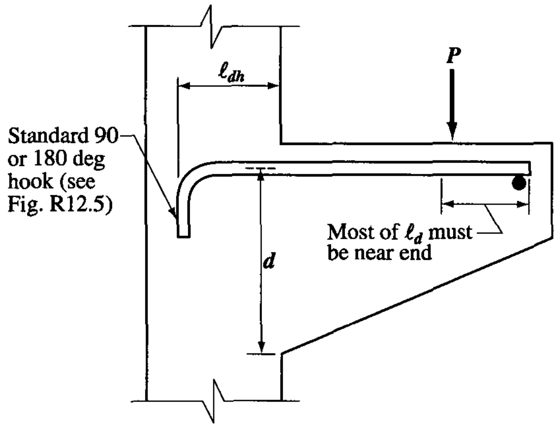


Fig. R12.10.6 - Special member largely dependent on end anchorage

SECTION R12.11 DEVELOPMENT OF POSITIVE MOMENT REINFORCEMENT

R12.11.1 Positive moment reinforcement is carried into the support to provide for some shifting of the moments due to changes in loading, settlement of supports, and lateral loads.

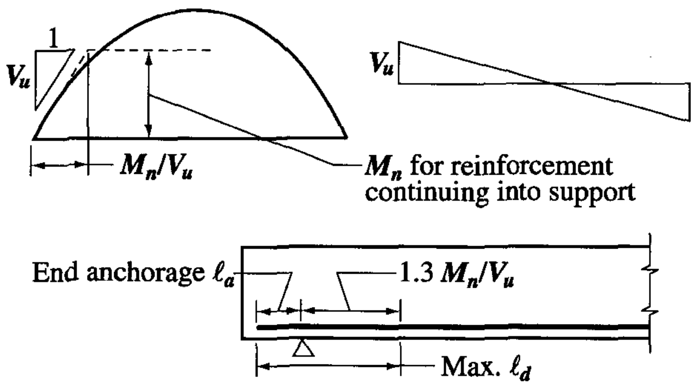
R12.11.2 When a flexural member is part of a primary lateral load resisting system, loads greater than those anticipated in design may cause reversal of moment at supports; some positive reinforcement should be well anchored into the support. This anchorage is required to ensure ductility of response in the event of serious overstress, such as from blast or earthquake. It is not sufficient to use more reinforcement at lower stresses.

R12.11.3 At simple supports and points of inflection such as "P.I." in Fig. R12.10.2, the diameter of the positive reinforcement should be small enough so that computed development length of the bar ℓ_d does not exceed $M_n / V_u + \ell_a$, or under favorable support conditions, $1.3M_n/V_u+\ell_a$. Fig.R12.11.3(a) illustrates the use of the provision.

At the point of inflection the value of ℓ_a should not exceed the actual bar extension used beyond the point of zero moment. The M_n/V_u, portion of the available length is a theoretical quantity not generally associated with an obvious maximum stress point. M_n is the nominal strength of the cross section without the ϕ-factor and is not the applied factored moment.

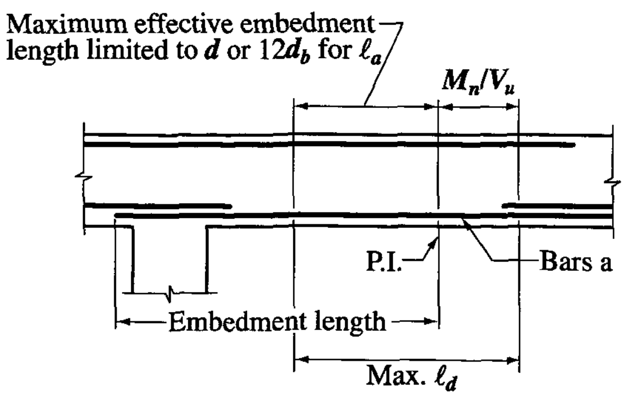
The ℓ_a to be used at points of inflection is limited to the effective depth of the member d or 12 bar diameters $(12d_b)$, whichever is greater. Fig. R12.11.3(b) illustrates this provision at points of inflection. The ℓ_a limitation is added since test data are not available to show that a long end anchorage length will be fully effective in developing a bar that has only a short length between a point of inflection and a point of maximum stress.

R12.11.4 The use of the strut and tie model for the design of reinforced concrete deep flexural members clarifies that there is significant tension in the reinforcement at the face of the support. This requires the tension reinforcement to be continuous or be developed through and beyond the support.[12.21]



Note: The 1.3 factor is usable only if the reaction confines the ends of the reinforcement.

(a) Maximum size of bar at simple support



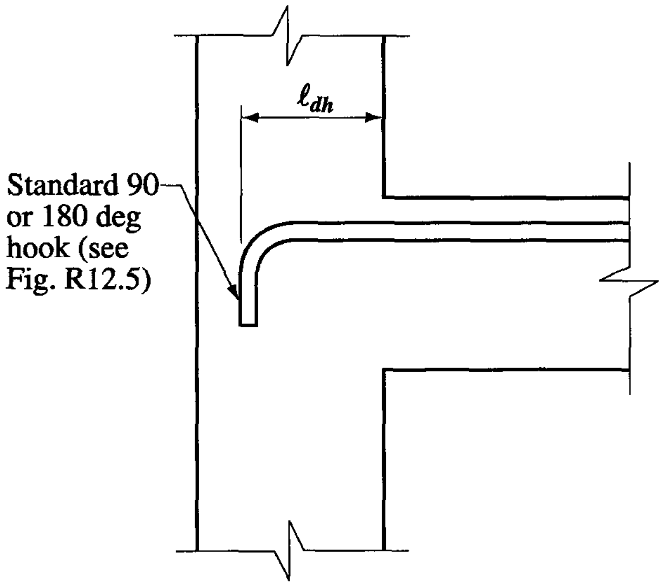
(b) Maximum size of bar "a" at point of inflection

Fig. R12.11.3 - Concept for determining maximum bar size per 12.11.3

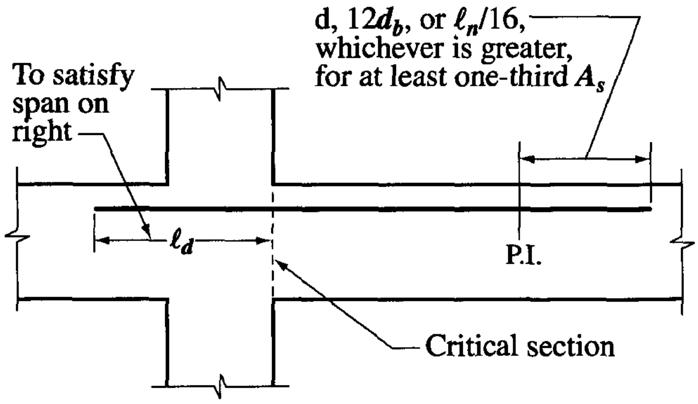
SECTION R12.12
DEVELOPMENT OF NEGATIVE MOMENT REINFORCEMENT

Fig. R12.12 illustrates two methods of satisfying requirements for anchorage of tension reinforcement beyond the face of support. For anchorage of reinforcement with hooks, see R12.5.

Section 12.12.3 provides for possible shifting of the moment diagram at a point of inflection, as discussed under R12.10.3. This requirement may exceed that of 12.10.3, and the more restrictive of the two provisions governs.



(a) Anchorage into exterior column



Note: Usually such anchorage becomes part of the adjacent beam reinforcement.

(b) Anchorage into adjacent beam

Fig. R12.12 - Development of negative moment reinforcement

SECTION R12.13
DEVELOPMENT OF WEB REINFORCEMENT

R12.13.1 Stirrups should be carried as close to the compression face of the member as possible because near ultimate load the flexural tension cracks penetrate deeply.

R12.13.2.1 For a Dia 16 mm bar or smaller bar, anchorage is provided by a standard stirrup hook, as defined in 7.1.3.

R12.13.2.2 Since it is not possible to bend a Dia 20, Dia 22, or Dia 25 mm stirrup tightly around a longitudinal bar and due to the force in a bar with a design stress greater than 300 MPa, stirrup anchorage depends on both the value of the hook and whatever development length is provided. A longitudinal bar within a stirrup hook limits the width of any flexural cracks, even in a tensile zone. Since such a stirrup hook cannot fail by splitting parallel to the plane of the hooked bar, the hook strength as utilized in 12.5.2 has been adjusted to reflect cover and confinement around the stirrup hook.

For stirrups with f_y of only 300 MPa, a standard stirrup hook provides sufficient anchorage and these bars are covered in 12.13.2.1. For bars with higher strength, the embedment should be checked. A 135 deg or 180 deg hook is preferred, but a 90 deg hook may be used provided the free end of the 90 deg hook is extended the full 12 bar diameters as required in 7.1.3.

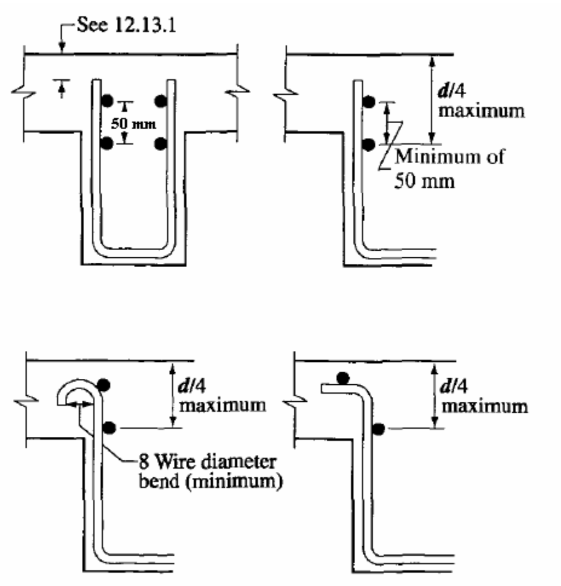


Fig. R12.13.2.3 - Anchorage in compression zone of welded plain wire fabric-U-stirrups

R12.13.2.3 The requirements for anchorage of welded plain wire fabric stirrups are illustrated in Fig. R12.13.2.3.

R12.13.2.4 Use of welded wire fabric for shear reinforcement has become commonplace in the precast, prestressed concrete industry. The rationale for acceptance of straight sheets of wire fabric as shear reinforcement is based on the findings reported in Reference 12.22.

The provisions for anchorage of single leg welded wire fabric in the tension face emphasize the location of the longitudinal wire at the same depth as the primary flexural reinforcement to avoid a splitting problem at the tension steel level. Fig. R12.13.2.4 illustrates the anchorage requirements for single leg, welded wire fabric. For anchorage of single leg, welded wire fabric, the SBC 304 has permitted hooks and embedment length in the compression and tension faces of members (see 12.13.2.1 and 12.13.2.3), and embedment only in the compression face (see 12.13.2.2). Section 12.13.2.4 provides for anchorage of straight, single leg, welded wire fabric using longitudinal wire anchorage with adequate embedment length in compression and tension faces of members.

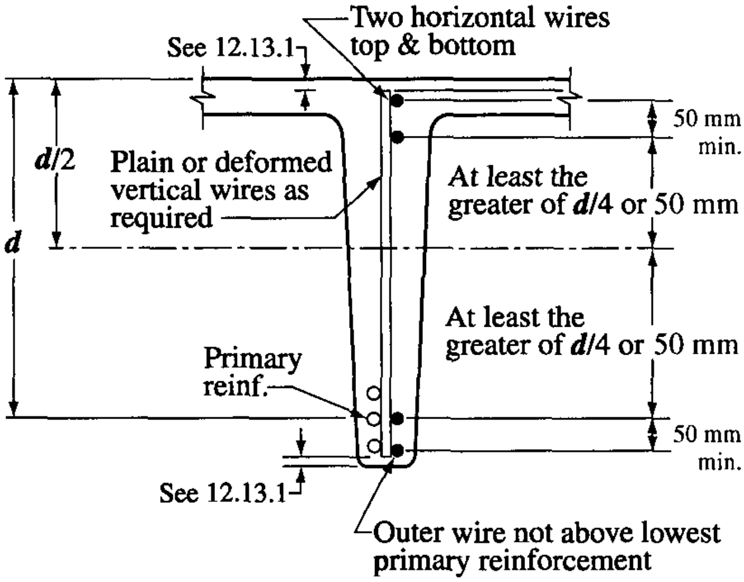


Fig. R12.13.2.4 - Anchorage of single leg welded wire fabric shear reinforcement

R12.13.2.5 In joists, a small bar or wire can be anchored by a standard hook not engaging longitudinal reinforcement, allowing a continuously bent bar to form a series of single-leg stirrups in the joist.

R12.13.5 These requirements for lapping of double U-stirrups to form closed stirrups control over the provisions of 12.15.

SECTION R12.14
SPLICES OF REINFORCEMENT - GENERAL

Splices should, if possible, be located away from points of maximum tensile stress. The lap splice requirements of 12.15 encourage this practice.

R12.14.2 **Lap splices**

R12.14.2.1 Because of lack of adequate experimental data on lap splices of Dia 40 and Dia 56 mm bars in compression and in tension, lap splicing of these bar sizes is prohibited except as permitted in 12.16.2 and 15.8.2.3 for compression lap.

R12.14.2.2 The increased length of lap required for bars in bundles is based on the reduction in the exposed perimeter of the bars. Only individual bars are lap spliced along the bundle.

R12.14.2.3 If individual bars in noncontact lap splices are too widely spaced, an unreinforced section is created. Forcing a potential crack to follow a zigzag line (5 to 1 slope) is considered a minimum precaution. The 150 mm maximum spacing is added because most research available on the lap splicing of deformed bars was conducted with reinforcement within this spacing.

R12.14.3 Mechanical and welded splices

R12.14.3.2 The maximum reinforcement stress used in design under the SBC 304 is the specified yield strength. To ensure sufficient strength in splices so that yielding can be achieved in a member and thus brittle failure avoided, the 25 percent increase above the specified yield strength was selected as both an adequate minimum for safety and a practicable maximum for economy.

R12.14.3.3 **A** full welded splice is primarily intended for large bars (Dia 20 mm and larger) in main members. The tensile strength requirement of 125 percent of specified yield strength is intended to provide sound welding that is also adequate for compression. See the discussion on strength in R12.14.3.2.

R12.14.3.4 The use of mechanical or welded splices of less strength than 125 percent of specified yield strength is permitted if the minimum design criteria of 12.15.4 are met. Therefore, lap welds of reinforcing bars, either with or without backup material, welds to plate connections, and end-bearing splices are allowed under certain conditions.

SECTION R12.15
SPLICES OF DEFORMED BARS AND DEFORMED WIRE IN TENSION

R12.15.1 Lap splices in tension are classified as Type A or B, with length of lap a multiple of the tensile development length ℓ_d. The development length ℓ_d used to obtain lap length should be based on f_y because the splice classifications already reflect any excess reinforcement at the splice location; therefore, the factor from 12.2.5 for excess A_s should not be used. When multiple bars located in the same plane are spliced at the same section, the clear spacing is the minimum clear distance between the adjacent splices. For splices in columns with offset bars, Fig. R12.15.1(a) illustrates the clear spacing to be used. For staggered splices, the clear spacing is the minimum distance between adjacent splices [distance x in Fig. R12.15.1(b)].

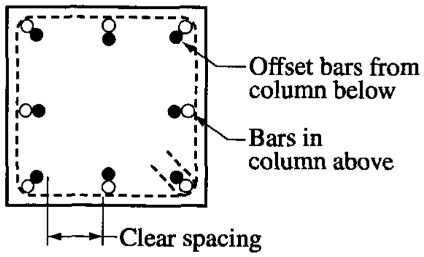
R12.15.2 The tension lap splice requirements of 12.15.1 encourage the location of splices away from regions of high tensile stress to locations where the area of steel provided is at least twice that required by analysis. Table R12.15.2 presents the splice requirements in tabular form.

TABLE R12.15.2-TENSION LAP SPLICES

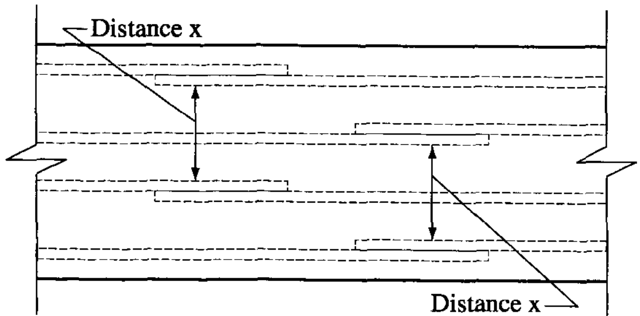
$\frac{A_{s,provided*}}{A_{s,required}}$	Maximum percent of A_s spliced within required lap length	
	50	100
Equal to or greater than 2	Class A	Class B
Less than 2	Class B	Class B

* Ratio of area reinforcement provided to area of reinforcement required by analysis at splice locations.

R12.15.3 A mechanical or welded splice should develop at least 125 percent of the specified yield strength when located in regions of high tensile stress in the reinforcement. Such splices need not be staggered, although such staggering is encouraged where the area of reinforcement provided is less than twice that required by the analysis.



(a) Offset column bars



(b) Staggered splices

Fig. R12.15.1 - Clear spacing of spliced bars

R12.15.4 See R12.14.3.5. Section 12.15.4 concerns the situation where mechanical or welded splices of strength less than 125 percent of the specified yield strength of the reinforcement may be used. It provides a relaxation in the splice requirements where the splices are staggered and excess reinforcement area is available. The criterion of twice the computed tensile force is used to cover sections containing partial tensile splices with various percentages of total continuous steel. The usual partial tensile splice is a flare groove weld between bars or bar and structural steel piece.

To detail such welding, the length of weld should be specified. Such welds are rated at the product of total weld length times effective size of groove weld (established by bar size) times allowable stress permitted by "Structural Welding SBC 304-Reinforcing Steel" (ANSI/AWS D1.4).

A full mechanical or welded splice conforming to 12.14.3.2 or 12.14.3.4 can be used without the stagger requirement in lieu of the lower strength mechanical or welded splice.

R12.15.5 A tension tie member, has the following characteristics: member having an axial tensile force sufficient to create tension over the cross section; a level of stress in the reinforcement such that every bar must be fully effective; and limited concrete cover on all sides. Examples of members that may be classified as tension ties are arch ties, hangers carrying load to an overhead supporting structure, and main tension elements in a truss.

In determining if a member should be classified as a tension tie, consideration should be given to the importance, function, proportions, and stress conditions of the member related to the above characteristics. For example, a usual large circular tank, with many bars and with splices well staggered and widely spaced should not be classified as a tension tie member, and Class B splices may be used.

SECTION R12.16
SPLICES OF DEFORMED BARS IN COMPRESSION

Bond research has been primarily related to bars in tension. Bond behavior of compression bars is not complicated by the problem of transverse tension cracking and thus compression splices do not require provisions as strict as those specified for tension splices.

R12.16.1 Tests (References: 12.20, 12.23) have shown that splice strengths in compression depend considerably on end bearing and do not increase proportionally in strength when the splice length is doubled. Accordingly, for yield strengths above 420 MPa, compression lap lengths are significantly increased, except where spiral enclosures are used (as in spiral columns) where increase is about 10 percent for an increase in yield strength from 420 to 520 MPa.

R12.16.2 The lap splice length is to be computed based on the larger of the compression splice length of the smaller bar; or the compression development length of the larger bar. Lap splices are generally prohibited for Dia 40 mm and larger bars; however, for compression only, lap splices are permitted for Dia 40 mm and larger bars to Dia 36 mm or smaller bars.

R12.16.4 **End-bearing splices**

R12.16.4.1 Experience with end-bearing splices has been almost exclusively with vertical bars in columns. If bars are significantly inclined from the vertical, special attention is required to ensure that adequate end-bearing contact can be achieved and maintained.

SECTION R12.17
SPECIAL SPLICE REQUIREMENTS FOR COLUMNS

In columns subject to flexure and axial loads, tension stresses may occur on one face of the column for moderate and large eccentricities as shown in Fig. R12.17. When such tensions occur, 12.17 requires tension splices to be used or an adequate tensile resistance to be provided. Furthermore, a minimum tension capacity is required in each face of all columns even where analysis indicates compression only.

The column splice should satisfy requirements for all load combinations for the column. Frequently, the basic gravity load combination will govern the design of the column itself, but a load combination including wind or seismic loads may induce greater tension in some column bars, and the column splice should be designed for this tension.

R12.17.2 Lap splices in columns

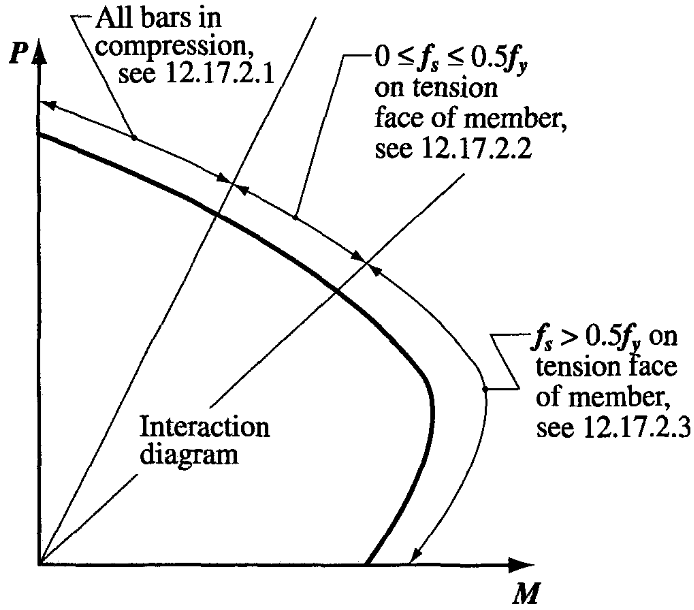


Fig. R12.17 - Special splice requirements for columns

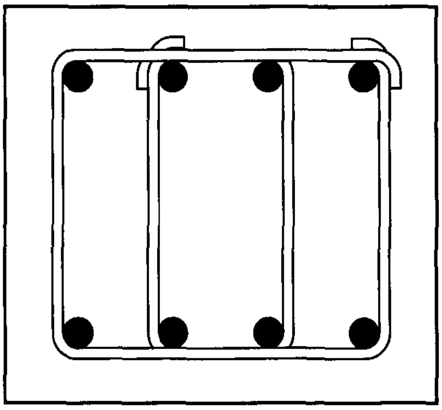


Fig. R.12.17.2 - Tie legs which cross the axis of bending are used to compute effective area. In the case shown, four legs are effective

R12.17.2.4 Reduced lap lengths are allowed when the splice is enclosed throughout its length by minimum ties.

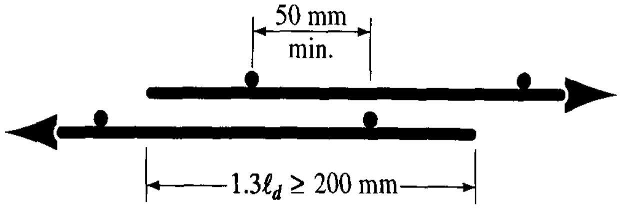
The tie legs perpendicular to each direction are computed separately and the requirement must be satisfied in each direction. This is illustrated in Fig. R12.17.2, where four legs are effective in one direction and two legs in the other direction. This calculation is critical in one direction, which normally can be determined by inspection.

R12.17.2.5 Compression lap lengths may be reduced when the lap splice is enclosed throughout its length by spirals because of increased splitting resistance. Spirals should meet requirements of 7.10.4 and 10.9.3.

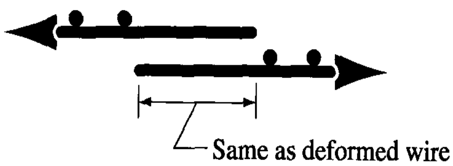
R12.17.3 **Mechanical or welded splices in columns.** Mechanical or welded splices are allowed for splices in columns but should be designed as a full mechanical splice or a full welded splice developing 125 percent f_y as required by 12.14.3.2 or

12.14.3.4. Splice capacity is traditionally tested in tension and full strength is required to reflect the high compression loads possible in column reinforcement due to creep effects. If a mechanical splice developing less than a full mechanical splice is used, then the splice is required to conform to all requirements of end-bearing splices of 12.16.4 and 12.17.4.

R12.17.4 **End-bearing splices in columns.** End-bearing splices used to splice column bars always in compression should have a tension capacity of 25 percent of the yield strength of the steel area on each face of the column, either by staggering the end-bearing splices or by adding additional steel through the splice location. The end-bearing splice should conform to 12.16.4.



(a) Section 12.18.1



(b) Section 12.18.2

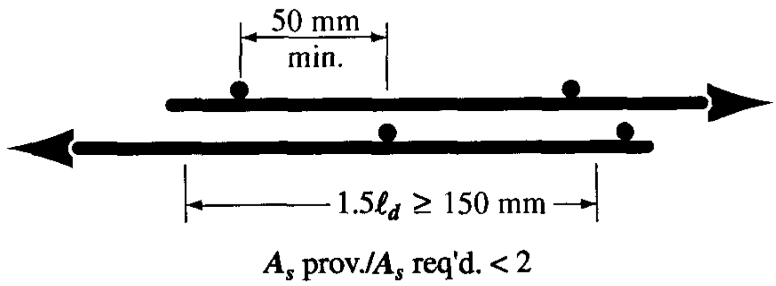
Fig. R12.18 - Lap splices of deformed fabric

SECTION R12.18
SPLICES OF WELDED DEFORMED WIRE FABRIC IN TENSION

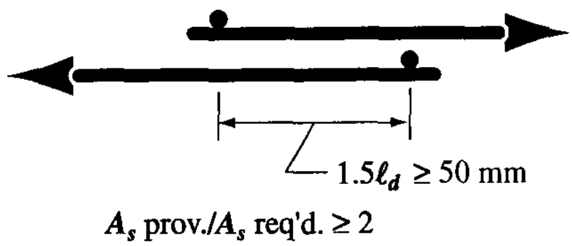
Splice provisions for deformed fabric are based on available tests.[12.24] The development length ℓ_d is that computed in accordance with the provisions of 12.7 without regard to the 200 mm minimum. The 200 mm applies to the overall splice length. See Fig. R12.18. If no cross wires are within the lap length, the provisions for deformed wire apply.

SECTION R12.19
SPLICES OF WELDED PLAIN WIRE FABRIC IN TENSION

The strength of lap splices of welded plain wire fabric is dependent primarily on the anchorage obtained from the cross wires rather than on the length of wire in the splice. For this reason, the lap is specified in terms of overlap of cross wires rather than in wire diameters or millimeters. The 50 mm additional lap required is to assure overlapping of the cross wires and to provide space for satisfactory consolidation of the concrete between cross wires. Research[12.25] has shown an increased splice length is required when fabric of large, closely spaced wires is lapped and as a consequence additional splice length requirements are provided for these fabrics, in addition to an absolute minimum of 150 mm. The development length ℓ_d is that computed in accordance with the provisions of 12.8 without regard to the 150 mm minimum. Splice requirements are illustrated in Fig. R12.19.



(a) Section 12.19.1



(b) Section 12.19.2

Fig. R12.19 - Lap splices of plain fabric

CHAPTER 13
TWO-WAY SLAB SYSTEM

SECTION R13.0
NOTATION

The design methods given in Chapter 13 are based on analysis of the results of an extensive series of test [13.1-13.7] and the well established performance record of various slab systems. Much of Chapter 13 is concerned with the selection and distribution of flexural reinforcement. The designer is cautioned that the problem related to safety of a slab system is the transmission of load from the slab to the columns by flexure, torsion, and shear. Design criteria for shear and torsion in slabs are given in Chapter 11.

Design aids for use in the engineering analysis and design of two-way slab systems are given in the ACI Design Handbook[13.8]. Design aids are provided to simplify application of the direct design and equivalent frame methods of Chapter 13.

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

SECTION R13.1
SCOPE

The fundamental design principles contained in Chapter 13 are applicable to all planar structural systems subjected to transverse loads. Some of the specific design rules, as well as historical precedents, limit the types of structures to which Chapter 13 applies. General characteristics of slab systems that may be designed according to Chapter 13 are described in this section. These systems include flat slabs, flat plates, two-way slabs, and waffle slabs. Slabs with paneled ceilings are two-way wide-band beam systems.

True one-way slabs, slabs reinforced to resist flexural stresses in only one direction, are excluded. Also excluded are soil-supported slabs, such as slabs on grade that do not transmit vertical loads from other parts of the structure to the soil.

For slabs with beams, the explicit design procedures of Chapter 13 apply only when the beams are located at the edges of the panel and when the beams are supported by columns or other essentially nondeflecting supports at the comers of the panel. Two-way slabs with beams in one direction, with both slab and beams supported by girders in the other direction, may be designed under the general requirements of Chapter 13. Such designs should be based upon analysis compatible with the deflected position of the supporting beams and girders.

For slabs supported on walls, the explicit design procedures in this chapter treat the wall as a beam of infinite stiffness; therefore, each wall should support the entire length of an edge of the panel (see 13.2.3). Wall-like columns less than a full panel length can be treated as columns.

SECTION R13.2
DEFINITION

R13.2.3 **A** panel includes all flexural elements between column centerlines. Thus, the column strip includes the beam, if any.

R13.2.4 For monolithic or fully composite construction, the beams include portions of the slab as flanges. Two examples of the rule are provided in Fig. R13.2.4.

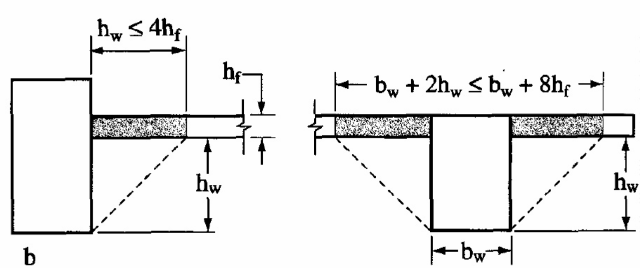


Fig. R13.2.4 - Examples of the portion of slab to be included with the beam under 13.2.4

SECTION R13.3
SLAB REINFORCEMENT

R13.3.2 The requirement that the center-to-center spacing of the reinforcement be not more than two times the slab thickness applies only to the reinforcement in solid slabs, and not to reinforcement joists or waffle slabs.

This limitation is to ensure slab action, cracking, and provide for the possibility of loads concentrated on small areas of the slab. See also R10.6.

R13.3.3- R13.3.5 Bending moments in slabs at spandrel beams can be subject to great variation. If spandrel beams are built solidly into walls, the slab approaches complete fixity. Without an integral wall, the slab could approach simply supported, depending on the torsional rigidity of the spandrel beam or slab edge. These requirements provide for unknown conditions that might normally occur in a structure.

R13.3.8 **Details of reinforcement in slabs without beams**

R13.3.8.4 Bent bars are not recommended because they are difficult to place properly, however, they are permitted if they comply with 13.3.8.3.

For moments resulting from combined lateral and gravity loadings, the minimum lengths and extensions of bars in Fig. 13.3.8 may not be sufficient.

R13.3.8.5 The continuous column strip bottom reinforcement provides the slab some residual ability to span to the adjacent supports should a single support be damaged. The two continuous column strip bottom bars or wires through the column may be termed integrity steel, and are provided to give the slab some residual capacity following a single punching shear failure at a single support.[13.9]

R13.3.8.6 This provision is used to require the same integrity steel as for other two-way slabs without beams in case of a punching shear failure at a support.

In some instances, there is sufficient clearance so that the bonded bottom bars can pass under shearheads and through the column. Where clearance under the shearhead is inadequate, the bottom bars should pass through holes in the shearhead arms or within the perimeter of the lifting collar. Shearheads should be kept as low as possible in the slab to increase their effectiveness.

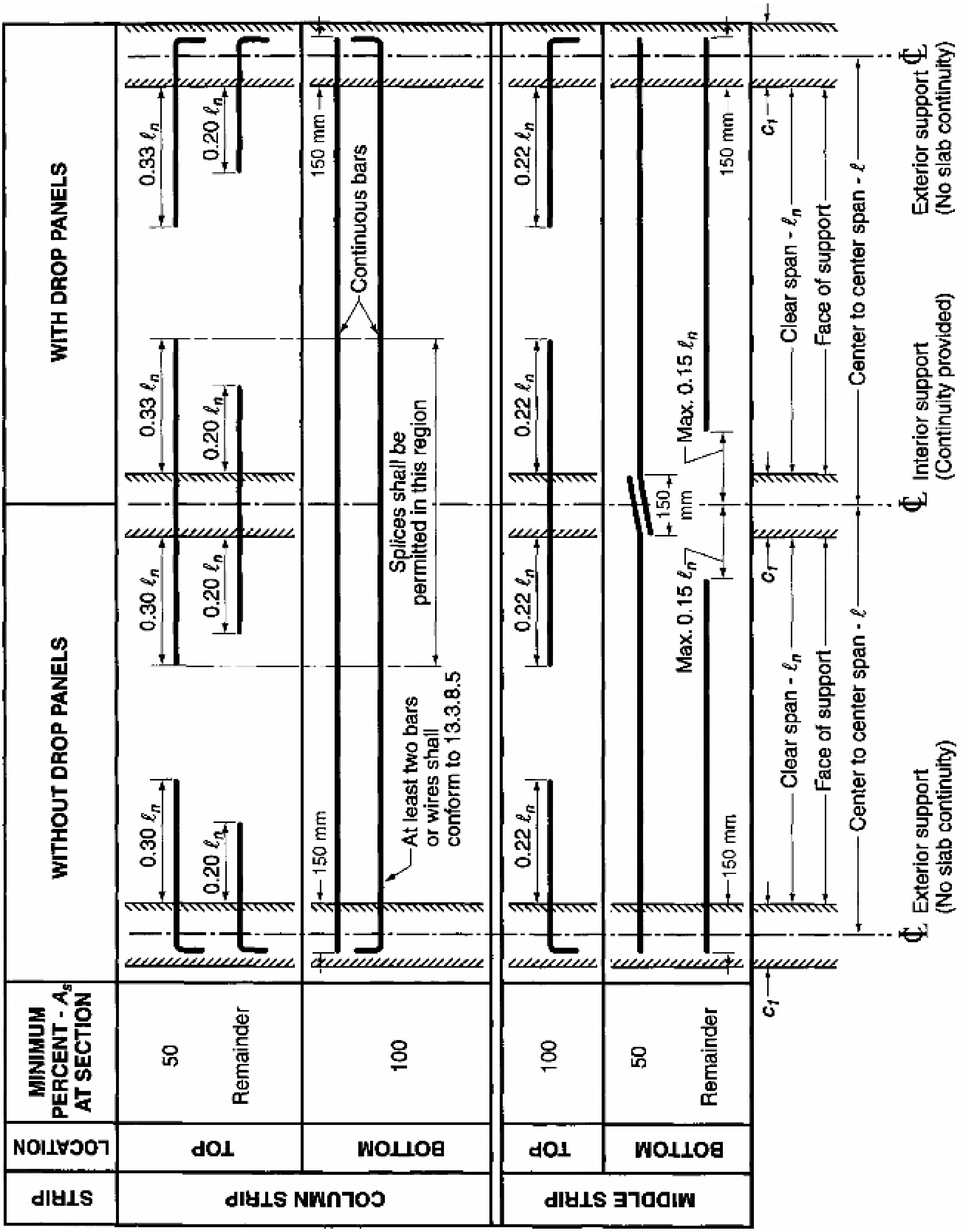


Fig. 13.3.8 - Minimum extensions for reinforcement in slabs without beams. (See 12.11.1 for reinforcement extension into supports)

SECTION R13.4
OPENINGS IN SLAB SYSTEMS

See R11.12.5.

SECTION R13.5
DESIGN PROCEDURES

R13.5.1 This section permits a designer to base a design directly on fundamental principles of structural mechanics, provided it can be demonstrated explicitly that all safety and serviceability criteria are satisfied. The design of the slab may be achieved through the combined use of classic solutions based on a linearly elastic continuum, numerical solutions based on discrete elements, or yield-line analyses, including, in all cases, evaluation of the stress conditions around the supports in relation to shear and torsion as well as flexure. The designer should consider that the design of a slab system involves more than its analysis, and justify any deviations in physical dimensions of the slab from common practice on the basis of knowledge of the expected loads and the reliability of the calculated stresses and deformations of the structure.

R13.5.1.1 For gravity load analysis of two-way slab systems, two analysis methods are given in 13.6 and 13.7. The specific provisions of both design methods are limited in application to orthogonal frames subject to gravity loads only. Both methods apply to two-way slabs with beams as well as to flat slabs and flat plates. In both methods, the distribution of moments to the critical sections of the slab reflects the effects of reduced stiffness of elements due to cracking and support geometry.

R13.5.1.2 During the life of a structure, construction loads, ordinary occupancy loads, anticipated overloads, and volume changes will cause cracking of slabs. Cracking reduces stiffness of slab members, and increases lateral flexibility when lateral loads act on the structure. Cracking of slabs should be considered in stiffness assumptions so that drift caused by wind or earthquake is not grossly underestimated.

The designer may model the structure for lateral load analysis using any approach that is shown to satisfy equilibrium and geometric compatibility and to be in reasonable agreement with test data.[13.10,13.11] The selected approach should recognize effects of cracking as well as parameters such as ℓ_2/ℓ_1, c_1/ℓ_1, and c_2/c_1. Some of the available approaches are summarized in Reference 13.12, which includes a discussion on the effects of cracking. Acceptable approaches include plate-bending finite-element models, the effective beam width model, and the equivalent frame model. In all cases, framing member stiffnesses should be reduced to account for cracking.

For nonprestressed slabs, it is normally appropriate to reduce slab bending stiffness to between one-half and one quarter of the uncracked stiffness. For prestressed construction, stiffnesses greater than those of cracked, nonprestressed slabs may be appropriate. When the analysis is used to determine design drifts or moment magnification, lower-bound slab stiffnesses should be assumed. When the analysis is used to study interactions of the slab with other framing elements, such as structural walls, it may be appropriate to consider a range of slab stiffnesses so that the relative importance of the slab on those interactions can be assessed.

R13.5.3 This section is concerned primarily with slab systems without beams. Tests and experience have shown that, unless special measures are taken to resist the torsional and shear stresses, all reinforcement resisting that part of the moment to be transferred to the column by flexure should be placed between lines that are one and one-half the slab or drop panel thickness, $1.5h$, on each side of the column. The calculated shear stresses in the slab around the column are required to conform to the requirements of 11.12.2. See R11.12.1.2 and R11.12.2.1 for more details on application of this section.

R13.5.3.3 Under certain conditions the designer is permitted to adjust the level of moment transferred by shear without revising member sizes. Tests indicate that some flexibility in distribution of unbalanced moments transferred by shear and flexure at both exterior and interior supports is possible. Interior, exterior, and corner supports refer to slab-column connections for which the critical perimeter for rectangular columns has 4, 3, or 2 sides, respectively.

At exterior supports, for unbalanced moments about an axis parallel to the edge, the portion of moment transferred by eccentricity of shear $\gamma_v M_u$ may be reduced provided that the factored shear at the support (excluding the shear produced by moment transfer) does not exceed 75 percent of the shear capacity ϕV_c as defined in 11.12.2.1 for edge columns or 50 percent for comer columns. Test[13.14,13.15] indicate that there is no significant interaction between shear and unbalanced moment at the exterior support in such cases. Note that as $\gamma_v M_u$ is decreased $\gamma_f M_u$ is increased.

Tests of interior supports indicate that some flexibility in distributing unbalanced moments transferred by shear and flexure is possible, but with more severe limitations than for exterior supports. For interior supports, the unbalanced moment transferred by flexure is permitted to be increased up to 25 percent provided that the factored shear (excluding the shear caused by the moment transfer) at the interior supports does not exceed 40 percent of the shear capacity ϕV_c as defined in 11.12.2.1.

Tests of slab-column connections indicate that a large degree of ductility is required because the interaction between shear and unbalanced moment is critical. When the factored shear is large, the column-slab joint cannot always develop all of the reinforcement provided in the effective width. The modifications for edge, corner, or interior slab-column connections in 13.5.3.3 are permitted only when the reinforcement ratio (within the effective width) required to develop the unbalanced moment $\gamma_f M_u$ does not exceed $0.375\rho_b$. The use of Eq. (13-1) without the modification permitted in 13.5.3.3 will generally indicate overstress conditions on the joint. The provisions of 13.5.3.3 are intended to improve ductile behavior of the column-slab joint. When a reversal of moments occurs at opposite faces of an interior support, both top and bottom reinforcement should be concentrated within the effective width. A ratio of top to bottom reinforcement of about 2 has been observed to be appropriate.

SECTION R13.6
DIRECT DESIGN METHOD

The direct design method consists of a set of rules for distributing moments to slab and beam sections to satisfy safety requirements and most serviceability requirements simultaneously. Three fundamental steps are involved as follows:

(1) Determination of the total factored static moment (see 13.6.2);

(2) Distribution of the total factored static moment to negative and positive sections (see 13.6.3);

(3) Distribution of the negative and positive factored moments to the column and middle strips and to the beams, if any (see 13.6.4 through 13.6.6). The distribution of moments to column and middle strips is also used in the equivalent frame method (see 13.7).

R13.6.1 **Limitations**

The direct design method was developed from consider- ations of theoretical procedures for the determination of moments in slabs with and without beams, requirements for simple design and construction procedures, and precedents supplied by performance of slab systems. Consequently, the slab systems to be designed using the direct design method should conform to the limitations in this section.

R13.6.1.1 The primary reason for the limitation in this section is the magnitude of the negative moments at the interior support in a structure with only two continuous spans. The rules given for the direct design method assume that the slab system at the first interior negative moment section is neither fixed against rotation nor discontinuous.

R13.6.1.2 If the ratio of the two spans (long spad/short span) of a panel exceeds two, the slab resists the moment in the shorter span essentially as a one-way slab.

R13.6.1.3 The limitation in this section is related to the possibility of developing negative moments beyond the point where negative moment reinforcement is terminated, as prescribed in Fig. 13.3.8.

R13.6.1.4 Columns can be offset within specified limits from a regular rectangular array. A cumulative total offset of 20 percent of the span is established as the upper limit.

R13.6.1.5 The direct design method is based on tests[13.16] for uniform gravity loads and resulting column reactions determined by statics. Lateral loads such as wind or seismic require a frame analysis. Inverted foundation mats designed as two-way slabs (see 15.10) involve application of known column loads. Therefore, even where the soil reaction is assumed to be uniform, a frame analysis should be performed.

R13.6.1.6 The elastic distribution of moments will deviate significantly from those assumed in the direct design method unless the requirements for stiffness are satisfied.

R13.6.1.7 Moment redistribution as permitted by 8.4 is not intended for use where approximate values for bending moments are used. For the direct design method, 10 percent modification is allowed by 13.6.7.

R13.6.1.8 The designer is permitted to use the direct design method even if the structure does not fit the limitations in this section, provided it can be shown by analysis that the particular limitation does not apply to that structure. For a slab system carrying a nonmovable load (such as a water reservoir in which the load on all panels is expected to be the same), the designer need not satisfy the live load limitation of 13.6.1.5.

R13.6.2 Total factored static moment for a span

R13.6.2.2 Eq. (13-3) follows directly from Nichol's derivation[13.17] with the simplifying assumption that the reactions are concentrated along the faces of the support perpendicular to the span considered. In general, the designer will find it expedient to calculate static moments for two adjacent half panels that include a column strip with a half middle strip along each side.

R13.6.2.4 The definition of ℓ_2 in this section is for computing the moment. However, ℓ_2 used in other sections shall be measured center to center of support.

R13.6.2.5 If a supporting member does not have a rectangular cross section or if the sides of the rectangle are not parallel to the spans, it is to be treated as a square support having the same area, as illustrated in Fig. R13.6.2.5.

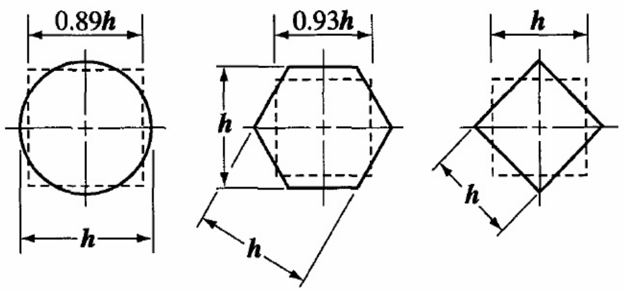


Fig. R13.6.2.5 - Examples of equivalent square section for supporting members

R13.6.3 Negative and positive factored moments

R13.6.3.3 The moment coefficients for an end span are based on the equivalent column stiffness expressions from References 13.18, 13.19, and 13.20. The coefficients for an unrestrained edge would be used, for example, if the slab were simply supported on a masonry or concrete wall.

Those for a fully restrained edge would apply if the slab were constructed integrally with a concrete wall having a flexural stiffness so large compared to that of the slab that little rotation occurs at the slab-to-wall connection.

For other than unrestrained or fully restrained edges, coefficients in the table were selected to be near the upper bound of the range for positive moments and interior negative moments. As a result, exterior negative moments were usually closer to a lower bound. The exterior negative moment capacity for most slab systems is governed by minimum reinforcement to control cracking. The final coefficients in the table have been adjusted so that the absolute sum of the positive and average moments equal M_o.

For two-way slab systems with beams between supports on all sides (two-way slabs), moment coefficients of column (2) of the table apply. For slab systems without beams between interior supports (flat plates and flat slabs), the moment coefficients of column (3) or (4) apply, without or with an edge (spandrel) beam, respectively.

R13.6.3.4 The differences in slab moment on either side of a column or other type of support should be accounted for in the design of the support. If an analysis is made to distribute unbalanced moments, flexural stiffness may be obtained on the basis of the gross concrete section of the members involved.

R13.6.3.5 Moments perpendicular to, and at the edge of, the slab structure should be transmitted to the supporting columns or walls. Torsional stresses caused by the moment assigned to the slab should be investigated.

R13.6.4, R13.6.5, and R13.6.6 Factored moments in column strips, beams, and middle strips

The rules given for assigning moments to the column strips, beams, and middle strips are based on studies[13.21] of moments in linearly elastic slabs with different beam stiffness tempered by the moment coefficients that have been used successfully.

R13.6.4.1 The negative moment factors may also be calculated as follows:

$$75 + 30 \cdot (\alpha_1 \ell_2 / \ell_1) \cdot (1 - \ell_2 / \ell_1);$$

$$\text{if } (\alpha_1 \ell_2 / \ell_1) > 1.0 \text{ use } 1.0.$$

For the purpose of establishing moments in the half column strip adjacent to an edge supported by a wall, ℓ_n in Eq. (13-3) may be assumed equal to ℓ_n of the parallel adjacent column to column span, and the wall may be considered as a beam having a moment of inertia I_b equal to infinity.

R13.6.4.2 The negative moment factors may also be calculated as follows:

$$100 - 10\beta_t + 12\beta_t (\alpha_1 \ell_2 / \ell_1) \cdot (1 - \ell_2 / \ell_1);$$

$$\text{if } \beta_t > 2.5\text{, use 2.5 if } (\alpha \ell_1 / \ell_2) > 1.0 \text{ use } 1.0.$$

The effect of the torsional stiffness parameter β_t is to assign all of the exterior negative factored moment to the column strip, and none to the middle strip, unless the beam torsional stiffness is high relative to the flexural stiffness of the supported slab. In the definition of β_t, the shear modulus has been taken as $E_{cb}/2$.

Where walls are used as supports along column lines, they can be regarded as very stiff beams with an $(\alpha_1 \ell_2 / \ell_1)$ value greater than one. Where the exterior support consists of a wall perpendicular to the direction in which moments are being determined, β_t may be taken as zero if the wall is of masonry without torsional resistance, and β_t may be taken as 2.5 for a concrete wall with great torsional resistance that is monolithic with the slab.

R13.6.4.4 The positive moment factors may also be calculated as follows:

$$60 + 30(\alpha_1 \ell_2 / \ell_1)(1.5 - \ell_2 / \ell_1);$$

$$\text{if } (\alpha_1 \ell_2 / \ell_1) > 1.0\text{, use } 1.0.$$

R13.6.5 **Factored moments in beams**

Loads assigned directly to beams are in addition to the uniform dead load of the slab; uniform superimposed dead loads, such as the ceiling, floor finish, or assumed equivalent partition loads; and uniform live loads. All of these loads are normally included with w_u in Eq. (13-3). Linear loads applied directly to beams include partition walls over or along beam center lines and additional dead load of the projecting beam stem. Concentrated loads include posts above or hangers below the beams. For the purpose of assigning directly applied loads, only loads located within the width of the beam stem should be considered as directly applied to the beams. (The effective width of a beam as defined in 13.2.4 is solely for strength and relative stiffness calculations.) Line loads and concentrated loads located on the slab away from the beam stem require special consideration to determine their apportionment to slab and beams.

R13.6.8 **Factored shear in slab systems with beams**

The tributary area for computing shear on an interior beam is shown shaded in Fig. R13.6.8. If the stiffness for the beam $(\alpha_1 \ell_2 / \ell_1)$ is less than 1.0, the shear on the beam may be obtained by linear interpolation. In such cases, the beams framing into the column will not account for all of the shear force applied on the column. The remaining shear force will produce shear stresses in the slab around the column that should be checked in the same manner as for flat slabs, as required by 13.6.8.4. Sections 13.6.8.1 through 13.6.8.3 do not apply to the calculation of torsional moments on the beams. These moments should be based on the calculated flexural moments acting on the sides of the beam.

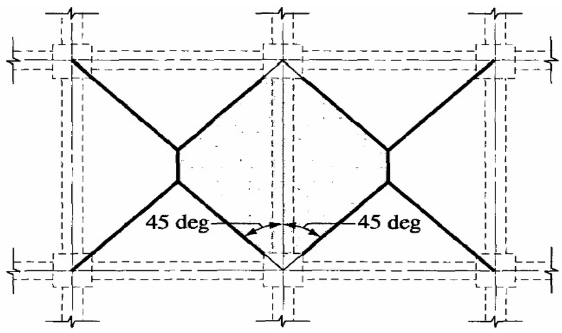


Fig. R13.6.8 - Tributary area for shear on an interior beam

R13.6.9 **Factored moments in columns and walls**

Eq. (13-4) refers to two adjoining spans, with one span longer than the other, and with full dead load plus one-half live load applied on the longer span and only dead load applied on the shorter span.

Design and detailing of the reinforcement transferring the moment from the slab to the edge column is critical to both the performance and the safety of flat slabs or flat plates without edge beams or cantilever slabs. It is important that complete design details be shown on design drawings, such as concentration of reinforcement over the column by closer spacing or additional reinforcement.

SECTION R13.7
EQUIVALENT FRAME METHOD

The equivalent frame method involves the representation of the three-dimensional slab system by a series of two-dimensional frames that are then analyzed for loads acting in the plane of the frames. The negative and positive moments so determined at the critical design sections of the frame are distributed to the slab sections in accordance with 13.6.4 (column strips), 13.6.5 (beams), and 13.6.6 (middle strips). The equivalent frame method is based on studies reported in References 13.18, 13.19, and 13.20.

R13.7.2 **Equivalent frame**

Application of the equivalent frame to a regular structure is illustrated in Fig. R13.7.2. The three-dimensional building is divided into a series of two-dimensional frame bents (equivalent frames) centered on column or support centerlines with each frame extending the full height of the building. The width of each equivalent frame is bounded by the centerlines of the adjacent panels. The complete analysis of a slab system for a building consists of analyzing a series of equivalent (interior and exterior) frames spanning longitudinally and transversely through the building.

The equivalent frame comprises three parts: (1) the horizontal slab strip, including any beams spanning in the direction of the frame, (2) the columns or other vertical supporting members, extending above and below the slab, and (3) the elements of the structure that provide moment transfer between the horizontal and vertical members.

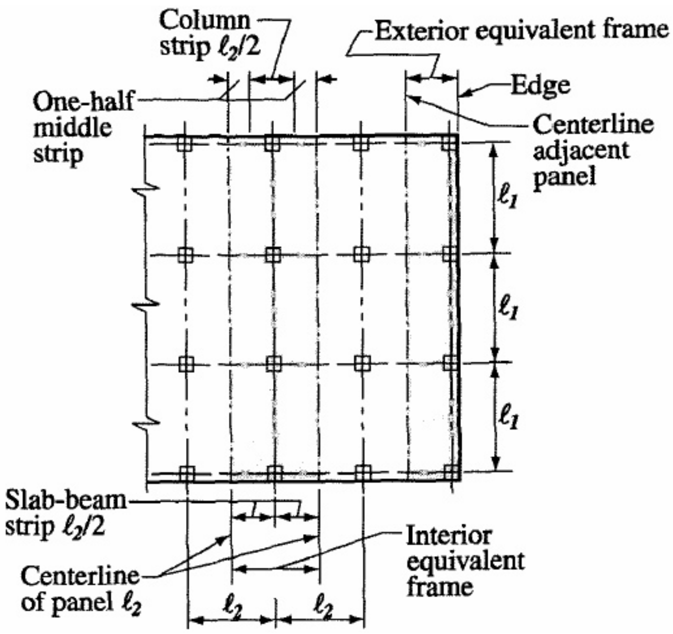


Fig. R13.7.2 - Definitions of equivalent frame

R13.7.3 **Slab-beams**

R13.7.3.3 A support is defined as a column, capital, bracket, or wall. A beam is not considered to be a support member for the equivalent frame.

R13.7.4 **Columns**

Column stiffness is based on the length of the column from mid-depth of slab above to mid-depth of slab below. Column moment of inertia is computed on the basis of its cross section, taking into account the increase in stiffness provided by the capital, if any.

When slab-beams are analyzed separately for gravity loads, the concept of an equivalent column, combining the stiffness of the slab-beam and torsional member into a composite element, is used. The column flexibility is modified to account for the torsional flexibility of the slab-to-column connection that reduces its efficiency for transmission of moments. The equivalent column

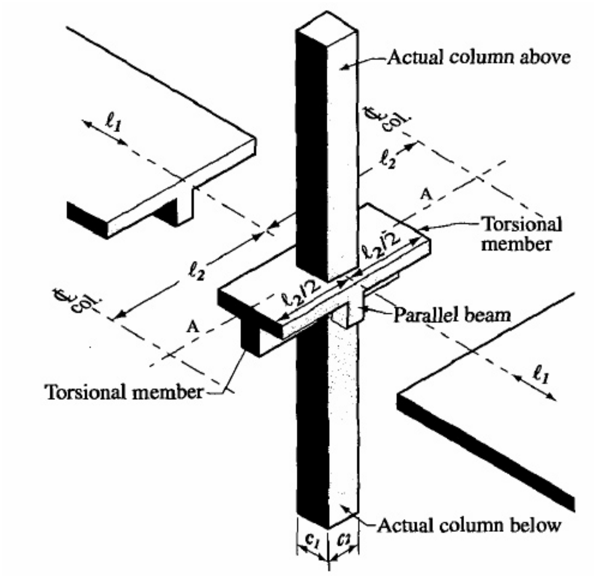


Fig. R13.7.4 - Equivalent column (column plus torsional members)

consists of the actual columns above and below the slab-beam, plus attached torsional members on each side of the columns extending to the centerline of the adjacent panels as shown in Fig. R13.7.4.

R13.7.5 **Torsional members**

Computation of the stiffness of the torsional member requires several simplifying assumptions. If no transverse-beam frames into the column, a portion of the slab equal to the width of the column or capital is assumed to be the torsional member. If a beam frames into the column, T-beam or L-beam action is assumed, with the flanges extending on each side of the beam a distance equal to the projection of the beam above or below the slab but not greater than four times the thickness of the slab. Furthermore, it is assumed that no torsional rotation occurs in the beam over the width of the support.

The member sections to be used for calculating the torsional stiffness are defined in 13.7.5.1. An approximate expression for the stiffness of the torsional member, based on the results of three-dimensional analysis of various slab configurations ((Refs 13.18, 13.19, and 13.20) is given below as:

Studies of three-dimensional analyses of various slab configurations suggest that a reasonable value of the torsional stiffness can be obtained by assuming a moment distribution along the torsional member that varies linearly from a maximum at the center of the column to zero at the middle of the panel. The assumed distribution of unit twisting moment along the column centerline is shown in Fig. R13.7.5.

An approximate expression for the stiffness of the torsional member, based on the results of three-dimensional analyses of various slab configurations (References 13.18, 13.19, and 13.20) is given below as

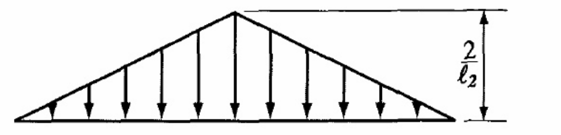


Fig. R13.7.5 - Distribution of unit twisting moment along column centerline AA shown in Fig. R13.7.4

$$K_t = \sum \frac{9 E_{cs} C}{\ell_2 \left(1 - \frac{c_2}{\ell_2}\right)^3}$$

where an expression for C is given in 13.0.

R13.7.6 Arrangement of live load

The use of only three-quarters of the full factored live load of maximum moments is thus possible before failure occurs. This procedure, in effect, permits some local over-stress under the full factored live load if it is distributed in the prescribed manner, but still ensures that the ultimate capacity of the slab system after redistribution of moment is not less than that required to carry the full factored dead and live loads on all panels.

R13.7.7 Factored moments

R13.7.7.1-
R13.7.7.3 These SBC 304 sections adjust the negative factored moments to the face of the supports. The adjustment is modified at an exterior support to limit reductions in the exterior negative moment. Fig. R13.6.2.5 illustrates several equivalent rectangular supports for use in establishing faces of supports for design with nonrectangular supports.

R13.7.7.4 If two different methods are prescribed to obtain a particular answer, the SBC 304 should not require a value greater than the least acceptable value. Due to the long satisfactory experience with designs having total factored static moments not exceeding those given by Eq. (13-3), it is considered that these values are satisfactory for design when applicable limitations are met.

CHAPTER 14
WALLS

SECTION R14.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

SECTION R14.1
SCOPE

Chapter 14 applies generally to walls as vertical load carrying members. Cantilever retaining walls are designed according to the flexural design provisions of Chapter 10. Walls designed to resist shear forces, such as shearwalls, should be designed in accordance with Chapter 14 and 11.10 as applicable.

SECTION R14.2
GENERAL

Walls should be designed to resist all loads to which they are subjected, including eccentric axial loads and lateral forces. Design is to be carried out in accordance with 14.4 unless the wall meets the requirements of 14.5.1.

SECTION R14.3
MINIMUM REINFORCEMENT

The requirements of 14.3 apply to wall designed according to 14.4, 14.5, or 14.8. For walls resisting horizontal shear forces in the plane of the wall, reinforcement designed according to 11.10.9.2 and 11.10.9.4 may exceed the minimum reinforcement in 14.3.

SECTION R14.5
EMPIRICAL DESIGN METHOD

The empirical design method applies only to solid rectangular cross sections. All other shapes should be designed according to 14.4.

Eccentric loads and lateral forces are used to determine the total eccentricity of the factored axial load P_u. When the resultant load for all applicable load combinations falls within the middle third of the wall thickness (eccentricity not greater than $h/6$) at all sections along the length of the undeformed wall, the empirical design method may be used. The design is then carried out considering P_u as the concentric load. The factored axial load P_u should be less than or equal to the design axial load strength ϕP_{nw} computed by Eq. (14-l), $P_u \leq \phi P_{nw}$.

Eq. 14-1 reflects the general range of end conditions encountered in wall designs. Values of effective vertical length factors k are given for commonly occurring wall end conditions.

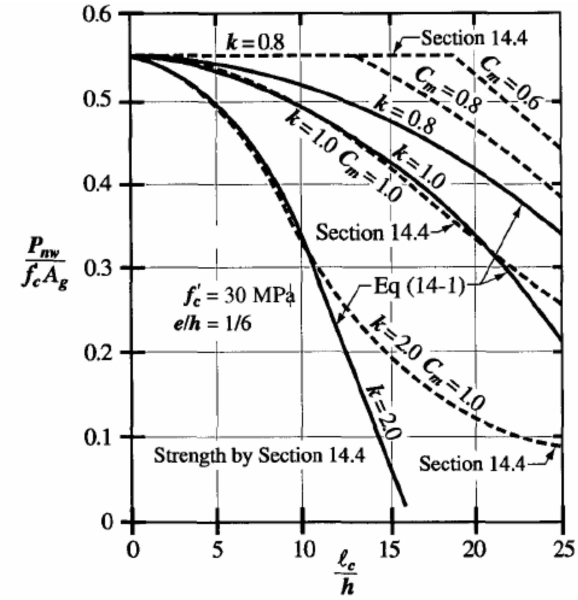


Fig. R14.5 - Empirical design of walls, Eq. (14-1) versus 14.4

The end condition "restrained against rotation" required for a k-factor of 0.8 implies attachment to a member having flexural stiffness EI/ℓ at least as large as that of the wall.The slenderness portion of Eq. (14-1) results in relatively comparable strengths by either 14.3 or 14.4 for members loaded at the middle third of the thickness with different braced and restrained end conditions. See Fig. R14.5.

The slenderness portion of Eq. (14-1) results in relatively comparable strengths by either 14.3 or 14.4 for members loaded at the middle third of the thickness with different braced and restrained end conditions. See Fig. R14.5.

R14.5.3 **Minimum thickness of walls designed by empirical design method**. The minimum thickness requirements need not be applied to walls designed according to 14.4.

SECTION R14.8
ALTERNATIVE DESIGN OF SLENDER WALLS

Section 14.8 is based on the corresponding requirements in Ref. 14.1 and experimental research.[14.2]

The procedure is presented as an alternative to the requirements of 10.10 for the out-of-plane design of precast wall panels, where the panels are restrained against overturning at the top.

Panels that have windows or other large openings are not considered to have constant cross section over the height of the panel. Such walls are to be designed taking into account the effects of openings.

Many aspects of the design of tilt-up walls and buildings are discussed in Refs. 14.3 and 14.4.

CHAPTER 15
FOOTINGS

SECTION R15.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

SECTION R15.1
SCOPE

While the provisions of Chapter 15 apply to isolated footings supporting a single column or wall, most of the provisions are generally applicable to combined footings and mats supporting several columns or walls or a combination thereof.[15.1, 15.2]

SECTION R15.2
LOADS AND REACTIONS

Footings are required to be proportioned to sustain the applied factored loads and induced reactions which include axial loads, moments, and shears that have to be resisted at the base of the footing or pile cap.

After the permissible soil pressure or the permissible pile capacity has been determined by principles of soil mechanics and in accordance with SBC 303. The size of the base area of a footing on soil or the number and arrangement of the piles should be established on the basis of unfactored (service) loads such as D, L, W, and E in whatever combination that governs the design, as per SBC 301.

Only the computed end moments that exist at the base of a column (or pedestal) need to be transferred to the footing; the minimum moment requirement for slenderness considerations given in 10.12.3.2 need not be considered for transfer of forces and moments to footings.

In cases in which eccentric loads or moments are to be considered, the extreme soil pressure or pile reaction obtained from this loading should be within the permissible values. Similarly, the resultant reactions due to service loads combined with moments, shears, or both, caused by wind or earthquake loads should not exceed the increased values that may be permitted by SBC 301.

To proportion a footing or pile cap for strength, the contact soil pressure or pile reaction due to the applied factored loading (see 8.1.1) should be determined. For a single concentrically loaded spread footing, the soil reaction q_s due to the factored loading is $q_s = U / A_f$ where U is the factored concentric load to be resisted by the footing, and A_f is the base area of the footing as determined by the principles stated in 15.2.2 using the unfactored loads and the permissible soil pressure.

q_s is a calculated reaction to the factored loading used to produce the same required strength conditions regarding flexure, shear, and development of reinforcement in the footing or pile cap, as in any other member.

In the case of eccentric loading, load factors may cause eccentricities and reactions that are different from those obtained by unfactored loads.

SECTION R15.5 SHEAR IN FOOTINGS

R15.5.1 and R15.5.2 The shear strength of footings are determined for the more severe condition of 11.12.1.1 or 11.12.1.2. The critical section for shear is measured from the face of supported member (column, pedestal, or wall), except for supported members on steel base plates.

Computation of shear requires that the soil reaction q_s be obtained from the factored loads and the design be in accordance with the appropriate equations of Chapter 11.

Where necessary, shear around individual piles may be investigated in accordance with 11.12.1.2. If shear perimeters overlap, the modified critical perimeter b_o should be taken as that portion of the smallest envelope of individual shear perimeter that will actually resist the critical shear for the group under consideration. One such situation is illustrated in Fig. R15.5.

R15.5.3 Pile caps supported on piles in more than one plane can be designed using three-dimensional strut-and-tie models satisfying Appendix A.[15.3] The effective concrete compressive strength is from A.3.2.2(b) because it is generally not feasible to provide confining reinforcement satisfying A.3.3.1 and A.3.3.2 in a pile cap.

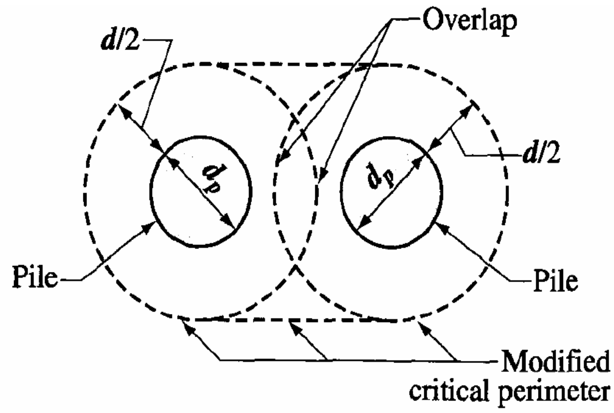


Fig. R15.5 - Modified critical perimeter for shear with overlapping critical perimeters

R15.5.4 When piles are located inside the critical sections d or $d/2$ from face of column, for one-way or two-way shear, respectively, an upper limit on the shear strength at a section adjacent to the face of the column should be considered CRSI Handbook [15.4] offers guidance for this situation.

SECTION R15.8 TRANSFER OF FORCE AT BASE OF COLUMN, WALL, OR REINFORCED PEDESTAL

Section 15.8 provides the specific requirements for force transfer from a column, wall, or pedestal (supported member) to a pedestal or footing (supporting member). Force transfer should be by bearing on concrete (compressive force only) and by reinforcement (tensile or compressive force). Reinforcement may consist of extended longitudinal bars, dowels, anchor bolts, or suitable mechanical connectors.

The requirements of 15.8.1 apply to both cast-in-place construction and precast construction. Additional requirements for cast-in-place construction are given in 15.8.2. Section 15.8.3 gives additional requirements for precast construction.

R15.8.1.1 Compressive force may be transmitted to a supporting pedestal or footing by bearing on concrete. For strength design, allowable bearing stress on the loaded area is equal to $0.85\phi f_c^{'}$, if the loaded area is equal to the area on which it is supported.

In the common case of a column bearing on a footing larger than the column, bearing strength should be checked at the base of the column and the top of the footing. Strength in the lower part of the column should be checked since the column reinforcement cannot be considered effective near the column base because the force in the reinforcement is not developed for some distance above the base, unless dowels are provided, or the column reinforcement is extended into the footing. The unit bearing stress on the column will normally be $0.85\phi f_c^{'}$. The permissible bearing strength on the footing may be increased in accordance with 10.17 and will usually be two times $0.85\phi f_c^{'}$. The compressive force that exceeds that developed by the permissible bearing strength at the base of the column or at the top of the footing should be carried by dowels or extended longitudinal bars.

R15.8.1.2 All tensile forces, whether created by uplift, moment, or other means, should be transferred to supporting pedestal or footing entirely by reinforcement or suitable mechanical connectors. Generally, mechanical connectors would be used only in precast construction.

R15.8.1.3 If computed moments are transferred from the column to the footing, the concrete in the compression zone of the column will be stressed to $0.85 f_c^{'}$ under factored load conditions and, as a result, all the reinforcement will generally have to be doweled into the footing.

R15.8.1.4 The shear-friction method given in 11.7 may be used to check for transfer of lateral forces to supporting pedestal or footing. Shear keys may be used, provided that the reinforcement crossing the joint satisfies 15.8.2.1, 15.8.3.1, and the shear-friction requirements of 11.7. In precast construction, resistance to lateral forces may be provided by shear-friction, shear keys, or mechanical devices.

R15.8.2.1 and
R15.8.2.2 A minimum amount of reinforcement is required between all supported and supporting members to ensure ductile behavior. The SBC 304 does not require that all bars in a column be extended through and be anchored into a footing. However, reinforcement with an area of 0.005 times the column area or an equal area of properly spliced dowels is required to extend into the footing with proper anchorage. This reinforcement is required to provide a degree of structural integrity during the construction stage and during the life of the structure.

R15.8.2.3 Lap splices of Dia 40 mm and larger longitudinal bars in compression only to dowels from a footing are specifically permitted in 15.8.2.3. The dowel bars should be Dia 36 mm or smaller in size. The dowel lap splice length should meet the larger of the two criteria: (a) be able to transfer the bar stress in the Dia 40 mm and larger bars, and (b) fully develop the stress in the dowels as a splice.

This provision is an exception to 12.14.2.1, which prohibits lap splicing of Dia 40 mm and larger bars. This exception results from many years of successful experience with the lap splicing of these large column bars with footing dowels of the smaller size. The reason for the restriction on dowel bar size is recognition of the anchorage length problem of the large bars, and to allow use of the smaller size dowels. A similar exception is allowed for compression splices between different size bars in 12.16.2.

R15.8.3.1 and
R15.8.3.2 For cast-in-place columns, 15.8.2.1 requires a minimum area of reinforcement equal to $0.005A_g$ across the column-footing interface to provide some degree of structural been arbitrarily chosen, it is not necessary to include integrity. For precast columns this requirement is expressed in terms of an equivalent tensile force that should be transferred. Thus, across the joint, $A_s f_y = 1.5A_g$ [see 16.5.1.3(a)]. The minimum tensile strength required for precast wall-to-footing connection [see 16.5.1.3(b)] is somewhat less than that required for columns, since an overload would be distributed laterally and a sudden failure would be less likely. Since the tensile strength values of 16.5.1.3 have a strength reduction factor ϕ for these calculations.

SECTION R15.10
COMBINED FOOTINGS AND MATS

R15.10.1 Any reasonable assumption with respect to the distribution of soil pressure or pile reactions can be used as long as it is consistent with the type of structure and the properties of the soil, and conforms with established principles of soil mechanics (see 15.1). Similarly, as prescribed in 15.2.2 for isolated footings, the base area or pile arrangement of combined footings and mats should be determined using the unfactored forces, moments, or both, transmitted by the footing to the soil, considering permissible soil pressures and pile reactions.

Design methods using factored loads and strength reduction factors ϕ can be applied to combined footings or mats, regardless of the soil pressure distribution.

Detailed recommendations for design of combined footings and mats are reported by Ref. 15.1. See also Ref. 15.2.

CHAPTER 16
PRECAST CONCRETE

SECTION R16.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

SECTION R16.1
SCOPE

R16.1.1 See 2.1 for definition of precast concrete.

Design and construction requirements for precast concrete structural members differ in some respects from those for cast-in-place concrete structural members and these differences are addressed in this chapter. Where provisions for cast-in-place concrete applied to precast concrete, they have not been repeated. Similarly, items related to composite concrete in Chapter 17 and to prestressed concrete in Chapter 18 that apply to precast concrete are not restated.

More detailed recommendations concerning precast concrete are given in References 16.1 through 16.7. Tilt-up concrete construction is a form of precast concrete. It is recommended that Reference 16.8 be reviewed for tilt-up structures.

SECTION R16.2
GENERAL

R16.2.1 Stresses developed in precast members during the period from casting to final connection may be greater than the service load stresses. Handling procedures may cause undesirable deformations. Care should be given to the methods of storing, transporting, and erecting precast members so that performance at service loads and strength under factored loads meet SBC 304 requirements.

R16.2.2 The structural behavior of precast members may differ substantially from that of similar members that are cast-in-place. Design of connections to minimize or transmit forces due to shrinkage, creep, temperature change, elastic deformation, differential settlement, wind, and earthquake require special consideration in precast construction.

R16.2.3 Design of precast members and connections is particularly sensitive to tolerances on the dimensions of individual members and on their location in the structure. To prevent misunderstanding, the tolerances used in design should be specified in the contract documents. The designer may specify the tolerance standard assumed in design. It is important to specify any deviations from accepted standards.

The tolerances required by 7.5 are considered to be a minimum acceptable standard for reinforcement in precast concrete. The designer should refer to publications of the Precast Prestressed Concrete Institute (PCI) (References 16.9, 16.10, 16.11) for guidance on industry established standard product and erection tolerances. Added guidance is given in Reference 16.12.

R16.2.4 The additional requirements may be included in either contract documents or shop drawings, depending on the assignment of responsibility for design.

SECTION R16.3
DISTRIBUTION OF FORCES AMONG MEMBERS

R16.3.1 Concentrated point and line loads can be distributed among members provided they have sufficient torsional stiffness and that shear can be transferred across joints. Torsionally stiff members such as hollow-core or solid slabs have more favorable load distribution properties than do torsionally flexible members such as double tees with thin flanges. The actual distribution of the load depends on many factors discussed in detail in References 16.13 through 16.19. Large openings can cause significant changes in distribution of forces.

R16.3.2 In-plane forces result primarily from diaphragm action in floors and roofs, causing tension or compression in the chords and shear in the body of the diaphragm. A continuous path of steel, steel reinforcement, or both, using lap splices, mechanical or welded splices, or mechanical connectors, should be provided to carry the tension, whereas the shear and compression may be carried by the net concrete section. A continuous path of steel through a connection includes bolts, weld plates, headed studs, or other steel devices. Tension forces in the connections are to be transferred to the primary reinforcement in the members.

In-plane forces in precast wall systems result primarily from diaphragm reactions and external lateral loads.

Connection details should provide for the forces and deformations due to shrinkage, creep, and thermal effects. Connection details may be selected to accommodate volume changes and rotations caused by temperature gradients and long-term deflections. When these effects are restrained, connections and members should be designed to provide adequate strength and ductility.

SECTION R16.4
MEMBER DESIGN

R16.4.1 For prestressed concrete members not wider than 4 m, such as hollow-core slabs, solid slabs, or slabs with closely spaced ribs, there is usually no need to provide transverse reinforcement to withstand shrinkage and temperature stresses in the short direction. This is generally true also for nonprestressed floor and roof slabs. The 4 m width is less than that in which shrinkage and temperature stresses can build up to a magnitude requiring transverse reinforcement. In addition, much of the shrinkage occurs before the members are tied into the structure. Once in the final structure, the members are usually not as rigidly connected transversely as monolithic concrete, thus the transverse restraint stresses due to both shrinkage and temperature change are significantly reduced.

The waiver does not apply to members such as single and double tees with thin, wide flanges.

R16.4.2 This minimum area of wall reinforcement, instead of the minimum values in 14.3, is recommended by Refs 16.4 and 16.20. The provisions for reduced minimum reinforcement and greater spacing recognize that precast wall panels have very little restraint at their edges during early stages of curing and develop less shrinkage stress than comparable cast-in-place walls.

SECTION R16.5
STRUCTURAL INTEGRITY

R16.5.1 The provisions of 7.13.3 apply to all precast concrete structures. Sections 16.5.1 and 16.5.2 give minimum requirements to satisfy 7.13.3. It is not intended that these minimum requirements override other applicable provisions of the SBC 304 for design of precast concrete structures.

The overall integrity of a structure can be substantially enhanced by minor changes in the amount, location, and detailing of member reinforcement and in the detailing of connection hardware.

R16.5.1.1 Individual members may be connected into a lateral load resisting system by alternative methods. For example, a load-bearing spandrel could be connected to a diaphragm (part of the lateral load resisting system). Structural integrity could be achieved by connecting the spandrel into all or a portion of the deck members forming the diaphragm. Alternatively, the spandrel could be connected only to its supporting columns, which in turn is connected to the diaphragm.

R16.5.1.2 Diaphragms are typically provided as part of the lateral load resisting system. The ties prescribed in 16.5.1.2 are the minimum required to attach members to the floor or roof diaphragms. The tie force is equivalent to the service load value of 3.0 kN/m.

R16.5.1.3 Base connections and connections at horizontal joints in precast columns and wall panels, including shear walls, are designed to transfer all design forces and moments. The minimum tie requirements of 16.5.1.3 are not additive to these design requirements. Common practice is to place the wall ties symmetrically about the vertical centerline of the wall panel and within the outer quarters of the panel width, wherever possible.

R16.5.1.4 In the event of damage to a beam, it is important that displacement of its supporting members be minimized, so that other members will not lose their load-carrying capacity. This situation shows why connection details that rely solely on friction caused by gravity loads are not used. An exception could be heavy modular unit structures (one or more cells in cell-type structures) where resistance to overturning or sliding in any direction has a large factor of safety. Acceptance of such systems should be based on the provisions of 1.4.

R16.5.2 The structural integrity minimum tie provisions for bearing wall structures, often called large panel structures, are intended to provide catenary hanger supports in case of loss of a bearing wall support, as shown by test.[16.21]

Forces induced by loading, temperature change, creep, and wind or seismic action may require a larger amount of tie force. It is intended that the general precast concrete provisions of 16.5.1 apply to bearing wall structures less than three stories in height.

Minimum ties in structures three or more stories in height, in accordance with 16.5.2.1, 16.5.2.2, 16.5.2.3, 16.5.2.4, and 16.5.2.5, are required for structural integrity (Fig. R16.5.2). These provisions are based on PCI's recommendations for design of precast concrete bearing wall buildings.[6.22] Tie capacity is based on yield strength.

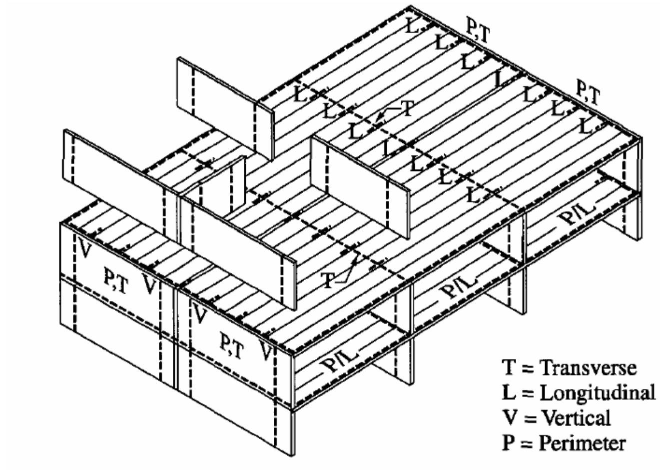


Fig. RI 6.5.2 - Typical arrangement of tensile ties in large panel structures

R16.5.2.1 Longitudinal ties may project from slabs and be lap spliced, welded, or mechanically connected, or they may be embedded in grout joints, with sufficient length and cover to develop the required force. Bond length for unstressed prestressing steel should be sufficient to develop the yield strength.[16.23] It is uncommon to have ties positioned in the walls reasonably close to the plane of the floor or roof system.

R16.5.2.3 Transverse ties may be uniformly spaced either encased in the panels or in a topping, or they may be concentrated at the transverse bearing walls.

R16.5.2.4 The perimeter tie requirements need not be additive with the longitudinal and transverse tie requirements.

SECTION R16.6
CONNECTION AND BEARING DESIGN

R16.6.1 The SBC 304 permits a variety of methods for connecting members. These are intended for transfer of forces both in-plane and perpendicular to the plane of the members.

R16.6.1.2 Various components in a connection (such as bolts, welds, plates, and inserts) have different properties that can affect the overall behavior of the connection.

R16.6.2.1 When tensile forces occur in the plane of the bearing, it may be desirable to reduce the allowable bearing stress, provide confinement reinforcement, or both. Guidelines are provided in Reference 16.4.

R16.6.2.2 This section differentiates between bearing length and length of the end of a precast member over the support (Fig. R16.6.2). Bearing pads distribute concentrated loads and reactions over the bearing area, and allow limited horizontal and rotational movements for stress relief. To prevent spalling under heavily loaded bearing areas, bearing pads should not extend to the edge of the support unless the edge is armored. Edges can be armored with anchored steel plates or angles. Section 11.9.7 gives requirements for bearing on brackets or corbels.

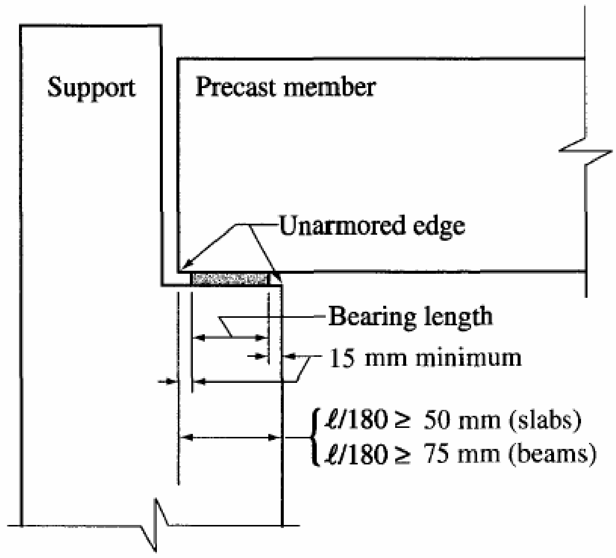


Fig. R16.6.2 - Bearing length on support

R16.6.2.3 It is unnecessary to develop positive bending moment reinforcement beyond the ends of the precast element if the system is statically determinate. Tolerances need to be considered to avoid bearing on plain concrete where reinforcement has been discontinued.

SECTION R16.7
ITEMS EMBEDDED AFTER CONCRETE PLACEMENT

R16.7.1 Section 16.7.1 is an exception to the provisions of 7.5.1. Many precast products are manufactured in such a way that it is difficult, if not impossible, to position reinforcement that protrudes from the concrete before the concrete is placed. Such items as ties for horizontal shear and inserts can be placed while the concrete is plastic, if proper precautions are taken. This exception is not applicable to reinforcement that is completely embedded, or to embedded items that will be hooked or tied to embedded reinforcement.

SECTION R16.9
HANDLING

R16.9.1 The SBC 304 requires acceptable performance at service loads and adequate strength under factored loads. However, handling loads should not produce permanent stresses, strains, cracking, or deflections inconsistent with the provisions of the SBC 304. A precast member should not be rejected for minor cracking or spalling where strength and durability are not affected. Guidance on assessing cracks is given in PCI reports on fabrication and shipment cracks.[16.24, 16.25]

R16.9.2 All temporary erection connections, bracing, shoring as well as the sequencing of removal of these items are shown on contract or erection drawings.

SECTION R16.10
STRENGTH EVALUATION OF PRECAST CONSTRUCTION

The strength evaluation procedures of Chapter 20 are applicable to precast members.

CHAPTER 17
COMPOSITE CONCRETE FLEXURAL MEMBERS

SECTION R17.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

SECTION R17.1
SCOPE

R17.1.1 The scope of Chapter 17 is intended to include all types of composite concrete flexural members. In some cases with fully cast-in-place concrete, it may be necessary to design the interface of consecutive placements of concrete as required for composite members. Composite structural steel-concrete members are not covered in this chapter. Design provisions for such composite members are covered in Reference 17.1.

SECTION R17.2
GENERAL

R17.2.4 Tests have indicated that the strength of a composite member is the same whether or not the first element cast is shored during casting and curing of the second element.

R17.2.6 The extent of cracking is dependent on such factors as environment, aesthetics, and occupancy. In addition, composite action should not be impaired.

R17.2.7 The premature loading of precast elements can cause excessive creep and shrinkage deflections. This is especially so at early ages when the moisture content is high and the strength low.

The transfer of shear by direct bond is important if excessive deflection from slippage is to be prevented. A shear key is an added mechanical factor of safety but it does not operate until slippage occurs.

SECTION R17.3
SHORING

The provisions of 9.5.5 cover the requirements pertaining to deflections of shored and unshored members.

SECTION R17.5
HORIZONTAL SHEAR STRENGTH

R17.5.1 Full transfer of horizontal shear between segments of composite members should be ensured by honzontal shear strength at contact surfaces or properly anchored ties, or both.

R17.5.2 The nominal horizontal shear strengths V_{nh} apply when the design is based on the load factors and ϕ factors of Chapter 9.

Prestressed members used in composite construction may have variations in depth of tension reinforcement along member length due to draped or depressed tendons. Because of this variation, the definition of d used in Chapter 11 for determination of vertical shear strength is also appropriate when determining horizontal shear strength.

R17.5.2.3 The permitted horizontal shear strengths and the requirement of 5 mm amplitude for intentional roughness are based on tests discussed in References 17.2 through 17.4.

R17.5.3.1 The distribution of horizontal shear stresses along the contact surface in a composite member will reflect the distribution of shear along the member. Horizontal shear failure will initiate where the horizontal shear stress is a maximum and will spread to regions of lower stress. Because the slip at peak horizontal shear resistance is small for a concrete-to-concrete contact surface, longitudinal redistribution of horizontal shear resistance is very limited. The spacing of the ties along the contact surface should, therefore, be such as to provide horizontal shear resistance distributed approximately as the shear acting on the member is distributed.

R17.5.4 Proper anchorage of ties extending across interfaces is required to maintain contact of the interfaces.

SECTION R17.6 TIES FOR HORIZONTAL SHEAR

The minimum areas and maximum spacings are based on test data given in References 17.2 through 17.6.

CHAPTER 18
PRESTRESSED CONCRETE

SECTION R18.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

The factored prestressing force P_{su} is the product of the load factor (1.2 from Section 9.2.5) and the maximum prestressing force allowed. Under 18.5.1 this is usually overstressing to $0.94 f_{py}$ but not greater than $0.8 f_{pu}$, which is permitted for short periods of time.

$$\begin{aligned} P_{su} &= (1.2)(0.80) f_{pu} A_{ps} \\ &= 0.96 f_{pu} A_{ps} \end{aligned}$$

SECTION R18.1
SCOPE

R18.1.1 The provisions of Chapter 18 were developed primarily for structural members such as slabs, beams, and columns that are commonly used in buildings. Many of the provisions may be applied to other types of construction, such as, pressure vessels, pavements, pipes, and crossties. Application of the provisions is left to the judgment of the engineer in cases not specifically cited in the SBC 304.

R18.1.3 Some sections of the SBC 304 are excluded from use in the design of prestressed concrete for specific reasons. The following discussion provides explanation for such exclusions:

Section 7.6.5 of the SBC 304 is excluded from application to prestressed concrete because the requirements for bonded reinforcement and unbonded tendons for cast-in-place members are provided in 18.9 and 18.12, respectively.

The empirical provisions of 8.10.2, 8.10.3, and 8.10.4 for T-beams were developed for nonprestressed reinforced concrete, and if applied to prestressed concrete would exclude many standard prestressed products in satisfactory use today. Hence, proof by experience permits variations.

By excluding 8.10.2, 8.10.3, and 8.10.4, no special requirements for prestressed concrete T-beams appear in the SBC 304. Instead, the determination of an effective width of flange is left to the experience and judgment of the engineer. Where possible, the flange widths in 8.10.2, 8.10.3, and 8.10.4 should be used unless experience has proven that variations are safe and satisfactory. It is not necessarily conservative in elastic analysis and design considerations to use the maximum flange width as permitted in 8.10.2.

Sections 8.10.1 and 8.10.5 provide general requirements for T-beams that are also applicable to prestressed concrete members. The spacing limitations for slab reinforcement are based on flange thickness, which for tapered flanges can be taken as the average thickness.

In Section 8.11, the empirical limits established for nonprestressed reinforced concrete joist floors are based on successful past performance of joist construction using standard joist forming systems. See R8.11. For prestressed joist construction, experience and judgment should be used. The provisions of 8.11 may be used as a guide.

For prestressed concrete, the limitations on reinforcement given in 10.5, 10.9.1, and 10.9.2 are replaced by those in 18.8.3, 18.9, and 18.11.2.

Section 10.6 does not apply to prestressed members in its entirety. However, 10.6.4 and 10.6.7 are referenced in 18.4.4 pertaining to Class C prestressed flexural members.

In Chapter 13, the design of continuous prestressed concrete slabs requires recognition of secondary moments. Also, volume changes due to the prestressing force can create additional loads on the structure that are not adequately covered in Chapter 13. Because of these unique properties associated with prestressing, many of the design procedures of Chapter 13 are not appropriate for prestressed concrete structures and are replaced by the provisions of 18.12.

The requirements for wall design in 14.5 and 14.6 are largely empirical, utilizing considerations not intended to apply to prestressed concrete.

SECTION R18.2
GENERAL

R18.2.1 and
R18.2.2 The design investigation should include all stages that may be significant. The three major stages are: (1) jacking stage, or prestress transfer stage–when the tensile force in the prestressing steel is transferred to the concrete and stress levels may be high relative to concrete strength; (2) service load stage–after long-term volume changes have occurred; and (3) the factored load stage–when the strength of the member is checked. There may be other load stages that require investigation. For example, if the cracking load is significant, this load stage may require study, or the handling and transporting stage may be critical.

From the standpoint of satisfactory behavior, the two stages of most importance are those for service load and factored load.

Service load stage refers to the loads defined in the general building SBC 304 (without load factors), such as live load and dead load, while the factored load stage refers to loads multiplied by the appropriate load factors.

Section 18.3.2 provides assumptions that may be used for investigation at service loads and after transfer of the prestressing force.

R18.2.5 Section 18.2.5 refers to the type of post-tensioning where the prestressing steel makes intermittent contact with an oversize duct. Precautions should be taken to prevent buckling of such members.

If the prestressing steel is in complete contact with the member being prestressed, or is unbonded with the sheathing not excessively larger than the prestressing steel, it is not possible to buckle the member under the prestressing force being introduced.

R18.2.6 In considering the area of the open ducts, the critical sections should include those that have coupler sheaths that may be of a larger size than the duct containing the prestressing steel. Also, in some instances, the trumpet or transition piece from the conduit to the anchorage may be of such a size as to create a critical section. If the effect of the open duct area on design is deemed negligible, section properties may be based on total area.

In post-tensioned members after grouting and in pretensioned members, section properties may be based on effective sections using transformed areas of bonded prestressing steel and nonprestressed reinforcement gross sections, or net sections.

SECTION R18.3 DESIGN ASSUMPTIONS

R18.3.3 This section defines three classes of behavior of prestressed flexural members. Class U members are assumed to behave as uncracked members. Class C members are assumed to behave as cracked members. The behavior of Class T members is assumed to be in transition between uncracked and cracked. The serviceability requirements for each class are summarized in Table R18.3.3. For comparison, Table R18.3.3 also shows corresponding requirements for nonprestressed members.

These classes apply to both bonded and unbonded prestressed flexural members, but prestressed two-way slab systems must be designed as Class U.

The precompressed tensile zone is that portion of the member cross section in which flexural tension occurs under dead and live loads. Prestressed concrete is usually designed so that the prestress force introduces compression into this zone, thus effectively reducing the magnitude of the tensile stress.

TABLE R18.3.3 — SERVICEABILITY DESIGN REQUIREMENTS

	Prestressed			Nonprestressed
	Class U	**Class T**	**Class C**	
Assumed behavior	**Uncracked**	**Transition between uncracked and cracked**	**Cracked**	**Cracked**
Section properties for stress calculation at service loads	Gross section 18.3.4	Gross section 18.3.4	Cracked section 18.3.4	No requirement
Allowable stress at transfer	18.4.1	18.4.1	18.4.1	No requirement
Allowable compressive stress based on uncracked section properties	18.4.2	18.4.2	No requirement	No requirement
Tensile stress at service loads 18.3.3	$\leq 0.7\sqrt{f_c'}$	$0.7\sqrt{f_c'} < f_t \leq \sqrt{f_c'}$	No requirement	No requirement
Deflection calculation basis	9.5.4.1 Gross section	9.5.4.2 Cracked section, bilinear	9.5.4.2 Cracked section, bilinear	9.5.2, 9.5.3 Cracked effective moment of inertia
Crack control	No requirement	No requirement	10.6.4 Modified by 18.4.4.1	10.6.4
Computation Δf_{ps} or f_s for crack control	---	---	Cracked section analysis	$M/(A_s$ *x lever arm*$)$ or $0.6 f_y$
Side skin reinforcement	No requirement	No requirement	10.6.7	10.6.7

R18.3.4 A method for computing stresses in a cracked section is given in Reference 18.1.

R18.3.5 Reference 18.2 provides information on computing deflections of cracked members.

SECTION R18.4
SERVICEABILITY REQUIREMENTS – FLEXURAL MEMBERS

Permissible stresses in concrete address serviceability. Permissible stresses do not ensure adequate structural strength, which should be checked in conformance with other SBC 304 requirements.

R18.4.1 The concrete stresses at this stage are caused by the force in the prestressing steel at transfer reduced by the losses due to elastic shortening of the concrete, relaxation of the prestressing steel, seating at transfer, and the stresses due to the weight of the member. Generally, shrinkage and creep effects are not included at this stage. These stresses apply to both pretensioned and post-tensioned concrete with proper modifications of the losses at transfer.

R18.4.l(b) and (c) The tension stress limits of $(1/4)\sqrt{f'_{ci}}$ and $(1/2)\sqrt{f'_{ci}}$ refer to tensile stress at locations other than the precompressed tensile zone. Where tensile stresses exceed the permissible values, the total force in the tensile stress zone may be calculated and reinforcement proportioned on the basis of this force at a stress of $0.6f_y$, but not more than 200 MPa. The effects of creep and shrinkage begin to reduce the tensile stress almost immediately; however, some tension remains in these areas after allowance is made for all prestress losses.

R18.4.2(a) and (b) The compression stress limit of $0.45f'_c$ was conservatively established to decrease the probability of failure of prestressed concrete members due to repeated loads. This limit seemed reasonable to preclude excessive creep deformation. At higher values of stress, creep strains tend to increase more rapidly as applied stress increases.

Designs with transient live loads that are large compared to sustain live and dead loads have been penalized by the previous single compression stress limit. Therefore, the stress limit of $0.60f'_c$, permits a one-third increase in allowable compression stress for members subject to transient loads.

Sustained live load is any portion of the service live load that will be sustained for a sufficient period to cause significant time-dependent deflections. Thus, when the sustained live and dead loads are a large percentage of total service load, the $0.45f'_c$ limit of 18.4.2(a) may control. On the other hand, when a large portion of the total service load consists of a transient or temporary service live load, the increased stress limit of 18.4.2(b) may apply.

The compression limit of $0.45f'_c$ for prestress plus sustained loads will continue to control the long-term behavior of prestressed members.

R18.4.3 This section provides a mechanism whereby development of new products, materials, and techniques in prestressed concrete construction need not be inhibited by SBC 304 limits on stress. Approvals for the design should be in accordance with 1.4 of the SBC 304.

R18.4.4 For conditions of corrosive environments, defined as an environment in which chemical attack (such as seawater, corrosive industrial atmosphere, or sewer gas) is encountered, cover greater than that required by 7.7.2 should be used, and tension stresses in the concrete reduced to eliminate possible cracking at service loads. The engineer should use judgment to determine the amount of increased cover and whether reduced tension stresses are required.

R18.4.4.1 Only tension steel nearest the tension face need be considered in selecting the value of c_c used in computing spacing requirements. To account for prestressing steel, such as strand, having bond characteristics less effective than deformed reinforcement, a 2/3 effectiveness factor is used.

For post-tensioned members designed as cracked members, it will usually be advantageous to provide crack control by the use of deformed reinforcement, for which the provisions of 10.6 may be used directly. Bonded reinforcement required by other provisions of this SBC 304 may also be used as crack control reinforcement.

R18.4.4.2 It is conservative to take the decompression stress f_{dc} equal to the effective prestress f_{se}.

R18.4.4.4 The steel area of reinforcement, bonded tendons, or a combination of both may be used to satisfy this requirement.

SECTION R18.5
PERMISSIBLE STRESSES IN PRESTRESSING STEEL

The SBC 304 does not distinguish between temporary and effective prestressing steel stresses. Only one limit on prestressing steel stress is provided because the initial prestressing steel stress (immediately after transfer) can prevail for a considerable time, even after the structure has been put into service. This stress, therefore, should have an adequate safety factor under service conditions and cannot be considered as a temporary stress. Any subsequent decrease in prestressing steel stress due to losses can only improve conditions and no limit on such stress decrease is provided in the SBC 304.

R18.5.1 For higher yield strength of low-relaxation wire and strand meeting the requirements of ASTM A 421M and A 416M, it is more appropriate to specify permissible stresses in terms of specified minimum ASTM yield strength rather than specified minimum ASTM tensile strength. For the low-relaxation wire and strands, with f_{py} equal to $0.90f_{pu}$, the $0.94f_{py}$ and $0.82f_{py}$ limits are equivalent to $0.85f_{pu}$ and $0.74f_{pu}$, respectively. The higher yield strength of the low-relaxation prestressing steel does not change the effectiveness of tendon anchorage devices; thus, the permissible stress at post-tensioning anchorage devices and couplers is not increased above the previously permitted value of $0.70f_{pu}$. For ordinary prestressing steel (wire, strands, and bars) with f_{py} equal to $0.85f_{pu}$ the $0.94f_{py}$ and $0.82f_{py}$ limits are equivalent to $0.80f_{pu}$, and $0.70f_{pu}$, respectively. For bar prestressing steel with f_{py} equal to $0.80f_{pu}$, the same limits are equivalent to $0.75f_{pu}$ and $0.66f_{pu}$, respectively.

Designers should be concerned with setting a limit on final stress when the structure is subject to corrosive conditions or repeated loadings.

SECTION R18.6
LOSS OF PRESTRESS

R18.6.1 For an explanation of how to compute prestress losses, see Ref. 18.3 through 18.6. Reasonably accurate estimates of prestress losses can be calculated in accordance with the recommendations in Reference 18.6, which include consideration of initial stress level ($0.70 f_{pu}$ or higher), type of steel (stress-relieved or low-relaxation wire, strand, or bar), exposure conditions, and type of construction (pretensioned, bonded post-tensioned, or unbonded post-tensioned).

Actual losses, greater or smaller than the computed values, have little effect on the design strength of the member, but affect service load behavior (deflections, camber, cracking load) and connections. At service loads, overestimation of prestress losses can be almost as detrimental as underestimation, since the former can result in excessive camber and horizontal movement.

R18.6.2 **Friction loss in post-tensioning tendons.** The coefficients tabulated in Table R18.6.2 give a range that generally can be expected. Due to the many types of prestressing steel ducts and sheathing available, these values can only serve as a guide. Where rigid conduit is used, the wobble coefficient K can be considered as zero. For large diameter prestressing steel in semirigid type conduit, the wobble factor can also be considered zero. Values of the coefficients to be used for the particular types of prestressing steel and particular types of ducts should be obtained from the manufacturers of the tendons. An unrealistically low evaluation of the friction loss can lead to improper camber of the member and inadequate prestress, Overestimation of the friction may result in extra prestressing force. This could lead to excessive camber and excessive shortening of a member. If the friction factors are determined to be less than those assumed in the design, the tendon stressing should be adjusted to give only that prestressing force in the critical portions of the structure required by the design.

TABLE R18.6.2-FRICTION COEFFICIENTS FOR POST-TENSIONEDTENDONS FOR USE IN EQ. (18-1) OR (18-2)

			Wobble coefficient, K	Curvature coefficient, μ
Grouted tendons in metal sheathing		Wire tendons	0.0010-0.0015	0.15-0.25
		High-strength bars	0.0001-0.0006	0.08-0.30
		7-wire strand	0.0005-0.0020	0.15-0.25
Unbonded tendons	Mastic coated	Wire tendons	0.0010-0.0020	0.05-0.15
		7-wire strand	0.0010-0.0020	0.05-0.15
	Pregreased	Wire tendons	0.0003-0.0020	0.05-0.15
		7-wire strand	0.0003-0.0020	0.05-0.15

R18.6.2.3 When the safety or serviceability of the structure may be involved, the acceptable range of prestressing steel jacking forces or other limiting requirements should either be given or approved by the structural engineer in conformance with the permissible stresses of 18.4 and 18.5.

SECTION R18.7
FLEXURAL STRENGTH

R18.7.1 Design moment strength of prestressed flexural members may be computed using strength equations similar to those for nonprestressed concrete members. When part of the prestressing steel is in the compression zone, a method based on applicable conditions of equilibrium and compatibility of strains at a factored load condition should be used.

For cross sections other than rectangular, the design moment strength ϕM_n is computed by an analysis based on stress and strain compatibility, using the stress-strain properties of the prestressing steel and the assumptions given in 10.2.

R18.7.2 Eq. (18-3) may underestimate the strength of beams with high percentages of reinforcement and, for more accurate evaluations of their strength, the strain compatibility and equilibrium method should be used. Use of Eq. (18-3) is appropriate when all of the prestressed reinforcement is in the tension zone. When part of the prestressed reinforcement is in the compression zone, a strain compatibility and equilibrium method should be used.

By inclusion of the ω' term, Eq. (18-3) reflects the increased value of f_{ps} obtained when compression reinforcement is provided in a beam with a large reinforcement index. When the term $\left[\rho_p f_{pu} / f_c' + (d/d_p)(\omega - \omega')\right]$ in Eq. (18-3) is small, the neutral axis depth is small, the compressive reinforcement does not develop its yield strength, and Eq. (18-3) becomes unconservative. This is the reason why the term $\left[\rho_p f_{pu} / f_c' + (d/d_p)(\omega - \omega')\right]$ in Eq. (18-3) may not be taken less than 0.17 if compression reinforcement is taken into account when computing f_{ps}. If the compression reinforcement is neglected when using Eq. (18-3), d' is taken as zero, then the term $\left[\rho_p f_{pu} / f_c' + (d/d_p)\omega\right]$ may be less than 0.17 and an increased and correct value of f_{ps} is obtained.

When d' is large, the strain in compression reinforcement can be considerably less than its yield strain. In such a case, the compression reinforcement does not influence f_{ps} as favorably as implied by Eq. (18-3). For this reason, the applicability of Eq. (18-3) is limited to beams in which d' is less than or equal to $0.15d_p$.

The term $\left(\rho_p f_{pu} / f_c' + (d/d_p)(\omega - \omega')\right)$ in Eq. (18-3)

may also be written $\left(\rho_p f_{pu}/f_c' + A_s f_y/(b d_p f_c') - A_s' f_y'/(b d_p f_c')\right)$.

This form may be more convenient, such as when there is no unprestressed tension reinforcement.

Eq. (18-5) reflects results of tests on members with unbonded tendons and span-to-depth ratios greater than 35

Eq. (18.5) reflects results of tests on members with unbonded tendons and span-to-depth ratios greater than 35 (one-way slabs, flat plates, and flat slabs).[18.7] These tests also indicate that Eq. (18-4), overestimates the amount of stress increase in such members. Although these same tests indicate that the moment strength of those shallow members designed using Eq. (18-4) meets the factored load strength requirements, this reflects the effect of the SBC 304 requirements for minimum bonded reinforcement, as well as the limitation on concrete tensile stress that often controls the amount of prestressing force provided.

SECTION R18.8 LIMITS FOR REINFORCEMENT OF FLEXURAL MEMBERS

R18.8.1 The net tensile strain limits for compression- and tension-controlled sections given in 10.3.3 and 10.3.4 apply to prestressed sections.

The net tensile strain limits for tension-controlled sections given in 10.3.4 may also be stated in terms of ω_p. The net tensile strain limit of 0.005 corresponds to $\omega_p = 0.32\beta_1$ for pre- stressed rectangular sections.

R18.8.2 This provision is a precaution against abrupt flexural failure developing immediately after cracking. A flexural member designed according to SBC 304 provisions requires considerable additional load beyond cracking to reach its flexural strength. Thus, considerable deflection would warn that the member strength is approaching. If the flexural strength were reached shortly after cracking, the warning deflection would not occur.

R18.8.3 Some bonded steel is required to be placed near the tension face of prestressed flexural members. The purpose of this bonded steel is to control cracking under full service loads or overloads.

SECTION R18.9 MINIMUM BONDED REINFORCEMENT

R18.9.1 Some bonded reinforcement is required by the SBC 304 in members prestressed with unbonded tendons to ensure flexural performance at ultimate member strength, rather than as a tied arch, and to limit crack width and spacing at service load when concrete tensile stresses exceed the modulus of rupture. Providing the minimum bonded reinforcement as stipulated in 18.9helps to ensure adequate performance.

Research has shown that unbonded post-tensioned members do not inherently provide large capacity for energy dissipation under severe earthquake loadings because the member response is primarily elastic. For this reason, unbonded post-tensioned structural elements reinforced in accordance with the provisions of this section should be assumed to carry only vertical loads and to act as horizontal diaphragms between energy dissipating elements under earthquake loadings of the magnitude defined in 21.2.1.1. The minimum bonded reinforcement areas required by Eq. (18-6) and (18-8) are absolute minimum areas independent of grade of steel or design yield strength.

R18.9.2 The minimum amount of bonded reinforcement for members other than two-way flat slab systems is based on research comparing the behavior of bonded and unbonded post-tensioned beams.[18.8] Based on this research, it is advisable to apply the provisions of 18.9.2 also to one-way slab systems.

R18.9.3 The minimum amount of bonded reinforcement in two-way flat slab systems is based on reports of References: 18.3 & 18.9.Limited research available for two-way flat slabs with drop panels 18.16 indicates that behavior of these particular systems is similar to the behavior of flat plates.

R18.9.3.1 For usual loads and span lengths, flat plate tests summarized in Reference: 18.3 indicate satisfactory performance without bonded reinforcement in the areas described in 18.9.3.1.

R18.9.3.2 In positive moment areas, where the concrete tensile stresses are between $(1/6)\sqrt{f_c'}$ and $(1/2)\sqrt{f_c'}$ a minimum bonded reinforcement area proportioned according to Eq. (18-7) is required. The tensile force N, is computed at service load on the basis of an uncracked, homogeneous section.

R18.9.3.3 Research on unbonded post-tensioned two way flat slab systems shows that bonded reinforcement in negative moment regions, proportioned on the basis of 0.075 percent of the cross-sectional area of the slab-beam strip, provides sufficient ductility and reduces crack width and spacing (References: 18.1, 18.3, 18.9, 18.10). To account for different adjacent tributary spans, Eq. (18-8) is given on the basis of the equivalent frame as defined in 13.7.2 and pictured in Fig. R13.7.2. For rectangular slab panels, Eq. (18-8) is conservatively based upon the larger of the cross-sectional areas of the two intersecting equivalent frame slab-beam strips at the column. This ensures that the minimum percentage of steel recommended by research is provided in both directions. Concentration of this reinforcement in the top of the slab directly over and immediately adjacent to the column is important. Research also shows that where low tensile stresses occur at service loads, satisfactory behavior has been achieved at factored loads without bonded reinforcement. However, the SBC 304 requires minimum bonded reinforcement regardless of service load stress levels to help ensure flexural continuity and ductility, and to limit crack widths and spacing due to overload, temperature, or shrinkage. Research on post-tensioned flat plate-to-column connections is reported in References 18.11, 18.12, 18.13, 18.14, and 18.15.

R18.9.4 Bonded reinforcement should be adequately anchored to develop factored load forces. The requirements of Chapter 12 will ensure that bonded reinforcement required for flexural strength under factored loads in accordance with 18.7.3, or for tensile stress conditions at service load in accordance with 18.9.3.2, will be adequately anchored to develop tension or compression forces. The minimum lengths apply for bonded reinforcement required by 18.9.2 or 18.9.3.3, but not required for flexural strength in accordance with 18.7.3. Research[18.1] on continuous spans shows that these minimum lengths provide adequate behavior under service load and factored load conditions.

SECTION R18.10
STATICALLY INDETERMINATE STRUCTURES

R18.10.3 For statically indeterminate structures, the moments due to reactions induced by prestressing forces, referred to as secondary moments, are significant in both the elastic and inelastic states. When hinges and full redistribution of moments occur to create a statically determinate structure, secondary moments disappear. However, the elastic deformations caused by a nonconcordant tendon change the amount of inelastic rotation required to obtain a given amount of moment redistribution. Conversely, for a beam with a given inelastic rotational capacity, the amount by which the moment at the support may be varied is changed by an amount equal to the secondary moment at the support due to prestressing. Thus, the SBC 304 requires that secondary moments be included in determining design moments.

To determine the moments used in design, the order of calculation should be: (a) determine moments due to dead and live load; (b) modify by algebraic addition of secondary moments; (c) redistribute as permitted. A positive secondary moment at the support caused by a tendon transformed downward from a concordant profile will reduce the negative moments near the supports and increase the positive moments in the midspan regions. A tendon that is transformed upward will have the reverse effect.

R18.10.4 **Redistribution of negative moments in continuous prestressed flexural members.** The provisions for redistribution of negative moments given in 8.4 apply equally to prestressed members. See Ref. 9.16 for a comparison of research results.

For the moment redistribution principles of 18.10.4 to be applicable to beams with unbonded tendons, it is necessary that such beams contain sufficient bonded reinforcement to ensure they will act as beams after cracking and not as a series of tied arches. The minimum bonded reinforcement requirements of 18.9 will serve this purpose.

SECTION R18.11 COMPRESSION MEMBERS-COMBINED FLEXURE AND AXIAL LOADS

R18.11.2 **Limits for reinforcement of prestressed compression members**

R18.11.2.3 The minimum amounts of reinforcement in 14.3 need not apply to prestressed concrete walls, provided the average prestress is 1.5 MPa or greater and a structural analysis is performed to show adequate strength and stability with lower amounts of reinforcement.

SECTION R18.12 SLAB SYSTEMS

R18.12.1 Use of the equivalent frame method of analysis (see 13.7) or more precise analysis procedures is required for determination of both service and factored moments and shears for prestressed slab systems. The equivalent frame method of analysis has been shown by tests of large structural models to satisfactorily predict factored moments and shears in prestressed slab systems. (See References 18.11, 18.12, 18.13, 18.17, 18.18, and 18.19). The referenced research also shows that analysis using prismatic sections or other approximations of stiffness may provide erroneous results on the unsafe side. Section 13.7.7.4 is excluded from application to prestressed slab systems because it relates to reinforced slabs designed by the direct design method, and because moment redistribution for prestressed slabs is covered in 18.10.4. Section 13.7.7.5 does not apply to prestressed slab systems because the distribution of moments between column strips and middle strips required by 13.7.7.5 is based on tests for nonprestressed concrete slabs. Simplified methods of analysis using average coefficients do not apply to prestressed concrete slab systems.

R18.12.2 Tests indicate that the moment and shear strength of prestressed slabs is controlled by total prestessing steel strength and by the amount and location of nonprestressed reinforcement, rather than by tendon distribution. (See References 18.11, 18.12, 18.13, 18.17, 18.18, and 18.19).

R18.12.3 For prestressed flat slabs continuous over two or more spans in each direction, the span-thickness ratio generally should not exceed 42 for floors and 48 for roofs; these limits may be increased to 48 and 52, respectively, if calculations verify that both short- and long-term deflection, camber, and vibration frequency and amplitude are not objectionable.

Short- and long-term deflection and camber should be computed and checked against the requirements of serviceability of the structure.

The maximum length of a slab between construction joints is generally limited to 30 to 45 m to minimize the effects of slab shortening, and to avoid excessive loss of prestress due to friction.

R18.12.4 This section provides specific guidance concerning tendon distribution that will permit the use of banded tendon distributions in one direction. This method of tendon distribution has been shown to provide satisfactory performance by structural research.

SECTION R18.13
POST-TENSIONED TENDON ANCHORAGE ZONES

R18.13.1 **Anchorage zone**. Based on the Principle of Saint-Venant, the extent of the anchorage zone may be estimated as approximately equal to the largest dimension of the cross section. Local zone and general zone are shown in Fig. R18.13.l(a). When anchorage devices located away from the end of the member are tensioned, large tensile stresses exist locally behind and ahead of the device. These tensile stresses are induced by incompatibility of deformations ahead of [as shown in Fig. R.18.13.l(b)] and behind the anchorage device. The entire shaded region should be considered, as shown in Fig. R18.13.l(b).

R18.13.2 **Local zone**. The local zone resists the very high local stresses introduced by the anchorage device and transfers them to the remainder of the anchorage zone. The behavior of the local zone is strongly influenced by the specific characteristics of the anchorage device and its confining reinforcement, and less influenced by the geometry and loading of the overall structure. Local-zone design sometimes cannot be completed until specific anchorage devices are determined at the shop drawing stage. When special anchorage devices are used, the anchorage device supplier should furnish the test information to show the device is satisfactory under AASHTO "Standard Specifications for Highway Bridges",[18.2] Division II, Article 10.3.2.3) and provide information regarding necessary conditions for use of the device. The main considerations in local-zone design are the effects of the high bearing pressure and the adequacy of any confining reinforcement provided to increase the capacity of the concrete resisting bearing stresses.

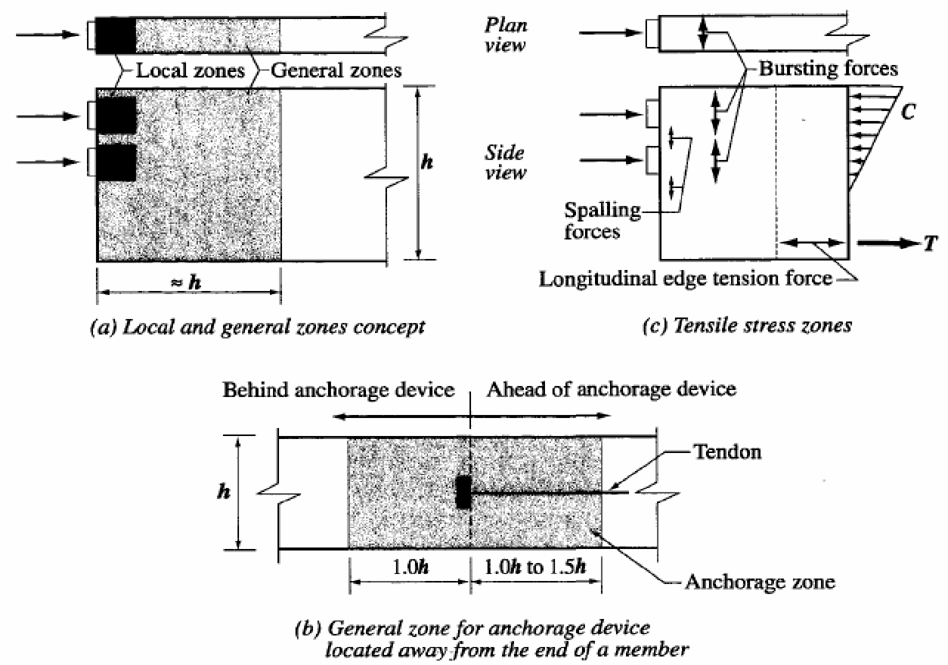


Fig.R18.13.1 - Anchorage zone

R18.13.3 **General zone.** Within the general zone the usual assumption of beam theory that plane sections remain plane is not valid.
Design should consider all regions of tensile stresses that can be caused by the tendon anchorage device, including bursting, spalling, and edge tension as shown in Fig. R18.13.l(c). Also, the compressive stresses immediately ahead [as shown in

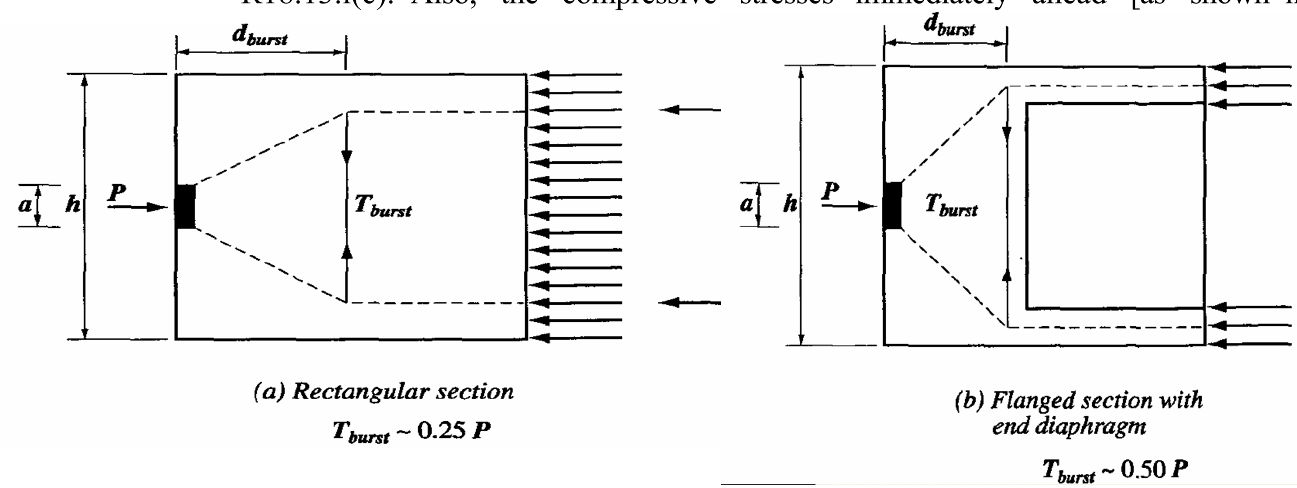


Fig.R18.13.3 - Effect of cross section change

Fig. R18.13.l(b)] of the local zone should be checked. Sometimes reinforcement requirements cannot be determined until specific tendon and anchorage device layouts are determined at the shop-drawing stage. Design and approval responsibilities should be clearly assigned in the project drawings and specifications.

Abrupt changes in section can cause substantial deviation in force paths. These deviations can greatly increase tension forces as shown in Fig. R18.13.3.

R18.13.4 **Nominal material strengths.** Some inelastic deformation of concrete is expected because anchorage zone design is based on a strength approach. The low value for the nominal compressive strength for unconfined concrete reflects this possibility. For well-confined concrete, the effective compressive strength could be increased (See Reference 18.23). The value for nominal tensile strength of bonded prestressing steel is limited to the yield strength of the prestressing steel because Eq. (18-3) may not apply to these nonflexural applications. The value for unbonded prestressing steel is based on the values of 18.7.2 (b) and (c), but is somewhat limited for these short-length, nonflexural applications. Test results given in Reference 18.23 indicate that the compressive stress introduced by auxiliary prestressing applied perpendicular to the axis of the main tendons is effective in increasing the anchorage zone capacity. The inclusion of the λ factor for lightweight concrete reflects its lower tensile strength, which is an indirect factor in limiting compressive stresses, as well as the wide scatter and brittleness exhibited in some lightweight concrete anchorage zone tests.

The designer is required to specify concrete strength at the time of stressing in the project drawings and specifications. To limit early shrinkage cracking, monostrand tendons are sometimes stressed at concrete strengths less than 20 MPa. In such cases, either oversized monostrand anchorages are used, or the strands are stressed in stages, often to levels 1/3 to 1/2 the final prestressing force.

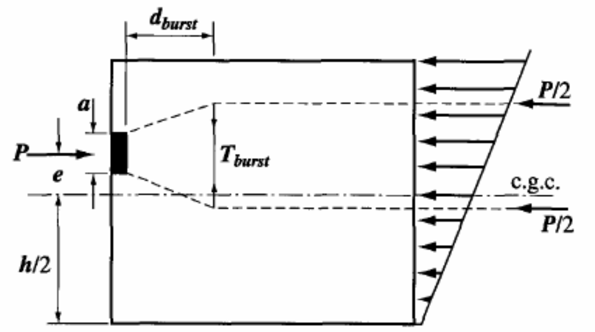


Fig. R18.13.5 - Strut-and-tie model example

R18.13.5 **Design methods.** The list of design methods in 18.13.5.1 include those procedures for which fairly specific guidelines have been given in References 18.20 and 18.21. These procedures have been shown to be conservative predictors of strength when compared to test results.[18.23] The use of strut-and-tie model is especially helpful for general zone design.[18.23] In many applications, where substantial or massive concrete regions surround the anchorages, simplified equations can be used except in the cases noted in 18.13.5.2.

For many cases, simplified equations based on References 18.20 and 18.21 can be used. Values for the magnitude of the bursting force, T_{burst}, and for its centroidal distance from the major bearing surface of the anchorage, d_{burst} may be estimated from Eq. (R18-1) and (R18-2), respectively. The terms of Eq. (R18-1) and (R18-2) are shown in Fig. R18.13.5 for a prestressing force with small eccentricity. In the applications of Eq. (R18-1) and (R18-2), the specified stressing sequence should be considered if more than one tendon is present.

$$T_{burst} = 0.25\sum P_{su}\left(1-\frac{a}{h}\right) \qquad \text{(R18-1)}$$

$$d_{burst} = 0.5(h-2e) \qquad \text{(R18-2)}$$

where:

$\sum P_{su}$ = the sum of the total factored prestressing force for the stressing arrangement considered, N;

a = the depth of anchorage device or single group of closely spaced devices in the direction considered, mm;

e = the eccentricity (always taken as positive) of the anchorage device or group of closely spaced devices with respect to the centroid of the cross section, mm;

h = the depth of the cross section in the direction considered, mm.

Anchorage devices should be treated as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage device in the direction considered.

The spalling force for tendons for which the centroid lies within the kern of the section may be estimated as 2 percent of the total factored prestressing force, except for multiple anchorage devices with center-to-center spacing greater than 0.4 times the depth of the section. For large spacings and for cases where the centroid of the tendons is located outside the kern, a detailed analysis is required. In addition, in the post-tensioning of thin sections, or flanged sections, or irregular sections, or when the tendons have appreciable curvature within the general zone, more general procedures such as those of AASHTO Articles 9.21.4 and 9.21.5 will be required. Detailed recommendations for design principles that apply to all design methods are given in Article 9.21.3.4 of Reference 18.20.

R18.13.5.3 The sequence of anchorage device stressing can have a significant effect on the general zone stresses. Therefore, it is important to consider not only the final stage of a stressing sequence with all tendons stressed, but also intermediate stages during construction. The most critical bursting forces caused by each of the sequentially post-tensioned tendon combinations, as well as that of the entire group of tendons, should be taken into account.

R18.13.5.4 The provision for three-dimensional effects was included to alert the designer to effects perpendicular to the main plane of the member, such as bursting forces in the thin direction of webs or slabs. In many cases these effects can be determined independently for each direction, but some applications require a fully three-dimensional analysis (for example diaphragms for the anchorage of external tendons).

R18.13.5.5 Where anchorages are located away from the end of a member, local tensile stresses are generated behind these anchorages [see Fig. R18.13.l(b)] due to compatibility requirements for deformations ahead of and behind the anchorages. Bonded tie-back reinforcement is required in the immediate vicinity of the anchorage to limit the extent of cracking behind the anchorage. The requirement $0.35P_{su}$ was developed using 25 percent of the unfactored prestressing force being resisted by reinforcement at $0.60f_y$.

SECTION R18.14
DESIGN OF ANCHORAGE ZONES FOR MONOSTRAND OR SINGLE 16 MM DIAMETER BAR TENDONS

R18.14.2 **General-zone design for slab tendons.** For monostrand slab tendons, the general-zone minimum reinforcement requirements are based on the recommendations of the ACI-ASCE Committee 423[18.22], which shows typical details. The horizontal bars parallel to the edge required by 18.14.2.2 should be continuous where possible.

The tests on which the recommendations of Ref. 18.24 were based were limited to anchorage devices for 12.5 mm diameter, 1860 MPa strand, unbonded tendons in normal-weight concrete. Thus, for larger strand anchorage devices and for all use in lightweight concrete slabs, the Ref. 18.22 recommended that the amount and spacing of reinforcement should be conservatively adjusted to provide for the larger anchorage force and smaller splitting tensile strength of lightweight concrete.[18.22]

Both Refs. 18.21 and 18.22 recommend that hairpin bars also be furnished for anchorages located within 300 mm of slab comers to resist edge tension forces. The words "ahead of" in 18.14.2.3 have the meaning shown in Fig. R18.13.1.

In those cases where multistrand anchorage devices are used for slab tendons, 18.15 is applicable.

The bursting reinforcement perpendicular to the plane of the slab required by 18.14.2.3 for groups of relatively closely spaced tendons should also be provided in the case of widely spaced tendons if an anchorage device failure could cause more than local damage.

R18.14.3 **General-zone design for groups of monostrand tendons in beams and girders.** Groups of monostrand tendons with individual monostrand anchorage devices are often used in beams and girders. Anchorage devices can be treated as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage device in the direction considered. If a beam or girder has a single anchorage device or a single group of closely spaced anchorage devices, the use of simplified equations such as those given in R18.13.5 is allowed, unless 18.13.5.2 governs. More complex conditions can be designed using strut-and-tie models. Detailed recommendations for use of such models are given in References 18.22 and 18.23 as well as in R18.13.5.

SECTION R18.15
DESIGN OF ANCHORAGE ZONES FOR MULTISTRAND TENDONS

R18.15.1 **Local zone design.** See R18.13.2.

R18.15.2 **Use of special anchorage devices**. Skin reinforcement is reinforcement placed near the outer faces in the anchorage zone to limit local crack width and spacing. Reinforcement in the general zone for other actions (flexure, shear, shrinkage, temperature and similar) may be used in satisfying the supplementary skin reinforcement requirement. Determination of the supplementary skin reinforcement depends on the anchorage device hardware used and frequently cannot be determined until the shop-drawing stage.

SECTION R18.16
CORROSION PROTECTION FOR UNBONDED TENDONS

R18.16.1 Suitable material for corrosion protection of unbonded prestressting steel should have the properties identified in Section 5.1 of Reference 18.23.

R18.16.2 Typically, sheathing is a continuous, seamless, high-density polythylene material that is extruded directly onto the coated prestressing steel.

SECTION R18.17
POST-TENSIONING DUCTS

R18.17.4 Water in ducts may cause distress to the surrounding concrete upon freezing. When strands are present, ponded water in ducts should also be avoided. A corrosion inhibitor should be used to provide temporary corrosion protection if prestressing steel is exposed to prolonged periods of moisture in the ducts before grouting.[18.24]

SECTION R18.18
GROUT FOR BONDED TENDONS

Proper grout and grouting procedures are critical to post-tensioned construction.[18.25, 18.26] Grout provides bond. between the prestressing steel and the duct, and provides corrosion protection to the prestressing steel

R18.18.2 The limitations on admixtures in 3.6 apply to grout. Substances known to be harmful to tendons, grout, or concrete are chlorides, fluorides, sulfites, and nitrates. Aluminum powder or other expansive admixtures, when approved, should produce an unconfined expansion of 5 to 10 percent. Neat cement grout is used in almost all building construction. Use of finely graded sand in the grout should only be considered with large ducts having large void areas.

R18.18.3 **Selection of grout proportions.** Grout proportioned in accordance with these provisions will generally lead to 7 day compressive strength on standard 50 mm cubes in excess of 18 MPa and 28 day strengths of about 30 MPa. The handling and placing properties of grout are usually given more consideration than strength when designing grout mixtures.

R18.18.4 **Mixing and pumping grout**. In an ambient temperature of 2^{o}C, grout with an initial minimum temperature of 15^{o}C may require as much as 5 days to reach 6 MPa. A minimum grout temperature of 15^{o}C is suggested because it is consistent with the recommended minimum temperature for concrete placed at an ambient temperature of 2^{o}C. Quickset grouts, when approved, may require shorter periods of protection and the recommendations of the suppliers should be followed. Test cubes should be cured under temperature and moisture conditions as close as possible to those of the grout in the member. Grout temperatures in excess of 30^{o}C will lead to difficulties in pumping.

SECTION R18.20
APPLICATION AND MEASUREMENT OF PRESTRESSING FORCE

R18.20.1 Elongation measurements for prestressed elements should be in accordance with the procedures outlined in the "Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products," published by the Precast Prestressed Concrete Institute.[18.27]

Elongation measurements for post-tensioned construction are affected by several factors that are less significant, or that do not exist, for pretensioned elements. The friction along prestressing steel in post-tensioning applications may be affected to varying degrees by placing tolerances and small irregularities in tendon profile due to concrete placement. The friction coefficients between the pre-stressing steel and the duct are also subject to variation.

R18.20.4 This provision applies to all prestressed concrete members. For cast-in-place post-tensioned slab systems, a member should be that portion considered as an element in the design, such as the joist and effective slab width in one-way joist systems, or the column strip or middle strip in two-way flat plate systems.

SECTION R18.21
POST-TENSIONING ANCHORAGES AND COUPLERS

R18.21.1 The prestressing steel material should comply with the minimum provisions of the applicable ASTM specifications as outlined in 3.5.5. The specified strength of anchorages and couplers exceeds the maximum design strength of the prestressing steel by a substantial margin, and, at the same time, recognizes the stress-riser effects associated with most available post-tensioning anchorages and couplers. Anchorage and coupler strength should be attained with a minimum amount of permanent deformation and successive set, recognizing that some deformation and set will occur when testing to failure. Tendon assemblies should conform to the 2 percent elongation requirements in Ref 18.28 and industry recommendation.[18.14] Anchorages and couplers for bonded tendons that develop less than 100 percent of the specified breaking strength of the prestressing steel should be used only where the bond transfer length between the anchorage or coupler and critical sections equals or exceeds that required to develop the prestressing steel strength. This bond length may be calculated by the results of tests of bond characteristics of untensioned prestressing strand [18.29] or by bond tests on other prestressing steel materials, as appropriate.

R18.21.3 For discussion on fatigue loading, see Reference 18.30.

For detailed recommendations on tests for static and cyclic loading conditions for tendons and anchorage fittings of unbonded tendons, see Section 4.1.3 of Reference 18.9, and Section 15.2.2 of Reference 18.28.

R18.21.4 For recommendations regarding protection see Sections 4.2 and 4.3 of Reference 18.9, and also Reference 18.23.

SECTION R18.22
EXTERNAL POST-TENSIONING

External attachment of tendons is a versatile method of providing additional strength, or improving serviceability, or both, in existing structures. It is well suited to repair or upgrade existing structures and permits a wide variety of tendon arrangements.

Additional information on external post-tensioning is given in Reference 18.31.

R18.22.3 External tendons are often attached to the concrete member at various locations between anchorages (such as midspan, quarter points, or third points) for desired load balancing effects, for tendon alignment, or to address tendon vibration concerns. Consideration should be given to the effects caused by the tendon profile shifting in relationship to the concrete centroid as the member deforms under effects of post-tensioning and applied load.

R18.22.4 Permanent corrosion protection can be achieved by a variety of methods. The corrosion protection provided should be suitable to the environment in which the tendons are located. Some conditions will require that the prestressing steel be protected by concrete cover or by cement grout in polyethylene or metal tubing; other conditions will permit the protection provided by coatings such as paint or grease. Corrosion protection methods should meet the fire protection requirements of the general building SBC 304, unless the installation of external post-tensioning is to only improve serviceability.

CHAPTER 19
SHELLS AND FOLDED PLATE MEMBERS

SECTION R19.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

SECTION R19.1
SCOPE AND DEFINITIONS

The SBC 304 and commentary provide information on the design, analysis, and construction of concrete thin shells and folded plates. Additional information may be found in Refs. 19.1 and 19.2.

Since Chapter 19 applies to concrete thin shells and folded plates of all shapes, extensive discussion of their design, analysis, and construction in the commentary is not possible. Additional information can be obtained from the references. Performance of shells and folded plates requires special attention to detail.[19.3]

R19.1.1 Discussion of the application of thin shells in special structures such as cooling towers and circular prestressed concrete tanks may be found in Refs. 19.4 and 19.5.

R19.1.3 Common types of thin shells are domes (surfaces of revolution),[19.6,19.7] cylindrical shell,[19.7] barrel vault,[19.8] conoids,[19.8] elliptical paraboloid,[19.8] hyperbolic paraboloid,[19.9] and groined vaults.[19.9]

R19.1.4 Folded plates may be prismatic,[19.6,19.7] nonprismatic[19.7] or faceted. The first two types consist generally of planar thin slabs joined along their longitudinal edges to form a beam-like structure spanning between supports. Faceted folded plates are made up of triangular or polygonal planar thin slabs joined along their edges to form three-dimensional spatial structures.

R19.1.5 Ribbed shell[19.8,19.9] generally have been used for larger spans where the increased thickness of the curved slab alone becomes excessive or uneconomical. Ribbed shells are also used because of the construction techniques employed and to enhance the aesthetic impact of the completed structure.

R19.1.6 Most thin shell structures require ribs or edge beams at their boundaries to carry the shell boundary forces, to assist in transmitting them to the supporting structure, and to accommodate the increased amount of reinforcement in these areas.

R19.1.7 Elastic analysis of thin shells and folded plates can be performed using any method of structural analysis based on assumptions that provide suitable approximations to the three-dimensional behavior of the structure. The method should determine the internal forces and displacements needed in the design of the shell proper, the rib or edge members, and the supporting structure.

Equilibrium of internal forces and external loads and compatibility of deformations should be satisfied.

Methods of elastic analysis based on classical shell theory, simplified mathematical or analytical models, or numerical solutions using finite element[19.10] finite difference[19.8] or numerical integration technique[19.8,19.11] are described in the cited references.

The choice of the method of analysis and the degree of accuracy required depends on certain critical factors. These include: the size of the structure, the geometry of the thin shell or folded plate, the manner in which the structure is supported, the nature of the applied load, and the extent of personal or documented experience regarding the reliability of the given method of analysis in predicting the behavior of the specific type of shell[19.8] or folded plate.[19.7]

R19.1.8 Inelastic analysis of thin shells and folded plates can be performed using a refined method of analysis based on the specific nonlinear material properties, nonlinear behavior due to the cracking of concrete, and time-dependent effects such as creep, shrinkage, temperature, and load history. These effects are incorporated in order to trace the response and crack propagation of a reinforced concrete shell through the elastic, inelastic, and ultimate ranges. Such analyses usually require incremental loading and iterative procedures to converge on solutions that satisfy both equilibrium and strain compatibility.[19.12,19.13]

SECTION R19.2
ANALYSIS AND DESIGN

R19.2.1 For types of shell structures where experience, tests, and analyses have shown that the structure can sustain reasonable overloads without undergoing brittle ailure, elastic analysis is an acceptable procedure. The designer may assume that reinforced concrete is ideally elastic, homogeneous, and isotropic, having identical properties in all directions. An analysis should be performed for the shell considering service load conditions. The analysis of shells of unusual size, shape, or complexity should consider behavior through the elastic, cracking, and inelastic stages.

R19.2.2 Several inelastic analysis procedures contain possible solution methods.[19.12,19.13]

R19.2.4 Experimental analysis of elastic model[19.14] has been used as a substitute for an analytical solution of a complex shell structure. Experimental analysis of reinforced micro concrete models through the elastic, cracking, inelastic, and ultimate stages should be considered for important shells of unusual size, shape, or complexity.

For model analysis, only those portions of the structure that significantly affect the items under study need be simulated. Every attempt should be made to ensure that the experiments reveal the quantitative behavior of the prototype structure.

Wind tunnel tests of a scaled-down model do not necessarily provide usable results and should be conducted by a recognized expert in wind tunnel testing of structural models.

R19.2.5 Solutions that include both membrane and bending effects and satisfy conditions of compatibility and equilibrium are encouraged. Approximate solutions that satisfy statics but not the compatibility of strains may be used only when extensive experience has proved that safe designs have resulted from their use. Such methods include beam-type analysis for barrel shells and folded plates having large ratios of span to either width or radius of curvature, simple membrane analysis for shells of revolution, and others in which the equations of equilibrium are satisfied, while the strain compatibility equations are not.

R19.2.6 If the shell is prestressed, the analysis should include its strength at factored loads as well as its adequacy under service loads, under the load that causes cracking, and under loads induced during prestressing. Axial forces due to draped tendons may not lie in one plane and due consideration should be given to the resulting force components. The effects of post-tensioning of shell supporting members should be taken into account.

R19.2.7 The thin shell's thickness and reinforcement are required to be proportioned to satisfy the strength provisions of this SBC 304, and to resist internal forces obtained from an analysis, an experimental model study, or a combination thereof. Reinforcement sufficient to minimize cracking under service load conditions should be provided. The thickness of the shell is often dictated by the required reinforcement and the construction constraints, by 19.2.8, or by the SBC 304 minimum thickness requirements.

R19.2.8 Thin shells, like other structures that experience in-plane membrane compressive forces, are subject to buckling when the applied load reaches a critical value. Because of the surface-like geometry of shells, the problem of calculating buckling load is complex. If one of the principal membrane forces is tensile, the shell is less likely to buckle than if both principal membrane forces are compressive. The kinds of membrane forces that develop in a shell depend on its initial shape and the manner in which the shell is supported and loaded. In some types of shells, post-buckling behavior should be considered in determining safety against instability.[19.2]

Investigation of thin shells for stability should consider the effect of (1) anticipated deviation of the geometry of the shell surface as-built from the idealized, geometry, (2) large deflections, (3) creep and shrinkage of concrete, (4) inelastic properties of materials, (5) cracking of concrete, (6) location, amount, and orientation of reinforcement, and (7) possible deformation of supporting elements.

Measures successfully used to improve resistance to buckling include the provision of two mats of reinforcement-one near each outer surface of the shell, a local increase of shell curvatures, the use of ribbed shells, and the use of concrete with high tensile strength and low creep.

A procedure for determining critical buckling loads of shells is given in the recommendation of Ref. 19.2. Some recommendations for buckling design of domes used in industrial applications are given in Refs.19.5 and 19.15.

R19.2.10 The stresses and strains in the shell slab used for design are those determined by analysis (elastic or inelastic) multiplied by appropriate load factors. Because of detrimental effects of membrane cracking, the computed tensile strain in the reinforcement under factored loads should be limited.

R19.2.11 When principal tensile stress produces membrane cracking in the shell, experiments indicate the attainable compressive strength in the direction parallel to the cracks is reduced.[19.16,19.17]

SECTION R19.4
SHELL REINFORCEMENT

R19.4.1 At any point in a shell, two different kinds of internal forces may occur simultaneously: those associated with membrane action, and those associated with bending of the shell. The membrane forces are assumed to act in the tangential plane midway between the surfaces of the shell, and are the two axial forces and the membrane shears. Flexural effects include bending moments, twisting moments, and the associated transverse shears. Limiting membrane crack width and spacing due to shrinkage, temperature, and service load conditions is a major design consideration.

R19.4.2 The requirement of ensuring strength in all directions is based on safety considerations. Any method that ensures sufficient strength consistent with equilibrium is acceptable. The direction of the principal membrane tensile force at any point may vary depending on the direction, magnitudes, and combinations of the various applied loads.

The magnitude of the internal membrane forces, acting at any point due to a specific load, is generally calculated on the basis of an elastic theory in which the shell is assumed as uncracked. The computation of the required amount of reinforcement to resist the internal membrane forces has been traditionally based on the assumption that concrete does not resist tension. The associated deflections, and the possibility of cracking, should be investigated in the serviceability phase of the design. Achieving this may require a working stress design for steel selection.

Where reinforcement is not placed in the direction of the principal tensile forces and where cracks at the service load level are objectionable, the computation of reinforcement may have to be based on a more refined approach[19.16,19.18,19.19] that considers the existence of cracks. In the cracked state, the concrete is assumed to be unable to resist either tension or shear. Thus, equilibrium is attained by equating tensile resisting forces in reinforcement and compressive resisting forces in concrete.

The alternative method to calculate orthogonal reinforcement is the shear-friction method. It is based on the assumption that shear integrity of a shell should be maintained at factored loads. It is not necessary to calculate principal stresses if the alternative approach is used.

R19.4.3 Minimum membrane reinforcement corresponding to slab shrinkage and temperature reinforcement are to be provided in at least two approximately orthogonal directions even if the calculated membrane forces are compressive in one or more directions.

R19.4.5 The requirement that the tensile reinforcement yields before the concrete crushes anywhere is consistent with 10.3.3. Such crushing can also occur in regions near supports and for some shells where the principal membrane forces are approximately equal and opposite in sign.

R19.4.6 Generally, for all shells, and particularly in regions of substantial tension, the orientation of reinforcement should approximate the directions of the principal tensile membrane forces. However, in some structures it is not possible to detail the reinforcement to follow the stress trajectories. For such cases, orthogonal component reinforcement is allowed.

R19.4.7 When the directions of reinforcement deviate significantly (more than 10 deg) from the directions of the principal membrane forces, higher strains in the shell occur to develop the capacity of reinforcement. This might lead to the development of unacceptable wide cracks. The crack width should be estimated and limited if necessary.

Permissible crack widths for service loads under different environmental conditions are given in Ref. 19.20. Crack width can be limited by an increase in the amount of reinforcement used, by reducing the stress at the service load level, by providing reinforcement in three or more directions in the plane of the shell, or by using closer spacing of smaller diameter bars.

R19.4.8 The practice of concentrating tensile reinforcement in the regions of maximum tensile stress has led to a number of successful and economical designs, primarily for long folded plates, long barrel vault shells, and for domes. The requirement of providing the minimum reinforcement in the remaining tensile zone is intended to limit crack width and spacing.

R19.4.9 The design method should ensure that the concrete sections, including consideration of the reinforcement, are capable of developing the internal forces required by the equations of equilibrium.[19.21] The sign of bending moments may change rapidly from point to point of a shell. For this reason, reinforcement to resist bending, where required, is to be placed near both outer surfaces of the shell. In many cases, the thickness required to provide proper cover and spacing for the multiple layers of reinforcement may govern the design of the shell thickness.

R19.4.10 The value of ϕ to be used is that prescribed in 9.3.2.1 for axial tension.

R19.4.11 and
R19.4.12 On curved shell surfaces it is difficult to control the alignment of precut reinforcement. This should be considered to avoid insufficient splice and development lengths. Sections 19.4.11 and 19.4.12 require extra reinforcement length to maintain the minimum lengths on curved surfaces.

SECTION R19.5
CONSTRUCTION

R19.5.1 When early removal of forms is necessary, the magnitude of the modulus of elasticity at the time of proposed form removal should be investigated to ensure safety of the shell with respect to buckling, and to restrict deflections.[19.3,19.22] The value of the modulus of elasticity E_c should be obtained from a flexural test of field-cured specimens. It is not sufficient to determine the modulus from the formula in 8.5.1, even if f_c' is determined for the field-cured specimen.

R19.5.2 In some types of shells, small local deviations from the theoretical geometry of the shell can cause relatively large changes in local stresses and in overall safety against instability. These changes can result in local cracking and yielding that may make the structure unsafe or can greatly affect the critical load producing instability. The effect of such deviations should be evaluated and any necessary remedial actions should be taken. Special attention is needed when using air supported form systems.[19.23]

CHAPTER 20
STRENGTH EVALUATION OF EXISTING STRUCTURES

SECTION R20.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

SECTION R20.1
STRENGTH EVALUATION – GENERAL

Chapter 20 does not cover load testing for the approval of new design or construction methods. (See 16.10 for recommendations on strength evaluation of precast concrete members.) Provisions of Chapter 20 may be used to evaluate whether a structure or a portion of a structure satisfies the safety requirements of the SBC. A strength evaluation may be required if the materials are considered to be deficient in quality, if there is evidence indicating faulty construction, if a structure has deteriorated, if a building will be used for a new function, or if, for any reason, a structure or a portion of it does not appear to satisfy the requirements of the SBC 304. In such cases, Chapter 20 provides guidance for investigating the safety of the structure.

If the safety concerns are related to an assembly of elements or an entire structure, it is not feasible to load test every element and section to the maximum. In such cases, it is appropriate that an investigation plan be developed to address the specific safety concerns. If a load test is described as part of the strength evaluation process, it is desirable for all parties involved to come to an agreement about the region to be loaded, the magnitude of the load, the load test procedure, and acceptance criteria before any load tests are conducted.

R20.1.2 Strength considerations related to axial load, flexure, and combined axial load and flexure are well understood. There are reliable theories relating strength and short-term displacement to load in terms of dimensional and material data for the structure.

To determine the strength of the structure by analysis, calculations should be based on data gathered on the actual dimensions of the structure, properties of the materials in place, and all pertinent details. Requirements for data collection are in 20.2.

R20.1.3 If the shear or bond strength of an element is critical in relation to the doubt expressed about safety, a test may be the most efficient solution to eliminate or confirm the doubt. A test may also be appropriate if it is not feasible to determine the material and dimensional properties required for analysis, even if the cause of the concern relates to flexure or axial load.

Wherever possible and appropriate, support the results of the load test by analysis.

R20.1.4 For a deteriorating structure, the acceptance provided by the load test may not be assumed to be without limits in terms of time. In such cases, a periodic inspection program is useful. A program that involves physical tests and periodic inspection can justify a longer period in service. Another option for maintaining the structure in service, while the periodic inspection program continues, is to limit the live load to a level determined to be appropriate.

The length of the specified time period should be based on consideration of (a) the nature of the problem, (b) environmental and load effects, (c) service history of the structure, and (d) scope of the periodic inspection program. At the end of a specified time period, further strength evaluation is required if the structure is to remain in service.

With the agreement of all concerned parties, special procedures may be devised for periodic testing that do not necessarily conform to the loading and acceptance criteria specified in Chapter 20 of SBC 304.

SECTION R20.2
DETERMINATION OF REQUIRED DIMENSIONS AND MATERIAL PROPERTIES

This section applies if it is decided to make an analytical evaluation (see 20.1.2).

R20.2.1 Critical sections are where each type of stress calculated for the load in question reaches its maximum value.

R20.2.2 For individual elements, amount, size, arrangement, and location should be determined at the critical sections for reinforcement or tendons, or both, designed to resist applied load. Nondestructive investigation methods are acceptable. In large structures, determination of these data for approximately 5 percent of the reinforcement or tendons in critical regions may suffice if these measurements confirm the data provided in the construction drawings.

R20.2.3 The number of tests may depend on the size of the structure and the sensitivity of structural safety to concrete strength. In cases where the potential problem involves flexure only, investigation of concrete strength can be minimal for a lightly reinforced section ($\rho f_y / f_c' \leq 0.15$ for rectangular section).

R20.2.4 The number of tests required depends on the uniformity of the material and is best determined by the engineer for the specific application.

SECTION R20.3
LOAD TEST PROCEDURE

R20.3.1 **Load arrangement**. It is important to apply the load at locations so that its effects on the suspected defect are a maximum and the probability of unloaded members sharing the applied load is a minimum. In cases where it is shown by analysis that adjoining unloaded elements will help carry some of the load, the load should be placed to develop effects consistent with the intent of the load factor.

R20.3.2 **Load intensity.** The required load intensity follows previous load test practice. The live load L may be reduced as permitted by SBC 301 governing safety considerations for the structure. The live load should be increased to compensate for resistance provided by unloaded portions of the structure in questions. The increase in live load is determined from analysis of the loading conditions in relation to the selected pass/fail criterion for the test.

The test load intensity remained the same. It is considered appropriate for designs using the load combinations and strength reduction factors of Chapter 9.

SECTION R20.4
LOADING CRITERIA

R20.4.2 Inspecting the structure after each load increment is advisable.

R20.4.3 Arching refers to the tendency for the load to be transmitted nonuniformly to the flexural element being tested. For example, if a slab is loaded by a uniform arrangement of bricks with the bricks in contact, arching would results in reduction of the load on the slab near the midspan of the slab.

SECTION R20.5
ACCEPTANCE CRITERIA

R20.5.1 A general acceptance criterion for the behavior of a structure under the test load is that it does not show evidence of failure. Evidence of failure includes cracking, spalling, or deflection of such magnitude and extent that the observed result is obviously excessive and incompatible with the safety requirements of the structure. No simple rules have been developed for application to all types of structures and conditions. If sufficient damage has occurred so that the structure is considered to have failed that test, retesting is not permitted since it is considered that damaged members should not be put into service even at a lower load rating.

Local spalling or flaking of the compressed concrete in flexural elements related to casting imperfections need not indicate overall structural distress. Crack widths are good indicators of the state of the structure and should be observed to help determine whether the structure is satisfactory. However, exact prediction or measurement of crack widths in reinforced concrete elements is not likely to be achieved under field conditions. Establish criteria before the test, relative to the types of cracks anticipated; where the cracks will be measured; how they will be measured; and approximate limits or criteria to evaluate new cracks or limits for the changes in crack width.

R20.5.2 The deflection limits and the retest option follow past practice. If the structure shows no evidence of failure, recovery of deflection after removal of the test load is used to determine whether the strength of the structure is satisfactory. In the case of a very stiff structure, however, the errors in measurements under field conditions may be of the same order as the actual deflections and recovery. To avoid penalizing a satisfactory structure in such a case, recovery measurements are waived if the maximum deflection is less than $\ell_t^2/(20{,}000h)$. The residual deflection $\Delta_{r\max}$ is the difference between the initial and final (after load removal) deflections for the load test or the repeat load test.

R20.5.3 Forces are transmitted across a shear crack plane by a combination of aggregate interlock at the interface of the crack that is enhanced by clamping action of transverse stirrup reinforcing and by dowel action of stirrups crossing the crack. As crack lengths increase to approach a horizontal projected length equal to the depth of the member and concurrently widen to the extent that aggregate interlock cannot occur, and as transverse stirrups if present begin to yield or display loss of anchorage so as to threaten their integrity, the member is assumed to be approaching imminent shear failure.

R20.5.4 The intent of 20.5.4 is to make the professionals in charge of the test pay attention to the structural implication of observed inclined cracks that may lead to brittle collapse in members without transverse reinforcement.

R20.5.5 Cracking along the axis of the reinforcement in anchorage zones may be related to high stresses associated with the transfer of forces between the reinforcement and the concrete. These cracks may be indicators of pending brittle failure of the element if they are associated with the main reinforcement. It is important that their causes and consequences be evaluated.

SECTION R20.6
PROVISION FOR LOWER LOAD RATING

Except for load tested members that have failed under a test (see 20.5), the building official may permit the use of a structure or member at a lower load rating that is judged to be safe and appropriate on the basis of the test results.

CHAPTER 21
SPECIAL PROVISIONS FOR SEISMIC DESIGN

SECTION R21.0
NOTATION

Units of measurement are given in the Notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as m or kN.

SECTION R21.1
DEFINITION

The design displacement is an index of the maximum lateral displacement expected in design for the design-basis earthquake. In References 21.1 through 21.4, the design-basis earthquake has approximately a 90 percent probability of nonexceedance in 50 years. In those documents, the design displacement is calculated using static or dynamic linear elastic analysis under SBC 304 specified actions considering effects of cracked sections, effects of torsion, effects of vertical forces acting through lateral displacements, and modification factors to account for expected inelastic response. The design displacement generally is larger than the displacement calculated from design-level forces applied to a linear-elastic model of the building.

The provisions of 21.6 are intended to result in a special moment frame constructed using precast concrete having minimum strength and toughness equivalent to that for a special moment frame of cast-in-place concrete.

The provisions of 21.13 are intended to result in an intermediate precast structural wall having minimum strength and toughness equivalent to that for an ordinary reinforced concrete structural wall of cast-in-place concrete. A precast concrete wall satisfying only the requirements of Chapters 1 through 18 and not the additional requirements of 21.13 or 21.8 is considered to have ductility and structural integrity less than that for an intermediate precast structural wall.

The provisions of 21.8 are intended to result in a special precast structural wall having minimum strength and toughness equivalent to that for a special reinforced concrete structural wall of cast-in-place concrete.

SECTION R21.2
GENERAL REQUIREMENTS

R21.2.1 **Scope**. Chapter 21 contains provisions considered to be the minimum requirements for a cast-in-place or precast concrete structure capable of sustaining a series of oscillations into the inelastic range of response without critical deterioration in strength. The integrity of the structure in the inelastic range of response should be maintained because the design forces defined in references 21.1, 21.5 and 21.6 provisions are considered less than those corresponding to linear response at the anticipated earthquake intensity.[21.1,21.6-21.8]

As a properly detailed cast-in-place or precast concrete structure responds to strong ground motion, its effective stiffness decreases and its energy dissipation increases. These changes tend to reduce the response accelerations and lateral inertia forces relative to values that would occur were the structure to remain linearly elastic and lightly damped[21.8]. Thus, the use of design forces representing earthquake effects such as those in Reference 21.2 requires that the lateral-force resisting system retain a substantial portion of its strength into the inelastic range under displacement reversals.

The provisions of Chapter 21 relate detailing requirements to type of structural framing, earthquake risk level at the site, level of inelastic deformation intended in structural design, and use and occupancy of the structure. Earthquake risk levels traditionally have been classified as low, moderate, and high. The seismic risk level of a region or the seismic performance or design category of a structure is regulated by provisions of SBC 301.

The design and detailing requirements should be compatible with the level of energy dissipation (or toughness) assumed in the computation of the design seismic loads. The terms ordinary, intermediate, and special are specifically used to facilitate this compatibility. The degree of required toughness, and therefore the level of required detailing, increases for structures progressing from ordinary through intermediate to special categories. It is essential that structures in higher seismic zones or assigned to higher seismic performance or design categories possess a higher degree of toughness. It is permitted, however, to design for higher toughness in the lower seismic zones or design categories and take advantage of the lower design force levels.

The provisions of Chapters 1 through 18 are intended to provide adequate toughness for structures in regions of low seismic risk, or assigned to ordinary categories. Therefore, it is not required to apply the provisions of Chapter 21 to lateral-force resisting systems consisting of ordinary structural walls.

Chapter 2 1 requires special details for reinforced concrete structures in regions of moderate seismic risk, or assigned to intermediate seismic performance or design categories. These requirements are contained in 21.2.1.3, 21.12, and 21.13. Although new provisions are provided in 21.13 for design of intermediate precast structural walls, general building codes that address seismic performance or design categories currently do not include intermediate structural walls.

Structures in regions of high seismic risk, or assigned to high seismic performance or design categories, may be subjected to strong ground shaking. Structures designed using seismic forces based upon response modification factors for special moment frames or special reinforced concrete structural walls are likely to experience multiple cycles of lateral displacements well beyond the point where reinforcement yields should the design earthquake ground shaking occur. The provisions of 2 1.2 through 2 1.11 have been developed to provide the structure with adequate toughness for this special response.

The requirements of Chapter 21 as they apply to various components of structures in regions of intermediate or high seismic risk, or assigned to intermediate or high seismic performance or design categories, are summarized in Table R21.2.1

The special proportioning and detailing requirements in Chapter 21 are based predominantly on field and laboratory experience with monolithic reinforced concrete building structures and precast concrete building structures designed and detailed to behave like monolithic building structures. Extrapolation of these requirements to other types of cast-in-place or precast concrete structures should be based on evidence provided by field experience, tests, or analysis. ACI T1.1-01, "Acceptance Criteria for Moment Frames Based on Structural Testing," can be used in conjunction with Chapter 21 to demonstrate that the strength and toughness of a proposed frame system equals or exceeds that provided by a comparable monolithic concrete system.

The toughness requirements in 21.2.1.5 refer to the concern for the structural integrity of the entire lateral-force resisting system at lateral displacements anticipated for ground motions corresponding to the design earthquake. Depending on the energy-dissipation characteristics of the structural system used, such displacements may be larger than for a monolithic reinforced concrete structure.

TABLE R21.2.1- SECTIONS OF CHAPTER 21 TO BE SATISFIED*

Component resisting earthquake effect, unless otherwise noted	**Level of seismic risk or assigned seismic performance or design categories (as defined in SBC 304 section)**	
	Intermediate (21.2.1.3)	**High (21.2.1.4)**
Frame members	21.12	21.2, 21.3, 21.4, 21.5
Structural walls and coupling beams	None	21.2, 21.7
Precast structural walls	21.13	21.2, 21.8
Structural diaphragms and trusses	None	21.2, 21.9
Foundations	None	21.2, 21.10
Frame members not proportioned to resist forces induced by earthquake motions	None	21.11

*In addition to requirements of Chapters 1 through 18 for structures at intermediate seismic risk (21.2.1.3), and for Chapters 1 through 17 for structures at high seismic risk (21.2.1.4).

R21.2.2 **Analysis and proportioning of structural members**. It is assumed that the distribution of required strength to the various components of a lateral-force resisting system will be guided by the analysis of a linearly elastic model of the system acted upon by the factored forces required by the governing SBC 304. If nonlinear response history analyses are to be used, base motions should be selected after a detailed study of the site conditions and local seismic history.

Because the design basis admits nonlinear response, it is necessary to investigate the stability of the lateral-force resisting system as well as its interaction with other structural and nonstructural members at displacements larger than those indicated by linear analysis. To handle this without having to resort to nonlinear response analysis, one option is to multiply by a factor of at least two the displacements from linear analysis by using the factored lateral forces, unless the governing SBC 304 specifies the factors to be used as in References 21.1 and

21.2. For lateral displacement calculations, assuming all the horizontal structural members to be fully cracked is likely to lead to better estimates of the possible drift than using uncracked stiffness for all members.

The main concern of Chapter 21 is the safety of the structure. The intent of 21.2.2.1 and 21.2.2.2 is to draw attention to the influence of nonstructural members on structural response and to hazards from falling objects.

Section 21.2.2.3 alerts the designer that the base of the structure as defined in analysis may not necessarily correspond to the foundation or ground level.

In selecting member sizes for earthquake-resistant structures, it is important to consider problems related to congestion of reinforcement. The designer should ensure that all reinforcement can be assembled and placed and that concrete can be cast and consolidated properly. Use of upper limits of reinforcement ratios permitted is likely to lead to insurmountable construction problems especially at frame joints.

R21.2.4 **Concrete in members resisting earthquake-induced forces.** Requirements of this section refer to concrete quality in frames, trusses, or walls proportioned to resist earthquake-induced forces. The maximum design compressive strength of lightweight aggregate concrete to be used in structural design calculations is limited to 35 MPa, primarily because of paucity of experimental and field data on the behavior of members made with lightweight aggregate concrete subjected to displacement reversals in the nonlinear range. If convincing evidence is developed for a specific application, the limit on maximum compressive strength of lightweight aggregate concrete may be increased to a level justified by the evidence.

R21.2.5 **Reinforcement in members resisting earthquake-induced forces**. Use of longitudinal reinforcement with strength substantially higher than that assumed in design will lead to higher shear and bond stresses at the time of development of yield moments. These conditions may lead to brittle failures in shear or bond and should be avoided even if such failures may occur at higher loads than those anticipated in design. Therefore, a ceiling is placed on the actual yield strength of the steel [see 21.2.5(a)].

The requirement for an ultimate tensile strength larger than the yield strength of the reinforcement [21.2.5(b)] is based on the assumption that the capability of a structural member to develop inelastic rotation capacity is a function of the length of the yield region along the axis of the member. In interpreting experimental results, the length of the yield region has been related to the relative magnitudes of ultimate and yield moments.[21.10] According to this interpretation, the larger the ratio of ultimate to yield moment, the longer the yield region. Chapter 21 requires that the ratio of actual tensile strength to actual yield strength is not less than 1.25. Members with reinforcement not satisfying this condition can also develop inelastic rotation, but their behavior is sufficiently different to exclude them from direct consideration on the basis of rules derived from experience with members reinforced with strain-hardening steel.

R21.2.6 **Mechanical splices**. In a structure undergoing inelastic deformations during an earthquake, the tensile stresses in reinforcement may approach the tensile strength of the reinforcement. The requirements for Type 2 mechanical splices are intended to avoid a splice failure when the reinforcement is subjected to expected stress levels in yielding regions. Type 1 splices are not required to satisfy the more stringent requirements for Type 2 splices, and may not be capable of resisting the stress levels expected in yielding regions. The locations of Type 1 splices are restricted because tensile stresses in rein- forcement in yielding regions can exceed the strength requirements of 3.5.2 and 12.14.3.3.

Recommended detailing practice would preclude the use of splices in regions of potential yield in members resisting earthquake effects. If use of mechanical splices in regions of potential yielding cannot be avoided, the designer should have documentation on the actual strength characteristics of the bars to be spliced, on the force-deformation characteristics of the spliced bar, and on the ability of the Type 2 splice to be used to meet the specified performance requirements.

R21.2.7 Welded splices

R21.2.7.1 Welding of reinforcement should be according to ANSI/AWS D1.4 as required in Chapter 3. The locations of welded splices are restricted because reinforcement tension stresses in yielding regions can exceed the strength requirements of 12.14.3.4.

R21.2.7.2 Welding of crossing reinforcing bars can lead to local embrittlement of the steel. If welding of crossing bars is used to facilitate fabrication or placement of reinforcement, it should be done only on bars added for such purposes. The prohibition of welding crossing reinforcing bars does not apply to bars that are welded with welding operations under continuous, competent control as in the manufacture of welded wire fabric.

SECTION R21.3
FLEXURAL MEMBERS OF SPECIAL MOMENT FRAMES

R21.3.1 **Scope**. This section refers to beams of special moment frames resisting lateral loads induced by earthquake motions. Any frame member subjected to a factored axial compressive force exceeding $\left(A_g f_c^{'}/10\right)$ is to be proportioned and detailed as described in 21.4.

Experimental evidence[21.11] indicates that, under reversals of displacement into the nonlinear range, behavior of continuous members having length-to-depth ratios of less than four is significantly different from the behavior of relatively slender members. Design rules derived from experience with relatively slender members do not apply directly to members with length-to-depth ratios less than four, especially with respect to shear strength.

Geometric constraints indicated in 21.3.1.3 and 21.3.1.4 were derived from practice with reinforced concrete frames resisting earthquake-induced forces[21.12].

R21.3.2 **Longitudinal reinforcement.** Section 10.3.5 limits the net tensile strain, ε_t, thereby indirectly limiting the tensile reinforcement ratio in a flexural member to a fraction of the amount that would produce balanced conditions. For a section subjected to bending only and loaded monotonically to yielding, this approach is feasible because the likelihood of compressive failure can be estimated reliably with the behavioral model assumed for determining the reinforcement ratio corresponding to balanced failure. The same behavioral model (because of incorrect assumptions such as linear strain distribution, well-defined yield point for the steel, limiting compressive strain in the concrete of 0.003, and compressive stresses in the shell concrete) fails to describe the conditions in a flexural member subjected to reversals of displacements well into the inelastic range. Thus, there is little rationale for continuing to refer to balanced conditions in earthquake-resistant design of reinforced concrete structures.

R21.3.2.1 The limiting reinforcement ratio of 0.02 is based primarily on considerations of steel congestion and, indirectly, on limiting shear stresses in girders of typical proportions. The requirement of at least two bars, top and bottom, refers again to construction rather than behavioral requirements.

R21.3.2.3 Lap splices of reinforcement are prohibited at regions where flexural yielding is anticipated because such splices are not reliable under conditions of cyclic loading into the inelastic range. Transverse reinforcement for lap splices at any location is mandatory because of the likelihood of loss of shell concrete.

R21.3.3 **Transverse reinforcement.** Transverse reinforcement is required primarily to confine the concrete and maintain lateral support for the reinforcing bars in regions where yielding is expected. Examples of hoops suitable for flexural members of frames are shown in Fig. R21.3.3.

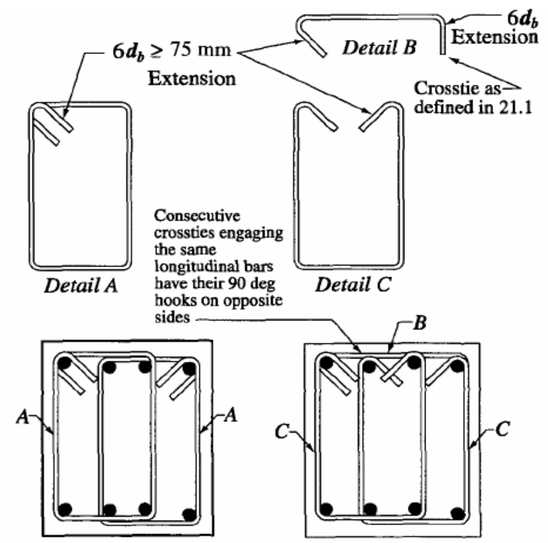


Fig. R21.3.3 - Examples of overlapping hoops

In the case of members with varying strength along the span or members for which the permanent load represents a large proportion of the total design load, concentrations of inelastic rotation may occur within the span. If such a condition is anticipated, transverse reinforcement also should be provided in regions where yielding is expected.

Because spalling of the concrete shell is anticipated during strong motion, especially at and near regions of flexural yielding, all web reinforcement should be provided in the form of closed hoops as defined in 21.3.3.5.

R21.3.4 Shear strength requirements

R21.3.4.1 Design forces. In determining the equivalent lateral forces representing earthquake effects for the type of frames considered, it is assumed that frame members will dissipate energy in the nonlinear range of response. Unless a frame member possesses a strength that is a multiple on the order of 3 or 4 of the design forces, it should be assumed that it will yield in the event of a major earthquake. The design shear force should be a good approximation of the maximum shear that may develop in a member. Therefore, required shear strength for frame members is related to flexural strengths of the designed member rather than to factored shear forces indicated by lateral load analysis. The conditions described by 21.3.4.1 are illustrated in Fig. R21.3.4.

Because the actual yield strength of the longitudinal reinforcement may exceed the specified yield strength and because strain hardening of the reinforcement is likely to take place at a joint subjected to large rotations, required shear strengths are determined using a stress of at least $1.25 f_y$ in the longitudinal reinforcement.

R21.3.4.2 Transverse reinforcement. Experimental studies[21.13,21.14] of reinforced concrete members subjected to cyclic loading have demonstrated that more shear reinforcement is required to ensure a flexural failure if the member is subjected to alternating nonlinear displacements than if the member is loaded in only one direction: the necessary increase of shear reinforcement being higher in the case of no axial load. This observation is reflected in the SBC 304 (see 21.3.4.2) by eliminating the term representing the contribution of concrete to shear strength. The added conservatism on shear is deemed necessary in locations where potential flexural hinging may occur. However, this stratagem, chosen for its relative simplicity, should not be interpreted to mean that no concrete is required to resist shear. On the contrary, it may be argued that the concrete core resists all of the shear with the shear (transverse) reinforcement confining and strengthening the concrete. The confined concrete core plays an important role in the behavior of the beam and should not be reduced to a minimum just because the design expression does not explicitly recognize it.

Notes on Fig. R21.3.4

1. Direction of shear force V_e depends on relative magnitudes of gravity loads and shear generated by end moments.
2. End moments M_{pr} based on steel tensile stress of $1.25 f_y$, where f_y is specified yield strength. (Both end moments should be considered in both directions, clockwise and counter-clockwise).
3. End moment M_{pr} for columns need no be greater than moments generated by the M_{pr} of the beams framing into the beam-column joints. V_e should not be less than that required by analysis of the structure.

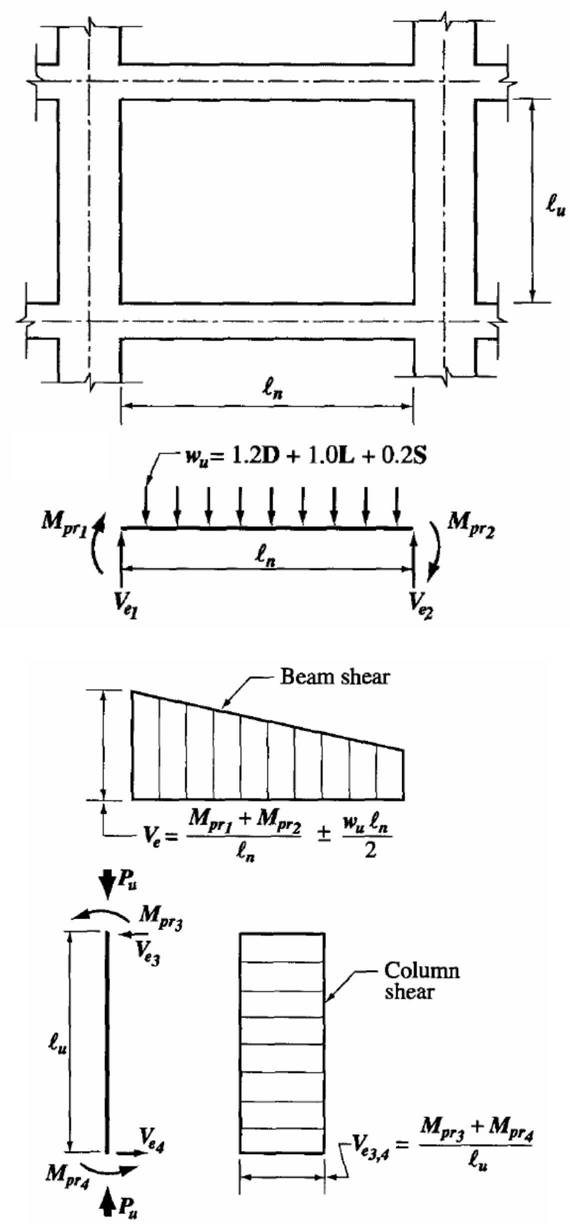


Fig. R21.3.4 - Design shears for girders and columns

SECTION R21.4
SPECIAL MOMENT FRAME MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

R21.4.1 **Scope.** Section 21.4.1 is intended primarily for columns of special moment frames. Frame members, other than columns, that do not satisfy 21.3.1 are to be proportioned and detailed according to this section.
The geometric constraints in 21.4.1.1 and 21.4.1.2 follow from previous practice.[21.12]

R21.4.2 **Minimum flexural strength of columns.** The intent of 21.4.2.2 is to reduce the likelihood of yielding in columns that are considered as part of the lateral force resisting system. If columns are not stronger than beams framing into a joint, there is likelihood of inelastic action. In the worst case of weak columns, flexural yielding can occur at both ends of all columns in a given story, resulting in a column failure mechanism that can lead to collapse.

In 21.4.2.2, the nominal strengths of the girders and columns are calculated at the joint faces, and those strengths are compared directly using Eq. (21-1).

When determining the nominal flexural strength of a girder section in negative bending (top in tension), longitudinal reinforcement contained within an effective flange width of a top slab that acts monolithically with the girder increases the girder strength. Research[21.15] on beam-column subassemblies under lateral loading indicates that using the effective flange widths defined in 8.10 gives reasonable estimates of girder negative bending strengths of interior connections at interstory displacement levels approaching 2 percent of story height. This effective width is conservative where the slab terminates in a weak spandrel.

If 21.4.2.2 cannot be satisfied at a joint, any positive contribution of the column or columns involved to the lateral strength and stiffness of the structure is to be ignored. Negative contributions of the column or columns should not be ignored. For example, ignoring the stiffness of the columns ought not to be used as a justification for reducing the design base shear. If inclusion of those columns in the analytical model of the building results in an increase in torsional effects, the increase should be considered as required by the provisions of SBC 301.

R21.4.3 **Longitudinal reinforcement.** The lower limit of the reinforcement ratio is to control time-dependent deformations and to have the yield moment exceed the cracking moment. The upper limit of the section reflects concern for steel congestion, load transfer from floor elements to column especially in low-rise construction, and the development of high shear stresses.

Spalling of the shell concrete, which is likely to occur near the ends of the column in frames of typical configuration, makes lap splices in these locations vulnerable. If lap splices are to be used at all, they should be located near the midheight where stress reversal is likely to be limited to a smaller stress range than at locations near the joints. Special transverse reinforcement is required along the lap-splice length because of the uncertainty in moment distributions along the height and the need for confinement of lap splices subjected to stress reversals. [21.16]

R21.4.4 **Transverse reinforcement.** Requirements of this section are concerned with confining the concrete and providing lateral support to the longitudinal reinforcement.

The effect of helical (spiral) reinforcement and adequately configured rectangular hoop reinforcement on strength and ductility of columns is well established[21.17] While analytical procedures exist for calculation of strength and ductility capacity of columns under axial and moment reversals[21.18],the axial load and deformation demands required during earthquake loading are not known with sufficient accuracy to justify calculation of required transverse reinforcement as a function of design earthquake demands. Instead, Eq. (10-5) and (21-3) are required, with the intent that spalling of shell concrete will not result in a loss of axial load strength of the column. Eq. (21-2) and (21-4) govern for large-diameter columns, and are intended to ensure adequate flexural curvature capacity in yielding regions.

Fig. R21.4.4 shows an example of transverse reinforcement provided by one hoop and three crossties. Crossties with a 90 deg hook are not as effective as either crossties with 135 deg hooks or hoops in providing confinement. Tests show that if crosstie ends with 90 deg hooks are alternated, confinement will be sufficient.

Sections 21.4.4.2 and 21.4.4.3 are interrelated requirements for configuration of rectangular hoop reinforcement. The requirement that spacing not exceed one-quarter of the minimum member dimension is to obtain adequate concrete confinement. The requirement that spacing not exceed six bar diameters is intended to restrain longitudinal reinforcement buckling after spalling. The 100 mm spacing is for concrete confinement; 21.4.4.2 permits this limit to be relaxed to a maximum of 150 mm if the spacing of crossties or legs of overlapping hoops is limited to 200 mm.

The unreinforced shell may spall as the column deforms to resist earthquake effects. Separation of portions of the shell from the core caused by local spalling creates a falling hazard. The additional reinforcement is required to reduce the risk of portions of the shell falling away from the column.

Section 21.4.4.4 stipulates a minimum length over which to provide closely-spaced transverse reinforcement at the member ends, where flexural yielding normally occurs. Research results indicate that the length should be increased by 50 percent or more in locations, such as the base of the building, where axial loads and flexural demands may be especially high.[21.19]

Columns supporting discontinued stiff members, such as walls or trusses, may develop considerable inelastic response. Therefore, it is required that these columns have special transverse reinforcement throughout their length. This covers all columns beneath the level at which the stiff member has been discontinued, unless the factored forces corresponding to earthquake effect are low (see 21.4.4.5).

Field observations have shown significant damage to columns in the unconfined region near the midheight. The requirements of 21.4.4.6 are to ensure a relatively uniform toughness of the column along its length.

R21.4.4.6 The provisions of 21.4.4.6 provide reasonable protection and ductility to the midheight of columns between transverse reinforcement. Observations after earthquakes have shown significant damage to columns in the nonconfined region, and the minimum ties or spirals required should provide a more uniform toughness of the column along its length.

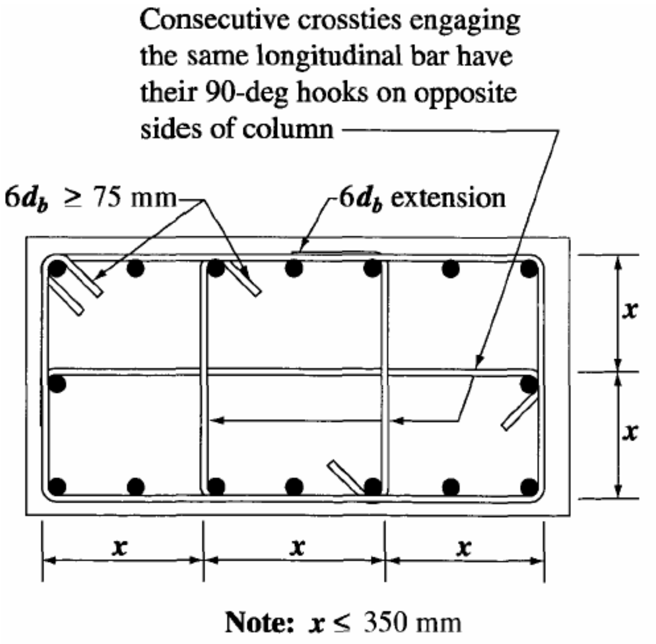


Fig. R21.4.4 - Example of transverse reinforcement in columns

R21.4.5 Shear strength requirements

R21.4.5.1 Design forces. The provisions of 21.3.4.1 also apply to members subjected to axial loads (for example, columns). Above the ground floor the moment at a joint may be limited by the flexural strength of the beams framing into the joint. Where beams frame into opposite sides of a joint, the combined strength may be the sum of the negative moment strength of the beam on one side of the joint and the positive moment strength of the beam on the other side of the joint. Moment strengths are to be determined using a strength reduction factor of 1.0 and reinforcing steel stress equal to at least $1.25f_y$. Distribution of the combined moment strength of the beams to the columns above and below the joint should be based on analysis. The value of M_{pr} in Fig. R21.3.4 may be computed from the flexural member strengths at the beam-column joints.

SECTION R21.5
JOINTS OF SPECIAL MOMENT FRAMES

R21.5.1 General requirements. Development of inelastic rotations at the faces of joints of reinforced concrete frames is associated with strains in the flexural reinforcement well in excess of the yield strain. Consequently, joint shear force generated by the flexural reinforcement is calculated for a stress of $1.45f_y$ in the reinforcement (see 21.5.1.1). A detailed explanation of the reasons for the possible development of stresses in excess of the yield strength in girder tensile reinforcement is provided in Reference 21.10.

R21.5.1.4 Research [21.20-21.24] has shown that straight beam bars may slip within the beam-column joint during a series of large moment reversals. The bond stresses on these straight bars may be very large. To substantially reduce slip during the formation of adjacent beam hinging, it would be necessary to have a ratio of column dimension to bar diameter of approximately 1/32, which would result in very large joints. On reviewing the available tests, the limit of 1/25 of the column depth in the direction of loading for the maximum size of beam bars for normalweight concrete and a limit of 1/30 for lightweight concrete were chosen. Due to the lack of specific data, the modification for lightweight concrete used a factor of 1.2. These limits provide reasonable control on the amount of potential slip of the beam bars in a beam-column joint, considering the number of anticipated inelastic excursions of the building frames during a major earthquake. A thorough treatment of this topic is given in Reference 21.25.

R21.5.2 **Transverse reinforcement.** No matter how low the calculated shear force in a joint of a frame resisting earthquake-induced forces, confining reinforcement (see 21.4.4) should be provided through the joint around the column reinforcement (see 21.5.2.1). In 21.5.2.2, confining reinforcement may be reduced if horizontal members frame into all four sides of the joint. A maximum limit on spacing to these areas is based on available data (References 21.26 through 21.29).

Section 21.5.2.3 refers to a joint where the width of the girder exceeds the corresponding column dimension. In that case, girder reinforcement not confined by the column reinforcement should be provided lateral support either by a girder framing into the same joint or by transverse reinforcement.

R21.5.3 **Shear strength.** The requirements in Chapter 21 for proportioning joints are based on Reference 21.10 in that behavioral phenomena within the joint are interpreted in terms of a nominal shear strength of the joint. Because tests of joints[21.20] and deep beams[21.11] indicated that shear strength was not as sensitive to joint (shear) reinforcement as implied by the expression developed by Reference 21.30 for beams and adopted to apply to joints by Reference 21.10. This SBC 304 set the strength of the joint as a function of only the compressive strength of the concrete (see 21.5.3) and to require a minimum amount of transverse reinforcement in the joint (see 21.5.2). The effective area of joint A_j is illustrated in Fig. R21.5.3.

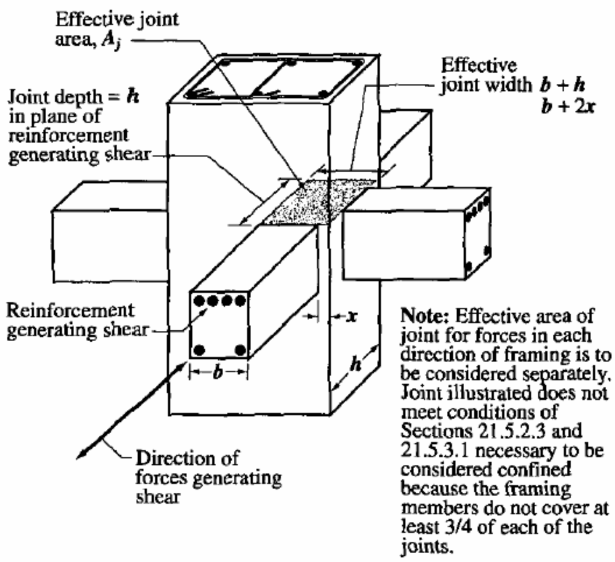


Fig. R21.5.3 - Effective joint area

In no case is A_j greater than the column cross-sectional area.

The three levels of shear strength required by 21.5.3.1 are based on the recommendation of Reference 21.10.

R21.5.4 **Development length of bars in tension**. Minimum development length for deformed bars with standard hooks embedded in normalweight concrete is determined using Eq. (21-6). Eq. (21-6) is based on the requirements of 12.5. Because Chapter 21 stipulates that the hook is to be embedded in confined concrete, the coefficients 0.7 (for concrete cover) and 0.8 (for ties) have been incorporated in the constant used in Eq. (21-6). The development length that would be derived directly from 12.5 is increased to reflect the effect of load reversals.

The development length in tension for a reinforcing bar with a standard hook is defined as the distance, parallel to the bar, from the critical section (where the bar is to be developed) to a tangent drawn to the outside edge of the hook. The tangent is to be drawn perpendicular to the axis of the bar (see Fig. R12.5).

Factors such as the actual stress in the reinforcement being more than the yield force and the effective development length not necessarily starting at the face of the joint were implicitly considered in the development of the expression for basic development length that has been used as the basis for Eq. (21-6).

For lightweight aggregate concrete, the length required by Eq. (21-6) is to be increased by 25 percent to compensate for variability of bond characteristics of reinforcing bars in various types of lightweight aggregate concrete.

Section 21.5.4.2 specifies the minimum development length for straight bars as a multiple of the length indicated by 21.5.4.1. Section 21.5.4.2(b) refers to top bars. If the required straight embedment length of a reinforcing bar extends beyond the confined volume of concrete (as defined in 21.3.3,21.4.4, or 21.5.2), the required development length is increased on the premise that the limiting bond stress outside the confined region is less than that inside.

or $$\ell_{dm} = 1.6(\ell_d - \ell_{dc}) + \ell_{dc}$$

where $$\ell_{dm} = 1.6\ell_d - 0.6\ell_{dc}$$

ℓ_{dm} = required development length if bar is not entirely embedded in confined concrete;

ℓ_d = required development length for straight bar embedded in confined concrete (see 21.5.4.3);

ℓ_{dc} = length of bar embedded in confined concrete

Lack of reference to Dia 40 mm bars and larger in 21.5.4 is due to the paucity of information on anchorage of such bars subjected to load reversals simulating earthquake effects.

SECTION R21.6 SPECIAL MOMENT FRAMES CONSTRUCTED USING PRECAST CONCRETE

The detailing provisions in 21.6.1 and 21.6.2 are intended to produce frames that respond to design displacements essentially like monolithic special moment frames.

Precast frame systems composed of concrete elements with ductile connections are expected to experience flexural yielding in connection regions. Reinforcement in ductile connections can be made continuous by using Type 2 mechanical splices or any other technique that provides development in tension or compression of at least 125 percent of the specified yield strength f_y of bars and the specified tensile strength of bars.[21.31,21.32,21.33,21.34] Requirements for mechanical splices are in addition to those in 21.2.6 and are intended to avoid strain concentrations over a short length of reinforcement adjacent to a splice device. Additional requirements for shear strength are provided in 21.6.1 to prevent sliding on connection faces. Precast frames composed of elements with ductile connections may be designed to promote yielding at locations not adjacent to the joints. Therefore, design shear, V_e as computed according to 21.3.4.1 or 21.4.5.1, may be conservative.

Precast concrete frame systems composed of elements joined using strong connections are intended to experience flexural yielding outside the connections. Strong connections include the length of the coupler hardware as shown in Fig. R21.6.2. Capacity-design techniques are used in 2 1.6.2(b) to ensure the strong connection remains elastic following formation of plastic hinges. Additional column requirements are provided to avoid hinging and strength deterioration of column-to-column connections.

Strain concentrations have been observed to cause brittle fracture of reinforcing bars at the face of mechanical splices in laboratory tests of precast beam-column connection.[21.35] Designers should carefully select locations of strong connections or take other measures, such as debonding of reinforcing bars in highly stressed regions, to avoid strain concentrations that can result in premature fracture of reinforcement.

R21.6.3 Precast frame systems not satisfying the prescriptive requirements of Chapter 21 have been demonstrated in experimental studies to provide satisfactory seismic performance characteristic.[21.36,21.37] ACI TI.1 defines a protocol for establishing a design procedure, validated by analysis and laboratory tests, for such frames. The design procedure should identify the load path or mechanism by which the frame resists gravity and earthquake effects. The tests should be configured to test critical behaviors, and the measured quantities should establish upper-bound acceptance values for components of the load path, which may be in terms of limiting stresses, forces, strains, or other quantities. The design procedure used for the structure should not deviate from that used to design the test specimens, and acceptance values should not exceed values that were demonstrated by the tests to be acceptable. Materials and components used in the structure should be similar to those used in the tests. Deviations may be acceptable if the engineer can demonstrate that those deviations do not adversely affect the behavior of the framing system.

SECTION R21.7
SPECIAL REINFORCED CONCRETE STRUCTURAL WALLS AND COUPLING BEAMS

R21.7.1 **Scope.** This section contains requirements for the dimensions and details of special reinforced concrete structural walls and coupling beams. Provisions for diaphragms are in 21.9.

R21.7.2 **Reinforcement**. Minimum reinforcement requirements (see 21.7.2.1) follow from preceding SBC 304s. The uniform distribution requirement of the shear reinforcement is related to the intent to control the width of inclined cracks. The requirement for two layers of reinforcement in walls carrying substantial design shears (see 21.7.2.2) is based on the observation that, under ordinary construction conditions, the probability of maintaining a single layer of reinforcement near the middle of the wall section is quite low. Furthermore, presence of reinforcement close to the surface tends to inhibit fragmentation of the concrete in the event of severe cracking during an earthquake.

Because the actual forces in longitudinal reinforcing bars of stiff members may exceed the calculated forces, it is required (see 21.7.2.3) that all continuous reinforcement be developed fully.

R21.7.3 **Design forces**. Design shears for structural walls are obtained from lateral load analysis with the appropriate load factors. However, the designer should consider the possibility of yielding in components of such structures, as in the portion of a wall between two window openings, in which case the actual shear may be in excess of the shear indicated by lateral load analysis based on factored design forces.

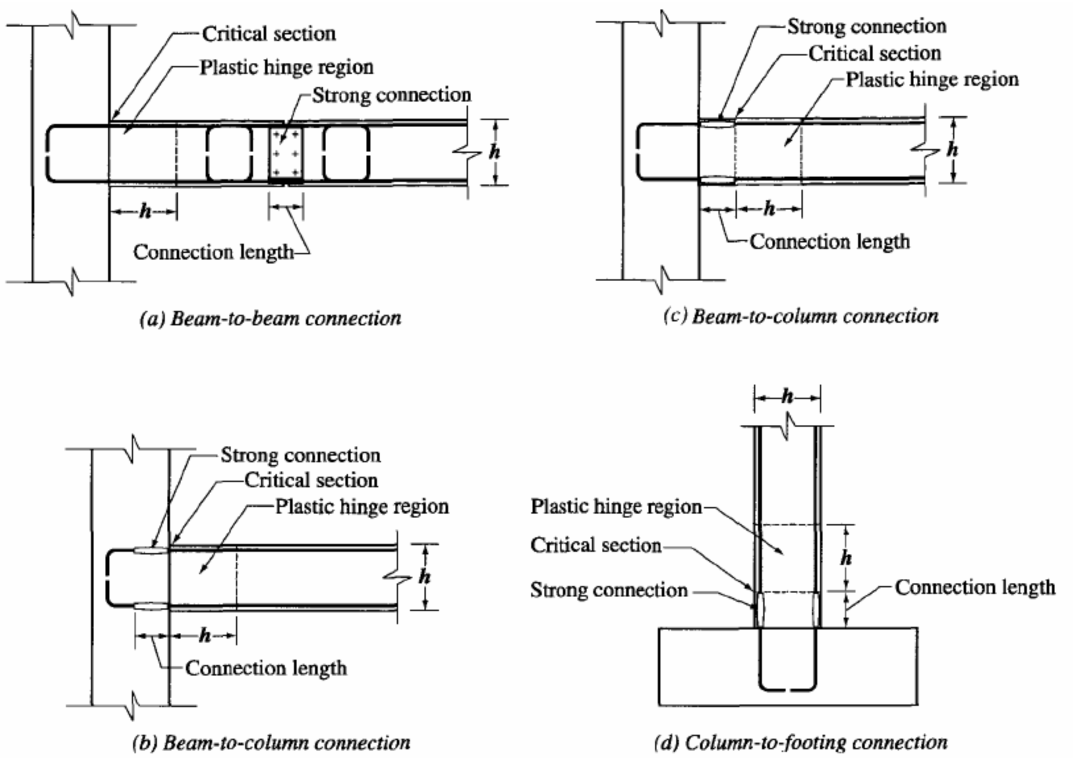


Fig. R21.6.2 - Strong connection examples

R21.7.4 **Shear strength.** Eq. (21-7) recognizes the higher shear strength of walls with high shear-to-moment ratios.[21.10,21.30,21.38] The nominal shear strength is given in terms of the net area of the section resisting shear. For a rectangular section without openings, the term A_{cv} refers to the gross area of the cross section rather than to the product of the width and the effective depth. The definition of A_{cv} in Eq. (21-7) facilitates design calculations for walls with uniformly distributed reinforcement and walls with openings.

A wall segment refers to a part of a wall bounded by openings or by an opening and an edge. Traditionally, a vertical wall segment bounded by two window openings has been referred to as a pier.

The ratio h_w / ℓ_w may refer to overall dimensions of a wall, or of a segment of the wall bounded by two openings, or an opening and an edge. The intent of 21.7.4.2 is to make certain that any segment of a wall is not assigned a unit strength larger than that for the whole wall. However, a wall segment with a ratio of h_w / ℓ_w higher than that of the entire wall should be proportioned for the unit strength associated with the ratio h_w / ℓ_w based on the dimensions for that segment.

To restrain the inclined cracks effectively, reinforcement included in ρ_n and ρ_v should be appropriately distributed along the length and height of the wall (see 21.7.4.3). Chord reinforcement provided near wall edges in concentrated amounts for resisting bending moment is not to be included in determining ρ_n and ρ_v within practical limits, shear reinforcement distribution should be uniform and at a small spacing.

If the factored shear force at a given level in a structure is resisted by several walls or several piers of a perforated wall, the average unit shear strength assumed for the total available cross-sectional area is limited to $2/3\sqrt{f_c'}$ with the additional requirement that the unit shear strength assigned to any single pier does not exceed $5/6\sqrt{f_c'}$. The upper limit of strength to be assigned to any one member is imposed to limit the degree of redistribution of shear force.

"Horizontal wall segments" in 21.7.4.5 refers to wall sections between two vertically aligned openings (see Fig. R21.7.4.5). It is, in effect, a pier rotated through 90 deg. A horizontal wall segment is also referred to as a coupling beam when the openings are aligned vertically over the building height.

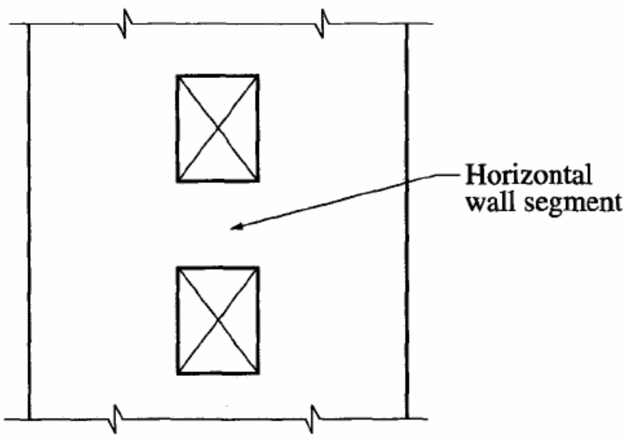


Fig. R21.7.4.5 - Wall with openings

R21.7.5 Design for flexure and axial loads

R21.7.5.1 Flexural strength of a wall or wall segmentis determined according to procedures commonly used for columns. Strength should be determined considering the applied axial and lateral forces. Reinforcement concentrated in boundary elements and distributed in flanges and webs should be included in the strength computations based on a strain compatibility analysis. The foundation supporting the wall should be designed to develop the wall boundary and web forces. For walls with openings, the influence of the opening or openings on flexural and shear strengths is to be considered and a load path around the opening or openings should be verified. Capacity design concepts and strut-and-tie models may be useful for this purpose.[21.39]

R21.7.5.2 Where wall sections intersect to form L-, T-, C-, or other cross-sectional shapes, the influence of the flange on the behavior of the wall should be considered by selecting appropriate flange widths. Tests [21.40] show that effective flange width increases with increasing drift level and the effectiveness of a flange in compression differs from that for a flange in tension. The value used for the effective compression flange width has little impact on the strength and deformation capacity of the wall; therefore, to simplify design, a single value of effective flange width based on an estimate of the effective tension flange width is used in both tension and compression. [21.40]

R21.7.6 Boundary elements of special reinforced concrete structural walls

R21.7.6.1 Two design approaches for evaluating detailing requirements at wall boundaries are included in 21.7.6.1. Section 21.7.6.2 allows the use of displacement-based design of walls, in which the structural details are determined directly on the basis of the expected lateral displacements of the wall. Requirements of 21.7.6.4 and 21.7.6.5 apply to structural walls designed by either 21.7.6.2 or 21.7.6.3.

R21.7.6.2 Section 21.7.6.2 is based on the assumption that inelastic response of the wall is dominated by flexural action at a critical, yielding section. The wall should be proportioned so that the critical section occurs where intended.

Eq. (21-8) follows from a displacement-based approach.[21.41,21.42] The approach assumes that special boundary elements are required to confine the concrete where the strain at the extreme compression fiber of the wall exceeds a critical value when the wall is displaced to the design displacement. The horizontal dimension of the special boundary element is intended to extend at least over the length where the compression strain exceeds the critical value. The height of the special boundary element is based on upper bound estimates of plastic hinge length and extends beyond the zone over which concrete spalling is likely to occur. The lower limit of 0.007 on the quantity δ_u / h_w requires moderate wall deformation capacity for stiff buildings.

The neutral axis depth c in Eq. (21-8) is the depth calculated according to 10.2, except the nonlinear strain requirements of 10.2.2 need not apply, corresponding to development of nominal flexural strength of the wall when displaced in the same direction as δ_u. The axial load is the factored axial load that is consistent with the design load combination that produces the displacement δ_u.

R21.7.6.3 By this procedure, the wall is considered to be acted on by gravity loads W and the maximum shear and moment induced by earthquake in a given direction. Under this loading, the compressed boundary at the critical section resists the tributary gravity load plus the compressive resultant associated with the bending moment.

Recognizing that this loading condition may be repeated many times during the strong motion, the concrete is to be confined where the calculated compressive stresses exceed a nominal critical value equal to $0.2f_c^{'}$. The stress is to be calculated for the factored forces on the section assuming linear response of the gross concrete section. The compressive stress of $0.2f_c^{'}$ is used as an index value and does not necessarily describe the actual state of stress that may develop at the critical section under the influence of the actual inertia forces for the anticipated earthquake intensity.

R21.7.6.4 The value of $c/2$ in 21.7.6.4(a) is to provide a minimum length of the special boundary element. Where flanges are heavily stressed in compression, the web-to-flange interface is likely to be heavily stressed and may sustain local crushing failure unless special boundary element reinforcement extends into the web. Eq. (21-3) does not apply to walls.

Because horizontal reinforcement is likely to act as web reinforcement in walls requiring boundary elements, it should be fully anchored in boundary elements that act as flanges (21.7.6.4). Achievement of this anchorage is difficult when large transverse cracks occur in the boundary elements. Therefore, standard 90 deg hooks or mechanical anchorage schemes are recommended instead of straight bar development.

R21.7.6.5 Cyclic load reversals may lead to buckling of boundary longitudinal reinforcement even in cases where the demands on the boundary of the wall do not require special boundary elements. For walls with moderate amounts of boundary longitudinal reinforcement, ties are required to inhibit buckling. The longitudinal reinforcement ratio is intended to include only the reinforcement at the wall boundary as indicated in Fig. R21.7.6.5. A larger spacing of ties relative to 21.7.6.4(c) is allowed due to the lower deformation demands on the walls.

The addition of hooks or U-stirrups at the ends of horizontal wall reinforcement provides anchorage so that the reinforcement will be effective in resisting shear forces. It will also tend to inhibit the buckling of the vertical edge reinforcement. In walls with low in-plane shear, the development of horizontal reinforcement is not necessary.

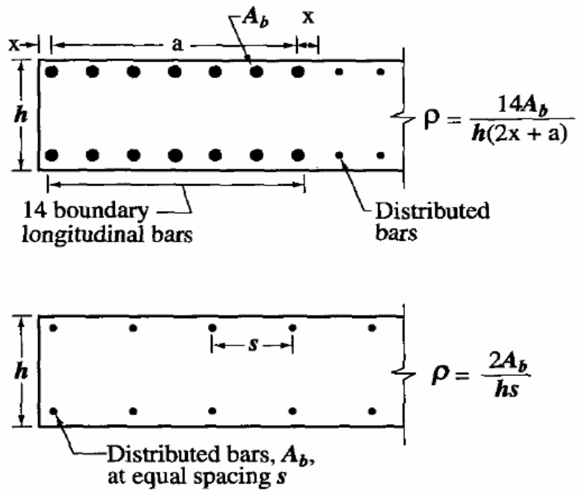


Fig. R21.7.6.5 - Longitudinal reinforcement ratios for typical wall boundary conditions

R21.7.7 **Coupling beams**. Coupling beams connecting structural walls can provide stiffness and energy dissipation. In many cases, geometric limits result in coupling beams that are deep in relation to their clear span. Deep coupling beams may be controlled by shear and may be susceptible to strength and stiffness deterioration under earthquake loading. Test results[21.43,21.44] have shown that confined diagonal reinforcement provides adequate resistance in deep coupling beams.

Experiments show that diagonally oriented reinforcement is effective only if the bars are placed with a large inclination. Therefore, diagonally reinforced coupling beams are restricted to beams having aspect ratio $\ell_n / h < 4$.

Each diagonal element consists of a cage of longitudinal and transverse reinforcement as shown in Fig. R21.7.7. The cage contains at least four longitudinal bars and confines a concrete core. The requirement on side dimensions of the cage and its core is to provide adequate toughness and stability to the cross section when the bars are loaded beyond yielding. The minimum dimensions and required reinforcement clearances may control the wall width.

When coupling beams are not used as part of the lateral force resisting system, the requirements for diagonal reinforcement may be waived. Nonprestressed coupling beams are permitted at locations where damage to these beams does not impair vertical load carrying capacity or egress ofthe structure, or integrity of the nonstructural components and their connections to the structure.

Tests in Reference 21.44 demonstrated that beams reinforced as described in Section 21.7.7 have adequate ductility at shear forces exceeding $(5/6)\sqrt{f_c'}b_w d$. Consequently, the use of a limit of $(5/6)\sqrt{f_c'}A_{cp}$ provides an acceptable upper limit.

When the diagonally oriented reinforcement is used, additional reinforcement in 21.7.7.4(f) is to contain the concrete outside the diagonal cores if the concrete is damaged byearthquake loading (Fig. R21.7.7).

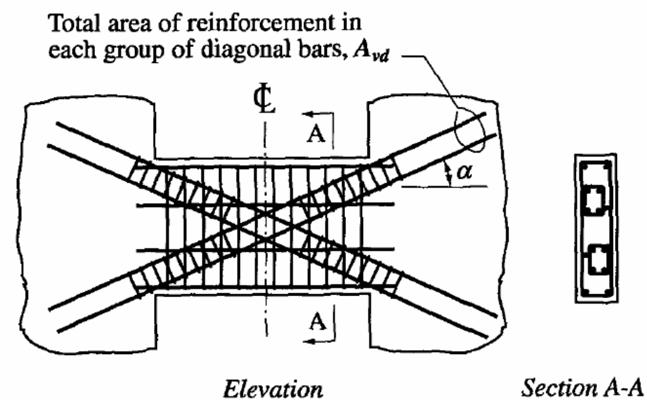


Fig. R21.7.7 - Coupling beam with diagonally oriented reinforcement

SECTION R21.9
STRUCTUREAL DIAPHRAGMS AND TRUSSES

R21.9.1 **Scope.** Diaphragms as used in building construction are structural elements (such as a floor or roof) that provide some or all of the following functions:

(a) Support for building elements (such as walls, partitions, and cladding) resisting horizontal forces but not acting as part of the building vertical lateral-force-resisting system;

(b) Transfer of lateral forces from the point of application to the building vertical lateral-force-resisting system;

(c) Connection of various components of the building vertical lateral-force-resisting system with appropriate strength, stiffness, and toughness so the building responds as intended in the design[21.45]

R21.9.2 **Cast-in-place composite-topping slab diaphragms.** A bonded topping slab is required so that the floor or roof system can provide restraint against slab buckling. Reinforcement is required to ensure the continuity of the shear transfer across precast joints. The connection requirements are introduced to promote a complete system with necessary shear transfers.

R21.9.3 **Cast-in-place topping slab diaphragms.** Composite action between the topping slab and the precast floor elements is not required, provided that the topping slab is designed to resist the design seismic forces.

R21.9.4 **Minimum thickness of diaphragms.** The minimum thickness of concrete diaphragms reflects current practice in joist and waffle systems and composite top ping slabs on precast floor and roof systems. Thicker slabs are required when the topping slab does not act compositely with the precast system to resist the design seismic forces.

R21.9.5 **Reinforcement.** Minimum reinforcement ratios for diaphragms correspond to the required amount of temperature and shrinkage reinforcement (7.12). The maximum spacing for web reinforcement is intended to control the width of inclined cracks. Minimum average prestress requirements (7.12.3) are considered to be adequate to limit the crack widths in post-tensioned floor systems; therefore, the maximum spacing requirements do not apply to these systems.

The minimum spacing requirement for welded wire fabric in topping slabs on precast floor systems (see 21.9.5.1) is to avoid fracture of the distributed reinforcement during an earthquake. Cracks in the topping slab open immediately above the boundary between the flanges of adjacent precast members, and the wires crossing those cracks are restrained by the transverse wire. [21.46] Therefore, all the deformation associated with cracking should be accommodated in a distance not greater than the spacing of the transverse wires. A minimum spacing of 250 mm for the transverse wires is required in 21.9.5.1 to reduce the likelihood of fracture of the wires crossing the critical cracks during a design earthquake. The minimum spacing requirements do not apply to diaphragms reinforced with individual bars, because strains are distributed over a longer length.

Compressive stress calculated for the factored forces on a linearly elastic model based on gross section of the structural diaphragm is used as an index value to determine whether confining reinforcement is required. A calculated compressive stress of $0.2f_c^{'}$ in a member is assumed to indicate that integrity of the entire structure is dependent on the ability of that member to resist substantial compressive force under severe cyclic loading. Therefore, transverse reinforcement in 21.4.4 is required in such members to provide confinement for the concrete and the reinforcement (21.9.5.3).

The dimensions of typical structural diaphragms often preclude the use of transverse reinforcement along the chords. Reducing the calculated compressive stress by reducing the span of the diaphragm is considered to be a solution.

R21.9.7 **Shear strength.** The shear strength requirements for monolithic diaphragms, Eq. (21-10) in 21.9.7.1, are the same as those for slender structural walls. The term A_{cv} refers to the thickness times the width of the diaphragm. This corresponds to the gross area of the effective deep beam that forms the diaphragm. The shear reinforcement should be placed perpendicular to the span of the diaphragm.

The shear strength requirements for topping slab diaphragms are based on a shear friction model, and the contribution of the concrete to the nominal shear strength is not included in Eq. (21-9) for topping slabs placed over precast floor elements. Following typical construction practice, the topping slabs are roughened immediately above the boundary between the flanges of adjacent precast floor members to direct the paths of shrinkage cracks. As a result, critical sections of the diaphragm are cracked under service loads, and the contribution of the concrete to the shear capacity of the diaphragm may have already been reduced before the design earthquake occurs.

R21.9.8 **Boundary elements of structural diaphragms**. For structural diaphragms, the design moments are assumed to be resisted entirely by chord forces acting at opposite edges of the diaphragm. Reinforcement located at the edges of collectors should be fully developed for its yield strength. Adequate confinement of lap splices is also required. If chord reinforcement is located within a wall, the joint between the diaphragm and the wall should be provided with adequate shear strength to transfer the shear forces.

Section 21.9.8.3 is intended to reduce the possibility of chord buckling in the vicinity of splices and anchorage zones.

SECTION R21.10
FOUNDATIONS

R21.10.1 **Scope**. Requirements for foundations supporting buildings assigned to high seismic performance or design categories, represent a consensus of a minimum level of good practice in designing and detailing concrete foundations including piles, drilled piers, and caissons. It is desirable that inelastic response in strong ground shaking occurs above the foundations, as repairs to foundations can be extremely difficult and expensive.

R21.10.2 **Footings, foundation mats, and pile caps**

R21.10.2.2 Tests [21.47] have demonstrated that flexural members terminating in a footing, slab or beam (a T-joint) should have their hooks turned inwards toward the axis of the member for the joint to be able to resist the flexure in the member forming the stem of the T.

R21.10.2.3 Columns or boundary members supported close to the edge of the foundation, as often occurs near property lines, should be detailed to prevent an edge failure of the footing, pile cap, or mat.

R21.10.2.4 The purpose of 21.10.2.4 is to alert the designer to provide top reinforcement as well as other required reinforcement.

R21.10.2.5 In region of high seismicity, it is desirable to reinforce foundation, and basement wall.

R21.10.3 **Grade beams and slabs on grade.** For seismic conditions, slabs on grade (soil-supported slabs) are often part of the lateral-force-resisting system and should be designed in accordance with this SBC 304 as well as other appropriate standards or guidelines. See 1.1.6.

R21.10.3.2 Grade beams between pile caps or footings can be separate beams beneath the slab on grade or can be a thickened portion of the slab on grade. The cross-sectional limitation and minimum tie requirements provide reasonable proportions.

R21.10.3.3 Grade beams resisting seismic flexural stresses from column moments should have reinforcing details similar to the beams of the frame above the foundation.

R21.10.3.4 Slabs on grade often act as a diaphragm to hold the building together at the ground level and minimize the effects of out-of-phase ground motion that may occur over the footprint of the building. In these cases, the slab on grade should be adequately reinforced and detailed. The design drawings should clearly state that these slabs on grade are structural members so as to prohibit saw cutting of the slab.

R21.10.4 **Piles, piers, and caissons.** Adequate performance of piles and caissons for seismic loadings requires that these provisions be met in addition to other applicable standards or guidelines. See R1.1.5.

R21.10.4.2 A load path is necessary at pile caps to transfer tension forces from the reinforcing bars in the column or boundary member through the pile cap to the reinforcement of the pile or caisson.

R21.10.4.3 Grouted dowels in a blockout in the top of a precast concrete pile need to be developed, and testing is a practical means of demonstrating capacity. Alternatively, reinforcing bars can be cast in the upper portion of the pile, exposed by chipping of concrete and mechanically connected or welded to an extension.

R21.10.4.4 During earthquakes, piles can be subjected to extremely high flexural demands at points of discontinuity, especially just below the pile cap and near the base of a soft or loose soil deposit. The requirement for confinement reinforcement at the top of the pile is based on numerous failures observed at this location in recent earthquakes. Transverse reinforcement is required in this region to provide ductile performance. The designer should also consider possible inelastic action in the pile at abrupt changes in soil deposits, such as changes from soft to firm or loose to dense soil layers. Where precast piles are to be used, the potential for the pile tip to be driven to an elevation different than that specified in the drawings needs to be considered when detailing the pile. If the pile reaches refusal at a shallower depth, a longer length of pile will need to be cut off. If this possibility is not forseen, the length of transverse reinforcement required by 21.10.4.4 may not be available after the excess pile length is cut off.

R21.10.4.7 Extensive structural damage has often been observed at the junction of batter piles and the buildings. The pile cap and surrounding structure should be designed for the potentially large forces that can be developed in batter piles.

SECTION R21.11
FRAME MEMBERS NOT PROPORTIONED TO RESIST FORCES INDUCED BY EARTHQUAKE MOTIONS

The detailing requirements for members that are part of the lateral-force resisting system assume that the members may undergo deformations that exceed the yield limit of the member without significant loss of strength. Members that are not part of the designated lateral-force-resisting system are not required to meet all the detailing requirements of members that are relied on to resist lateral forces. They should, however, be able to resist the gravity loads at lateral displacements corresponding to the design level prescribed by the governing SBC 304 for earthquake-resistant design. The design displacement is defined in 21.1.

Section 21.1 recognizes that actual displacements resulting from earthquake forces may be larger than the displacements calculated using the design forces and commonly used analysis models. Section 21.11.1 defines a nominal displacement for the purpose of prescribing detailing requirements. This section is consistent with the strength design approach of Ref. 21.2. Actual displacements may exceed the value of 21.11.1.

Section 21.11.2 prescribes detailing requirements intended to provide a system capable of sustaining gravity loads under moderate excursions into the inelastic range. Section 21.11.3 prescribes detailing requirements intended to provide a system capable of sustaining gravity loads under larger displacements.

Models used to determine design deflections of buildings should be chosen to produce results that conservatively bound the values expected during the design earthquake considering vertical, horizontal, and diaphragm systems as appropriate.

For gravity load factors, see R9.2.

The poor performance of some buildings with precast concrete gravity systems during the Northridge Earthquake was attributed to several factors addressed in 21.1 1.4. Columns should contain ties over their entire height, frame members not proportioned to resist earthquake forces should be tied together, and longer bearing lengths should be used to maintain integrity of the gravity system during shaking. The 50 mm increase in bearing length is based on an assumed 4 percent story drift ratio and 1.3 m beam depth, and is considered to be conservative for the ground motions expected in high seismic zones. In addition to the provisions of 21.11.4, precast frame members assumed not to contribute to lateral resistance should also satisfy 21.1 1.1 through 21.1 1.3.

SECTION R21.12
REQUIREMENTS FOR INTERMEDIATE MOMENT FRAMES

The objective of the requirements in 21.12.3 is to reduce the risk of failure in shear during an earthquake. The designer is given two options by which to determine the factored shear force.

According to option (a) of 21.12.3, the factored shear force is determined from the nominal moment strength of the member and the gravity load on it. Examples for a beam and a column are illustrated in Fig. R21.12.3.

To determine the maximum beam shear, it is assumed that its nominal moment strengths $\phi = 1.0$ are developed simultaneously at both ends of its clear span. A_s indicated in Fig. R21.12.3, the shear associated with this condition $\left[\left(M_{nl} + M_{nr}\right)/\ell_n\right]$ added algebraically to the effect of the factored gravity loads indicates the design shear for of the beam. For this example, both the dead load w_D and the live load w_L have been assumed to be uniformly distributed.

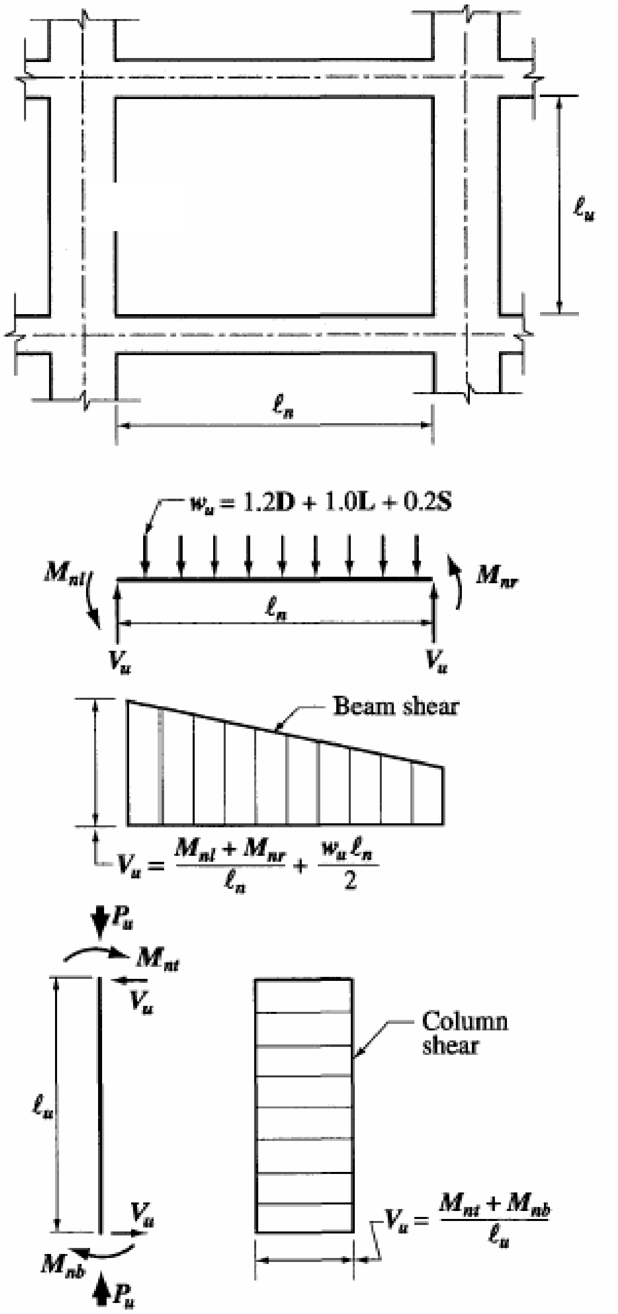


Fig. R21.12.3 - Design shears for frames in regions of moderate seismic risk (see 21.12)

Determination of the design shear for a column is also illustrated for a particular example in Fig. R21.12.3. The factored design axial load, P_u, should be chosen to develop the largest moment strength of the column.

In all applications of option (a) of 21.12.3, shears are required to be calculated for moment, acting clockwise and counter-clockwise. Fig. R21.12.3 demonstrates only one of the two conditions that are to be considered for every member. Option (b) bases V_u on the load combination including the earthquake effect, E, which should be doubled. For example, the load combination defined by Eq. (9-5) would be:

$$U = 1.2D + 2.0E + 1.0L$$

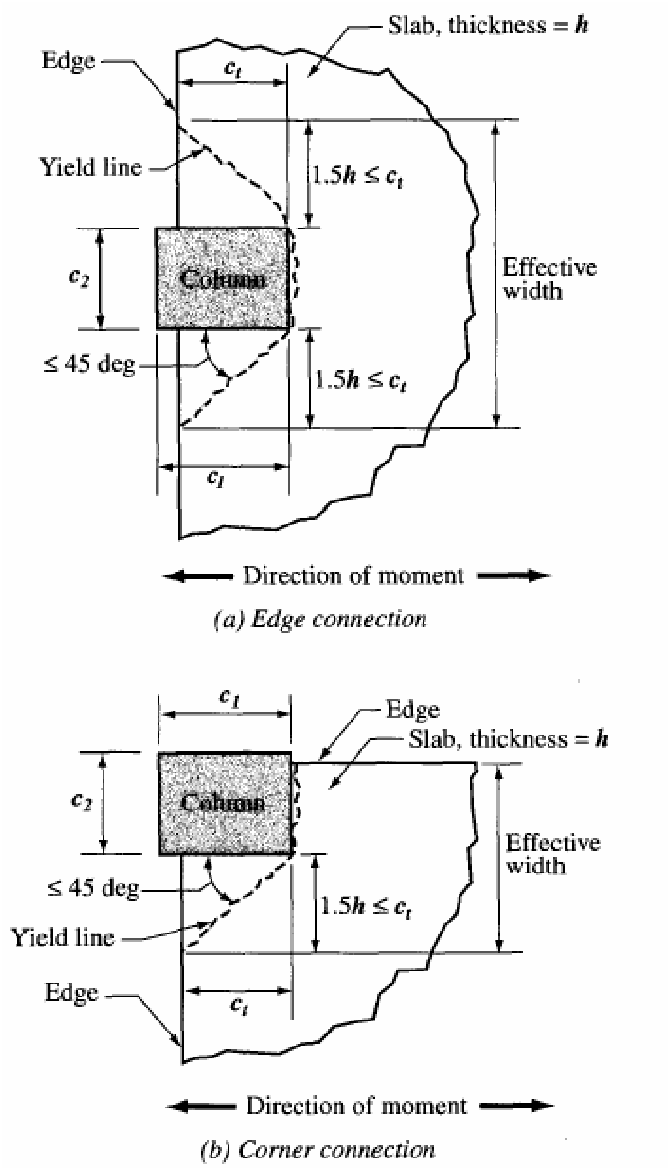
where E is the value specified by the governing SBC 304. Section 21.12.4 contains requirements for providing beams with a threshold level of toughness. Transverse reinforcement at the ends of the beam shall be hoops. In most cases, stirrups required by 21.12.3 for design shear force will be more than those required by 21.12.4. Requirements of 21.12.5 serve the same purpose for columns.

Section 21.12.6 applies to two-way slabs without beams, such as flat plates.

Using load combinations of Eq. (9-5) and (9-7) may result in moments requiring top and bottom reinforcement at the supports.

The moment M_s refers, for a given design load combination with E acting in one horizontal direction, to that portion of the factored slab moment that is balanced by the supporting members at a joint. It is not necessarily equal to the total design moment at support for a load combination including earthquake effect. In accordance with 13.5.3.2, only a fraction $\left(\gamma_f M_s\right)$ of the moment M_s is assigned to the slab effective width. For edge and corner connections, flexural reinforcement perpendicular to the edge is not considered fully effective unless it is placed within the effective slab width.[21.48,49] See Fig. 21.12.6.1.

Application of the provisions of 21.12.6 are illustrated in Fig. R21.12.6.2 and Fig. R21.12.6.3.



Fig, R21.12.6.1 - Effective width for reinforcement placement in edge and corner connections

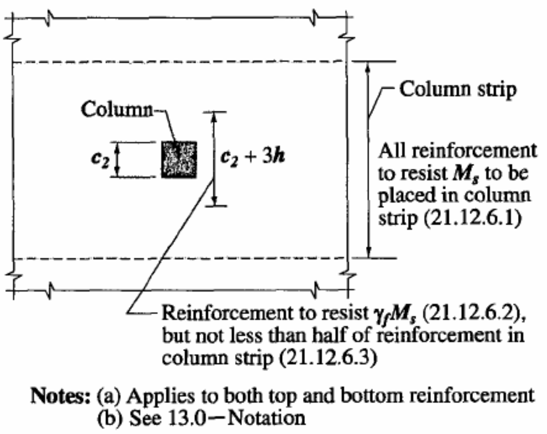


Fig. R21.12.6.2 - Location of reinforcement in slabs

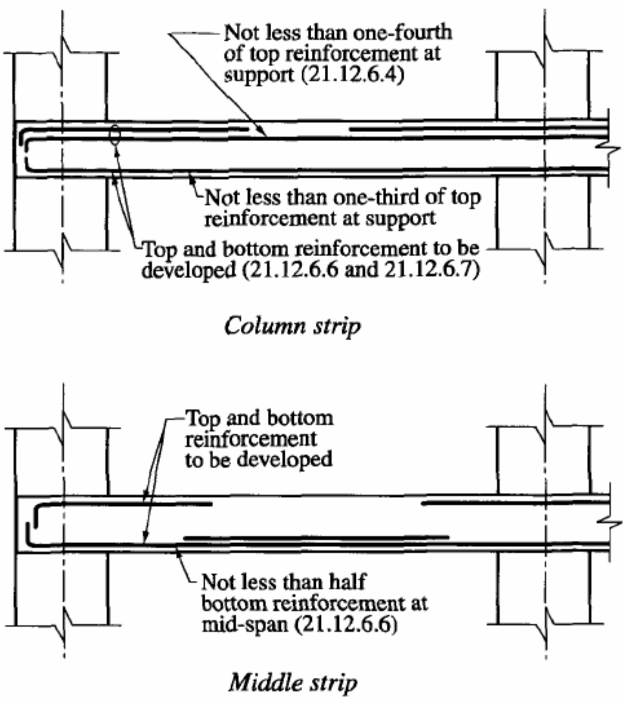


Fig. R21.12.6.3 - Arrangement of reinforcement in slabs

R21.12.6.8 The requirements apply to two-way slabs that are part of the primary lateral force-resisting system. Slab-column connections in laboratory tests [21.49] exhibited reduced lateral displacement ductility when the shear at the column connection exceeded the recommended limit.

SECTION R21.13
INTERMEDIATE PRECAST STRUCTURAL WALLS

Connections between precast wall panels or between wall panels and the foundation are required to resist forces induced by earthquake motions and to provide for yielding in the vicinity of connections. When Type 2 mechanical splices are used to directly connect primary reinforcement, the probable strength of the splice should be at least 1-1/2 times the specified yield strength of the reinforcement.

COMMENTARY REFERENCES

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Gratitude

In appreciation and gratitude

to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence

& Aviation and Inspector General

For their continuous support and gracious consideration,

the Saudi Building Code National Committee (SBCNC)

is honored to present the first issue of

the Saudi Building Code (SBC).

Saudi Building Code Requirements

201	Architectural	
301	Structural – Loading and Forces	
302	Structural – Testing and Inspection	
303	Structural – Soil and Foundations	
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PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11th June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Structural Requirements for Masonry Construction (SBC 305) were developed based on ICC code in addition to American Concrete Institute (ACI) materials. ACI grants permission to the SBCNC to include ACI materials in the SBC, and ACI is not responsible for any modifications or changes that SBCNC has made to accommodate local conditions.

The development process of SBC 305 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made on the base code and only SI units were used throughout the Code. The changes were intended to compose a comprehensive set of provisions, to the best possible extent, for materials, environmental conditions, and construction practices prevailing in the Kingdom.

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CHAPTER 1
GENERAL

SECTION 1.1
SCOPE

1.1.0 The Saudi Building Code for Masonry Structures referred to as SBC 305, provides minimum requirements for design and construction of Masonry Structures. SBC 305 shall govern the materials, design, construction, and quality of masonry.

SECTION 1.2
DESIGN METHODS

1.2.0 Masonry shall comply with the provisions of one of the following design methods in this code as well as the requirements of Chapter 1 through Chapter 4. Masonry designed by the working stress design provisions of Section 1.2.1, the strength design provisions of Section 1.2.2 or the seismic provisions of Section 1.2.3 shall comply with Chapter 5.

1.2.1 **Working stress design.** Masonry designed by the working stress design method shall comply with the provisions of Chapter 6 and Chapter 7.

1.2.2 **Strength design.** Masonry designed by the strength design method shall comply with the provisions of Chapter 6 and Chapter 8.

1.2.3 **Seismic design.** Masonry shall be designed in accordance with Chapter 6. Special inspection during construction shall be provided as set forth in SBC 302.

1.2.4 **Empirical design.** Masonry designed by the empirical design method shall comply with the provisions of Chapter 6 and Chapter 9.

1.2.5 **Glass masonry.** Glass masonry shall comply with the provisions of Chapter 10.

1.2.6 **Masonry veneer.** Masonry veneer shall comply with the provisions of Chapter 14.

SECTION 1.3
CONSTRUCTION DOCUMENTS

1.3.0 The construction documents shall show all of the items required by this code including the following:

1. Specified size, grade, type and location of reinforcement, anchors and wall ties.
2. Reinforcing bars to be welded and welding procedure.
3. Size and location of structural elements.
4. Provisions for dimensional changes resulting from elastic deformation, creep, shrinkage, temperature and moisture.
5. All loads used in the design of masonry.
6. Specified compressive strength of masonry at stated ages or stages of construction for which masonry is designed, except where specifically exempted by Code provisions.
7. Size and location of conduits, pipes, and sleeves.

1.3.1 **Fireplace drawings.** The construction documents shall describe in sufficient detail the location, size and construction of masonry fireplaces. The thickness and characteristics of materials and the clearances from walls, partitions and ceilings shall be clearly indicated.

1.3.2 The construction documents shall be consistent with design assumptions.

1.3.3 Construction documents shall specify the minimum level of quality assurance as defined in Chapter 5, or shall include an itemized quality assurance program that exceeds the requirements of Chapter 5.

1.3.4 Calculations pertinent to design shall be filed with the drawings when required by the building official. When automatic data processing is used, design assumptions, program documentation and identified input and output data may be submitted in lieu of calculations.

1.3.5 **Approval of special systems of design or construction.** Sponsors of any system of design or construction within the scope of this Code, the adequacy of which has been shown by successful use or by analysis or test, but that does not conform to or is not covered by this Code, shall have the right to present the data on which their design is based to a board of examiners appointed by the building official. The board shall be composed of registered engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of this Code. The rules, when approved and promulgated by the building official, shall be of the same force and effect as the provisions of this Code.

SECTION 1.4
LOADING

1.4.1 **General.** Masonry shall be designed to resist applicable loads.

1.4.2 **Load provisions.** Service loads shall be in accordance with SBC 301 with such live load reductions as are permitted in SBC 301.

1.4.3 **Lateral load resistance.** Buildings shall be provided with a structural system designed to resist wind and earthquake loads and to accommodate the effect of the resulting deformation.

1.4.4 **Other effects.** Consideration shall be given to effects of forces and deformations due to prestressing, vibrations, impact, shrinkage, expansion, temperature changes, creep, unequal settlement of supports, and differential movement.

1.4.5 **Lateral load distribution.** Lateral loads shall be distributed to the structural system in accordance with member stiffnesses and shall comply with the requirements of this section.

1.4.5.1 Flanges of intersecting walls designed in accordance with Section 3.13.4.2 shall be included in stiffness determination.

1.4.5.2 Distribution of load shall be consistent with the forces resisted by foundations.

1.4.5.3 Distribution of load shall include the effect of horizontal torsion of the structure due to eccentricity of wind or seismic loads resulting from the non-uniform distribution of mass.

CHAPTER 2
DEFINITIONS AND NOTATIONS

SECTION 2.1
DEFINITIONS

2.1.0 The following words and terms shall have the meanings shown herein.

ADOBE CONSTRUCTION. Construction in which the exterior load-bearing and non-load-bearing walls and partitions are of unfired clay masonry units.

Adobe, stabilized. Unfired clay masonry units to which admixtures such as emulsified asphalt, are added during the manufacturing process to limit the units' water absorption so as to increase their durability.

Adobe, unstabilized. Unfired clay masonry units that do not meet the definition of "Adobe, stabilized."

ANCHOR. Metal rod, wire or strap that secures masonry to its structural support.

Anchor pullout. Anchor failure defined by the anchor sliding out of the material in which it is embedded without breaking out a substantial portion of the surrounding material.

ARCHITECTURAL TERRA COTTA. Plain or ornamental hard-burned modified clay units, larger in size than brick, with glazed or unglazed ceramic finish.

AREA.

Bedded. The area of the surface of a masonry unit that is in contact with mortar in the plane of the joint.

Gross cross-sectional. The area delineated by the out-to-out specified dimensions of masonry in the plane under consideration.

Net cross-sectional. The area of masonry units, grout and mortar crossed by the plane under consideration based on out-to-out specified dimensions.

BACKING. The wall or surface to which the veneer is secured.

BED JOINT. The horizontal layer of mortar on which a masonry unit is laid.

BOND BEAM. A horizontal grouted element within masonry in which reinforcement is embedded.

BOND REINFORCING. The adhesion between steel reinforcement and mortar or grout.

BRICK.

Calcium silicate (sand lime brick). A masonry unit made of sand and lime.

Clay or shale. A masonry unit made of clay or shale, usually formed into a rectangular prism while in the plastic state and burned or fired in a kiln.

Concrete. A masonry unit having the approximate shape of a rectangular prism and composed of inert aggregate particles embedded in a hardened cementitious matrix.

BUTTRESS. A projecting part of a masonry wall built integrally therewith to provide lateral stability.

CAST STONE. A building stone manufactured from Portland cement concrete precast and used as a trim, veneer or facing on or in buildings or structures.

CAMBER. A deflection that is intentionally built into a structural element to improve appearance or to nullify the deflection of the element under the effects of loads, shrinkage and creep.

CELL. A void space having a gross cross-sectional area greater than 950 mm^2.

CHIMNEY. A primarily vertical enclosure containing one or more passageways for conveying flue gases to the outside atmosphere.

CHIMNEY TYPES.

High-heat appliance type. An approved chimney for removing the products of combustion from fuel-burning, high-heat appliances producing combustion gases in excess of 1100^{o}C measured at the appliance flue outlet see Section 13.11.3.

Low-heat appliance type. An approved chimney for removing the products of combustion from fuel-burning, low-heat appliances producing combustion gases not in excess of 540^{o}C under normal operating conditions, but capable of producing combustion gases of 760^{o}C during intermittent forces firing for periods up to 1 hour. Temperatures shall be measured at the appliance flue outlet.

Masonry type. A field-constructed chimney of solid masonry units or stones.

Medium-heat appliance type. An approved chimney for removing the products of combustion from fuel-burning, medium-heat appliances producing combustion gases not exceeding 1100^{o}C measured at the appliance flue outlet see Section 13.11.2.

CLEANOUT. An opening to the bottom of a grout space of sufficient size and spacing to allow the removal of debris.

COLLAR JOINT. Vertical longitudinal joint between wythes of masonry or between masonry and backup construction that is permitted to be filled with mortar or grout.

COLUMN, MASONRY. An isolated vertical member whose horizontal dimension measured at right angles to its thickness does not exceed three times its thickness and whose height is at least four times its thickness.

COMPOSITE ACTION. Transfer of stress between components of a member designed so that in resisting loads, the combined components act together as a single member.

COMPOSITE MASONRY. Multiwythe masonry members acting with composite action.

COMPRESSIVE STRENGTH OF MASONRY. Maximum compressive force resisted per unit of net cross-sectional area of masonry, determined by the testing of masonry prisms or a function of individual masonry units, mortar and grout.

CONNECTOR. A mechanical device for securing two or more pieces, parts or members together, including anchors, wall ties and fasteners.

COVER. Distance between surface of reinforcing bar and edge of member.

DEPTH. The dimension of a member measured in the plane of a cross section perpendicular to the neutral axis.

DESIGN STORY DRIFT. The difference of deflections at the top and bottom of the story under consideration, calculated by multiplying the deflections determined from an elastic analysis by the appropriate deflection amplification factor, C_d from SBC 301.

DIAPHRAGM. A roof or floor system designed to transmit lateral forces to shear walls or other lateral-load-resisting elements.

DIMENSIONS.

Actual. The measured dimension of a masonry unit or element.

Nominal. A dimension equal to a specified dimension plus an allowance for the joints with which the units are to be laid. Thickness is given first, followed by height and then length.

Specified. The dimensions specified for the manufacture or construction of masonry, masonry units, joints or any other component of a structure.

EFFECTIVE HEIGHT. For braced members, the effective height is the clear height between lateral supports and is used for calculating the slenderness ratio. The effective height for unbraced members is calculated in accordance with engineering mechanics.

FIREPLACE. A hearth and fire chamber or similar prepared place in which a fire may be made and which is built in conjunction with a chimney.

FIREPLACE THROAT. The opening between the top of the firebox and the smoke chamber.

FOUNDATION PIER. An isolated vertical foundation member whose horizontal dimension measured at right angles to its thickness does not exceed 3 times its thickness and whose height is equal to or less than 4 times its thickness.

GROUTED MASONRY.

Grouted hollow-unit masonry. That form of grouted masonry construction in which certain designated cells of hollow units are continuously filled with grout.

Grouted multiwythe masonry. That form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.

HEAD JOINT. Vertical mortar joint placed between masonry units within the wythe at the time the masonry units are laid.

HEADER (Bonder). A masonry unit that connects two or more adjacent wythes of masonry.

HEIGHT, WALLS. The vertical distance from the foundation wall or other immediate support of such wall to the top of the wall.

MASONRY. A built-up construction or combination of building units or materials of clay, shale, concrete, glass, gypsum, stone or other approved units bonded together with or without mortar or grout or other accepted method of joining.

Ashlar masonry. Masonry composed of various sized rectangular units having sawed, dressed or squared bed surfaces, properly bonded and laid in mortar.

Coursed ashlar. Ashlar masonry laid in courses of stone of equal height for each course, although different courses shall be permitted to be of varying height.

Glass unit masonry. Nonload-bearing masonry composed of glass units bonded by mortar.

Plain masonry. Masonry in which the tensile resistance of the masonry is taken into consideration and the effects of stresses in reinforcement are neglected.

Random ashlar. Ashlar masonry laid in courses of stone set without continuous joints and laid up without drawn patterns. When composed of material cut into modular heights, discontinuous but aligned horizontal joints are discernible.

Reinforced masonry. Masonry construction in which reinforcement acting in conjunction with the masonry is used to resist forces.

Solid masonry. Masonry consisting of solid masonry units laid contiguously with the joints between the units filled with mortar.

Veneer, masonry. A masonry wythe that provides the exterior finish of a wall system and transfer out-of-plane load directly to a backing, but is not considered to add load resisting capacity to the wall system.

MASONRY UNIT. Brick, tile, stone, glass block or concrete block conforming to the requirements specified in Chapter 3.

Clay. A building unit larger in size than a brick, composed of burned clay, shale, fired clay or mixtures thereof.

Concrete. A building unit or block larger in size than 300 mm by 100 mm by 100 mm made of cement and suitable aggregates.

Hollow. A masonry unit whose net cross-sectional area in any plane parallel to the load-bearing surface is less than 75% of its gross cross-sectional area measured in the same plane.

Solid. A masonry unit whose net cross-sectional area in every plane parallel to the load-bearing surface is 75% or more of its gross cross-sectional area measured in the same plane.

MEAN DAILY TEMPERATURE. The average daily temperature of temperature extremes predicted by a local weather bureau for the next 24 hours.

MORTAR. A plastic mixture of approved cementitious materials, fine aggregates and water used to bond masonry or other structural units.

MORTAR, SURFACE-BONDING. A mixture to bond concrete masonry units that contains hydraulic cement, glass fiber reinforcement with or without inorganic fillers or organic modifiers and water.

PLASTIC HINGE. The zone in a structural member in which the yield moment is anticipated to be exceeded under loading combinations that include earthquakes.

PRISM. An assemblage of masonry units and mortar with or without grout used as a test specimen for determining properties of the masonry.

RUBBLE MASONRY. Masonry composed of roughly shaped stones.

Coursed rubble. Masonry composed of roughly shaped stones fitting approximately on level beds and well bonded.

Random rubble. Masonry composed of roughly shaped stones laid without regularity of coursing but well bonded and fitted together to form well-divided joints.

Rough or ordinary rubble. Masonry composed of unsquared field stones laid without regularity of coursing but well bonded.

RUNNING BOND. The placement of masonry units such that head joints in successive courses are horizontally offset at least one-quarter the unit length.

SHEAR WALL.

Detailed plain masonry shear wall. A masonry shear wall designed to resist lateral forces neglecting stresses in reinforcement, and designed in accordance with 6.1.1.2.

Intermediate reinforced masonry shear wall. A masonry shear wall designed to resist lateral forces considering stresses in reinforcement, and designed in accordance with 6.1.1.4.

Ordinary plain masonry shear wall. A masonry shear wall designed to resist lateral forces neglecting stresses in reinforcement, and designed in accordance with 6.1.1.1.

Ordinary reinforced masonry shear wall. A masonry shear wall designed to resist lateral forces considering stresses in reinforcement, and designed in accordance with 6.1.1.3.

Special reinforced masonry shear wall. A masonry shear wall designed to resist lateral forces considering stresses in reinforcement, and designed in accordance with 6.1.1.5.

SHELL. The outer portion of a hollow masonry unit as placed in masonry.

SPECIFIED. Required by construction documents.

SPECIFIED COMPRESSIVE STRENGTH OF MASONRY, $f_m^{'}$. Minimum compressive strength, expressed as force per unit of net cross-sectional area, required of the masonry used in construction by the construction documents, and upon which the project design is based. Whenever the quantity $f_m^{'}$ is under the radical sign, the square root of numerical value only is intended and the result has units of MPa.

STACK BOND. The placement of masonry units in a bond pattern is such that head joints in successive courses are vertically aligned. For the purpose of this code, requirements for stack bond shall apply to masonry laid in other than running bond.

STONE MASONRY. Masonry composed of field, quarried or cast stone units bonded by mortar.

Ashlar stone masonry. Stone masonry composed of rectangular units having sawed, dressed or squared bed surfaces and bonded by mortar.

Rubble stone masonry. Stone masonry composed of irregular-shaped units bonded by mortar.

STRENGTH.

Design strength. Nominal strength multiplied by a strength reduction factor.

Nominal strength. Strength of a member or cross-section calculated in accordance with these provisions before application of any strength-reduction factors.

Required strength. Strength of a member or cross section required to resist factored loads.

TIE, LATERAL. Loop of reinforcing bar or wire enclosing longitudinal reinforcement.

TIE, WALL. A connector that connects wythes of masonry walls together.

TILE. A ceramic surface unit, usually relatively thin in relation to facial area, made from clay or a mixture of clay or other ceramic materials, called the body of the tile, having either a “glazed” or “unglazed” face and fired above red heat in the course of manufacture to a temperature sufficiently high enough to produce specific physical properties and characteristics.

TRANSVERSE REINFORCEMENT. Reinforcement placed perpendicular to the axis of the member.

UNREINFORCED MASONRY. Masonry in which the tensile resistance of masonry is taken into consideration and the resistance of the reinforcing steel is neglected.

TILE, STRUCTURAL CLAY. A hollow masonry unit composed of burned clay, shale, fire clay or mixture thereof, and having parallel cells.

VENEER, ADHERED. Masonry veneer secured to and supported by the backing through adhesion.

VENEER, ANCHORED. Masonry veneer secured to and supported laterally by the backing through anchors and supported vertically by the foundation or other structural elements.

WALL. A vertical element with a horizontal length-to-thickness ratio greater than three, used to enclose space.

Cavity wall. A wall built of masonry units or of concrete, or a combination of these materials, arranged to provide an airspace within the wall, and in which the inner and outer parts of the wall are tied together with metal ties.

Composite wall. A wall built of a combination of two or more masonry units bonded together, one forming the backup and the other forming the facing elements.

Dry-stacked, surface-bonded walls. A wall built of concrete masonry units where the units are stacked dry, without mortar on the bed or head joints, and where both sides of the wall are coated with a surface-bonding mortar.

Masonry-bonded hollow wall. A wall built of masonry units so arranged as to provide an airspace within the wall, and in which the facing and backing of the wall are bonded together with masonry units.

Parapet wall. The part of any wall entirely above the roof line.

WEB. An interior solid portion of a hollow masonry unit as placed in masonry.

WYTHE. Each continuous, vertical section of a wall, one masonry unit in thickness.

SECTION 2.2
NOTATIONS

A_b	=	Cross-sectional area of an anchor bolt, mm^2
A_g	=	Gross cross-sectional area of masonry, mm^2
A_n	=	Net cross-sectional area of masonry, mm^2
A_p	=	Projected area on the masonry surface of a right circular cone for anchor bolt allowable shear and tension calculations, mm^2
A_{pt}	=	Projected area on masonry surface of a right circular cone for calculating tensile breakout capacity of anchor bolts, mm^2
A_{pv}	=	Projected area on masonry surface of one-half of a right circular cone for calculating shear breakout capacity of anchor bolts, mm^2
A_s	=	Effective cross-sectional area of reinforcement, mm^2
A_v	=	Cross-sectional area of shear reinforcement, mm^2
A_1	=	Bearing area, mm^2
A_2	=	Effective bearing area, mm^2
A_{st}	=	Total area of laterally tied longitudinal reinforcing steel in a reinforced masonry column or pilaster, mm^2
α	=	Depth of an equivalent compression zone at nominal strength, mm
B_a	=	Allowable axial force on an anchor bolt, N
B_{an}	=	Nominal axial strength of an anchor bolt, N
B_v	=	Allowable shear force on an anchor bolt, N
B_{vn}	=	Nominal shear strength of an anchor bolt, N
b	=	Effective width of rectangular member or width of flange for T and I sections, mm
b_a	=	Total applied design axial force on an anchor bolt, N
b_{af}	=	Factored axial force in an anchor bolt, N
b_v	=	Total applied design shear force on an anchor bolt, N
b_{vf}	=	Factored shear force in an anchor bolt, N
b_w	=	Width of wall beam, mm
C_d	=	Deflection amplification factor
c	=	Distance from the fiber of maximum compressive strain to the neutral axis, mm
D	=	Dead load or related internal moments and forces
d	=	Distance from extreme compression fiber to centroid of tension reinforcement, mm
d_b	=	Nominal diameter of reinforcement or anchor bolt, mm
d_v	=	Actual depth of masonry in direction of shear considered, mm
E	=	Load effects of earthquake or related internal moments and forces
E_m	=	Modulus of elasticity of masonry in compression, MPa
E_s	=	Modulus of elasticity of steel, MPa
E_v	=	Modulus of rigidity (shear modulus) of masonry, MPa
e	=	Eccentricity of axial load, mm
e_b	=	Projected leg extension of bent-bar anchor measured from inside edge of anchor at bend to farthest point of anchor in the plane of the hook, mm
e_u	=	Eccentricity of P_{uf}, mm

F_α = Allowable compressive stress due to axial load only, MPa
F_b = Allowable compressive stress due to flexure only, MPa
F_s = Allowable tensile or compressive stress in reinforcement, MPa
F_v = Allowable shear stress in masonry, MPa
f_a = Calculated compressive stress in masonry due to axial load only, MPa
$f_g^{'}$ = Specified compressive strength of grout, MPa
$f_m^{'}$ = Specified compressive strength of masonry at age of 28 days, MPa
f_r = Modulus of rupture, MPa
f_s = Calculated tensile or compressive stress in reinforcement, MPa
f_v = Calculated shear stress in masonry, MPa
fy = Specified yield stress of the reinforcement or the anchor bolt, MPa
H = Lateral pressure of soil or related internal moments and forces
h = Effective height of column, wall, or pilaster, mm
I_{cr} = Moment of inertia of cracked cross-sectional area of a member, mm^4
I_{eff} = Effective moment of inertia, mm^4
I_g = Moment of inertia of gross cross-sectional area of a member, mm^4
I_n = Moment of inertia of net cross-sectional area of a member, mm^4
j = Ratio of distance between centroid of flexural compressive forces and centroid of tensile forces to depth, d
K = The lesser of the masonry cover, clear spacing between adjacent reinforcement, or five times d_b, mm
k_c = Coefficient of creep of masonry, per MPa
k_e = Coefficient of irreversible moisture expansion of clay masonry
k_m = Coefficient of shrinkage of concrete masonry
k_t = Coefficient of thermal expansion of masonry per degree Celsius
L = Live load or related internal moments and forces
L_s = Distance between supports, mm
L_w = Length of wall, mm
l = Clear span between supports, mm
l_b = Effective embedment length of plate, headed or bent anchor bolts, mm
l_{be} = Anchor bolt edge distance, measured in the direction of load, from edge of masonry to center of the cross section of anchor bolt, mm
l_d = Required development length of reinforcement, mm
l_{de} = Embedment length of reinforcement, mm
l_e = Equivalent embedment length provided by standard hooks, mm
M = Maximum moment at the section under consideration, N-mm
M_α = Maximum moment in member due to the applied loading for which deflection is computed, N-mm
M_{cr} = Nominal cracking moment strength, N-mm
M_n = Nominal moment strength, N-mm
M_{ser} = Service moment at midheight of a member, including P-delta effects, N-mm
M_u = Factored moment, N-mm
N_v = Compressive force acting normal to shear surface, N
P = Axial load, N
P_a = Allowable compressive force in reinforced masonry due to axial load, N
P_e = Euler buckling load, N
P_n = Nominal axial strength, N
P_u = Factored axial load, N
P_{uf} = Factored load from tributary floor or roof areas, N

P_{uw}	=	Factored weight of wall area tributary to wall section under consideration, N
Q	=	First moment about the neutral axis of a section of that portion of the cross section lying between the neutral axis and extreme fiber, mm^3
R	=	Seismic response modification factor
r	=	Radius of gyration, mm
S_n	=	Section modulus of the net cross-sectional area of a member, mm^3
s	=	Spacing of reinforcement, mm
S_l	=	Total linear drying shrinkage of concrete masonry units determined in accordance with ASTM C 426.
T	=	Forces and moments caused by restraint of temperature, shrinkage, and creep strains or differential movements
t	=	Specified wall thickness dimension or the least lateral dimension of a column, mm
v	=	Shear stress, MPa
V	=	Shear force, N
V_m	=	Shear strength provided by masonry, N
V_n	=	Nominal shear strength, N
V_u	=	Required shear strength due to factored loads, N
V_s	=	Shear strength provided by shear reinforcement, N
W	=	Wind load, or related internal moments and forces
w_u	=	Out-of-plane factored uniformly distributed load, N/mm
β	=	0.25 for fully grouted masonry or 0.15 for other than fully grouted masonry
β_b	=	Ratio of area of reinforcement cut off to total area of tension reinforcement at a section
γ	=	Reinforcement size factor
Δ	=	Calculated story drift, mm
Δ_α	=	Allowable story drift, mm
δ_s	=	Horizontal deflection at midheight under service loads, mm
δ_u	=	Deflection due to factored loads, mm
ε_{mu}	=	Maximum usable compressive strain of masonry
ϕ	=	Strength reduction factor
ρ_{max}	=	Maximum reinforcement ratio
ρ_n	=	Ratio of distributed shear reinforcement on plane perpendicular to plane of A_{mv}

CHAPTER 3
MASONRY CONSTRUCTION MATERIALS

SECTION 3.1
CONCRETE MASONRY UNITS

3.1.0 Concrete masonry units shall conform to the following standards: ASTM C 55 for concrete brick; ASTM C 73 for calcium silicate face brick; ASTM C 90 for load-bearing concrete masonry units or ASTM C 744 for prefaced concrete and calcium silicate masonry units.

SECTION 3.2
CLAY OR SHALE MASONRY UNITS

3.2.0 Clay or shale masonry units shall conform to the following standards: ASTM C 34 for structural clay load-bearing wall tile; ASTM C 56 for structural clay nonload-bearing wall tile; ASTM C 62 for building brick (solid masonry units made from clay or shale); ASTM C 1088 for solid units of thin veneer brick; ASTM C 126 for ceramic-glazed structural clay facing tile, facing brick and solid masonry units; ASTM C 212 for structural clay facing tile; ASTM C 216 for facing brick (solid masonry units made from clay or shale) and ASTM C 652 for hollow brick (hollow masonry units made from clay or shale).

Exception: Structural clay tile for nonstructural use in fire-proofing of structural members and in wall furring shall not be required to meet the compressive strength specifications. The fire-resistance rating shall be determined in accordance with ASTM E 119.

SECTION 3.3
STONE MASONRY UNITS

3.3.0 Stone masonry units shall conform to the following standards: ASTM C 503 for marble building stone (exterior): ASTM C 568 for limestone building stone; ASTM C 615 for granite building stone; ASTM C 616 for sandstone building stone or ASTM C 629 for slate building stone.

SECTION 3.4
CERAMIC TILE

3.4.0 Ceramic tile shall be as defined in, and shall conform to the requirements of ANSI A137.1.

SECTION 3.5
GLASS UNIT MASONRY

3.5.0 Hollow glass units shall be partially evacuated and have a minimum average glass face thickness of 5 mm. Solid glass-block units shall be provided when required. The surfaces of units intended to be in contact with mortar shall be treated with a polyvinyl butyral coating or latex-based paint. Reclaimed units shall not be used.

SECTION 3.6
SECOND-HAND UNITS

3.6.0 Second-hand masonry units shall not be reused unless they conform to the requirements of new units. The units shall be of whole, sound materials and free from cracks and other defects that will interfere with proper laying or use. Old mortar shall be cleaned from the unit before reuse.

SECTION 3.7
MORTAR

3.7.0 Mortar for use in masonry construction shall conform to ASTM C 270 and shall conform to the proportion specifications of Table 3.7.1 or the property specifications of Table 3.7.2. Type S or N mortar shall be used for glass unit masonry. The amount of water used in mortar for glass unit masonry shall be adjusted to account for the lack of absorption. Retempering of mortar for glass unit masonry shall not be permitted after initial set. Unused mortar shall be discarded within 2½ hours after initial mixing except that unused mortar for glass unit masonry shall be discarded within 1½ hours after initial mixing.

TABLE 3.7.1
MORTAR PROPORTIONS

Mortar	Type	PROPORTIONS BY VOLUME (cementitious Materials)							HYDRATED LIME[e] OR LIME PUTTY	AGGREGATE MEASURED IN A DAMP, LOOSE CONDITION
		Portland cement[a] or blended cement[b]	Masonry cement[c]			Mortar cement[d]				
			M	S	N	M	S	N		
Cement-lime	M	1	–	–	–	–	–	–	¼	Not less than 2¼ and not more than 3 times the sum of the separate volumes of cementitious materials
	S	1	–	–	–	–	–	–	Over ¼ to ½	
	N	1	–	–	–	–	–	–	Over ½ to 1¼	
	O	1	–	–	–	–	–	–	Over 1¼ to 2½	
Mortar cement	M	1	–	–	–	–	–	1	–	
	M	–	–	–	–	1	–	–	–	
	S	½	–	–	–	–	–	1	–	
	S	–	–	–	–	–	1	–	–	
	N	–	–	–	–	–	–	1	–	
	O	–	–	–	–	–	–	1	–	
Masonry cement	M	1	–	–	1	–	–	–	–	
	M	–	1	–	–	–	–	–	–	
	S	½	–	–	1	–	–	–	–	
	S	–	–	1	–	–	–	–	–	
	N	–	–	–	1	–	–	–	–	
	O	–	–	–	1	–	–	–	–	

a. Portland cement conforming to the requirements of ASTM C 150.
b. Blended cement conforming to the requirements of ASTM C 595.
c. Masonry cement conforming to the requirements of ASTM C 91.
d. Mortar cement conforming to the requirements of ASTM C 1329.
e. Hydrated lime conforming to the requirements of ASTM C 207.

SECTION 3.8
SURFACE-BONDING MORTAR

3.8.0 Surface-bonding mortar shall comply with ASTM C 887. Surface bonding of concrete masonry units shall comply with ASTM C 946.

SECTION 3.9
MORTARS FOR CERAMIC WALL AND FLOOR TILE

3.9.0 Portland cement mortars for installing ceramic wall and floor tile shall comply with ANSI A108.1A and ANSI A108.1B and be of the compositions indicated in Table 3.9.1.

TABLE 3.7.2
MORTAR PROPERTY[a]

MORTAR	TYPE	AVERAGE COMPRESSIVE[b] STRENGTH AT 28 DAYS Minimum (MPa)	WATER RETENTION Minimum (%)	AIR CONTENT Minimum (%)
Cement-lime	M	17.25	75	12
	S	12.25	75	12
	N	5.0	75	14[c]
	O	2.25	75	14[c]
Mortar cement	M	17.25	75	12
	S	12.25	75	12
	N	5.0	75	14[c]
	O	2.25	75	14[c]
Masonry cement	M	17.25	75	18
	S	12.25	75	1
	N	5.0	75	20[d]
	O	2.25	75	20[d]

a. This aggregate ratio (measured in damp, loose condition) shall not be less than $2^1/_4$ and not more than 3 times the sum of the separate volumes of cementitious materials.
b. In accordance with ASTM C 270.
c. When structural reinforcement is incorporated in cement-lime or mortar cement mortars, the maximum air content shall not exceed 12 %.
d. When structural reinforcement is incorporated in masonry cement mortar, the maximum air content shall not exceed 18 percent.

TABLE 3.9.1
CERAMIC TILE MORTAR COMPOSITIONS

LOCATION	MORTAR	COMPOSITION
Walls	Scratchcoat	1 cement; $^1/_5$ hydrated lime; 4 dry or 5 damp sand
	Setting bed and Leveling coat	1 cement; ½ hydrated lime; 5 damp sand to 1 cement 1 hydrated lime, 7 damp sand
Floors	Setting bed	1 cement; $^1/_{10}$ hydrated lime; 5 dry or 6 damp sand; or 1 cement; 5 dry or 6 damp sand
Ceilings	Scratchcoat and sand bed	1 cement; ½ hydrated lime; 2½ dry sand or 3 damp sand

3.9.1 **Dry-set Portland cement mortars.** Premixed prepared Portland cement mortars, which require only the addition of water and are used in the installation of ceramic tile, shall comply with ANSI A118.1. The shear bond strength for tile set in such mortar shall be as required in accordance with ANSI A118.1. Tile set in dry-set Portland cement mortar shall be installed in accordance with ANSI A108.5.

3.9.2 **Electrically conductive dry-set mortars.** Pre-mixed prepared Portland cement mortars, which require only the addition of water and comply with ANSI A118.2, shall be used in the installation of electrically conductive ceramic tile. Tile set in electrically conductive dry-set mortar shall be installed in accordance with ANSI A108.7.

3.9.3 **Latex-modified Portland cement mortar.** Latex-modified Portland cement thin-set mortars in which latex is added to dry-set mortar as a replacement for all or part of the gauging water that are used for the installation of ceramic tile shall comply with ANSI A118.4. Tile set in latex-modified Portland cement shall be installed in accordance with ANSI A108.5.

3.9.4 **Epoxy mortar.** Ceramic tile set and grouted with chemical-resistant epoxy shall comply with ANSI A118.3. Tile set and grouted with epoxy shall be installed in accordance with ANSI A108.6.

3.9.5 **Furan mortar and grout.** Chemical-resistant furan mortar and grout that are used to install ceramic tile shall comply with ANSI A118.5. Tile set and grouted with furan shall be installed in accordance with ANSI A108.8.

3.9.6 **Modified epoxy-emulsion mortar and grout.** Modified epoxy-emulsion mortar and grout that are used to install ceramic tile shall comply with ANSI A118.8. Tile set and grouted with modified epoxy-emulsion mortar and grout shall be installed in accordance with ANSI A108.9.

3.9.7 **Organic adhesives.** Water-resistant organic adhesives used for the installation of ceramic tile shall comply with ANSI A136.1. The shear bond strength after water immersion shall not be less than 275 kPa for Type I adhesive, and not less than 140 kPa for Type II adhesive, when tested in accordance with ANSI A136.1. Tile set in organic adhesives shall be installed in accordance with ANSI A108.4.

3.9.8 **Portland cement grouts.** Portland cement grouts used for the installation of ceramic tile shall comply with ANSI A118.6. Portland cement grouts for tile work shall be installed in accordance with ANSI A108.10.

SECTION 3.10
GROUT

3.10.0 Grout shall conform to Table 3.10.1 or to ASTM C 476. When grout conforms to ASTM C 476, the grout shall be specified by proportion requirements or property requirements.

TABLE 3.10.1
GROUT PROPORTIONS BY VOLUME FOR MASONRY CONSTRUCTION

TYPE	PARTS BY VOLUME OF PORTLAND CEMENT OR BLENDED CEMENT	PARTS BY VOLUME OF HYDRATED LIME OR LIME PUTTY	AGGREGATE, MEASURED IN A DAMP, LOOSE CONDITION	
			Fine	Coarse
Fine grout	1	0-$^{1}/_{10}$	2¼-3 times the sum of the volumes of the cementitious materials	–
Coarse grout	1	0-$^{1}/_{10}$	2¼-3 times the sum of the volumes of the cementitious materials	1-2 times the sum of the volumes of the cementitious materials

SECTION 3.11
METAL REINFORCEMENT AND ACCESSORIES

3.11.0 Metal reinforcement and accessories shall conform to Section 3.11.1 through Section 3.11.6 in conjunction with SBC 304 Appendix F.

3.11.1 **Deformed reinforcing bars.** Deformed reinforcing bars shall conform to one of the following standards: ASTM A 615 for deformed and plain billet-steel bars for concrete reinforcement; ASTM A 706 for low-alloy steel deformed bars for concrete reinforcement; ASTM A 767 for zinc-coated reinforcing steel bars; ASTM A 775 for epoxy-coated reinforcing steel bars and ASTM A 996 for rail steel and axle steel deformed bars for concrete reinforcement.

3.11.2 **Joint reinforcement.** Joint reinforcement shall comply with ASTM A 951. The maximum spacing of cross-wires in ladder-type joint reinforcement and of point of connection of cross wires to longitudinal wires of truss-type reinforcement shall be 400 mm.

3.11.3 **Deformed reinforcing wire.** Deformed reinforcing wire shall conform to ASTM A 496.

3.11.4 **Wire fabric.** Wire fabric shall conform to ASTM A 185 for plain steel-welded wire fabric for concrete reinforcement or ASTM A 496 for welded deformed steel wire fabric for concrete reinforcement.

3.11.5 **Anchors, ties and accessories.** Anchors, ties and accessories shall conform to the following standards: ASTM A 36 for structural steel; ASTM A 82 for plain steel wire for concrete reinforcement; ASTM A 185 for plain steel-welded wire fabric for concrete reinforcement; ASTM A 167, Type 304, for stainless and heat-resisting chromium-nickel steel plate, sheet and strip and ASTM A 366 for cold-rolled carbon steel sheet, commercial quality.

3.11.6 **Corrosion protection.** Corrosion protection for carbon steel accessories used in exterior wall construction or interior walls exposed to a mean relative humidity exceeding 75% shall comply with either Section 3.11.7.1 or Section 3.11.7.2. Corrosion protection for carbon steel accessories used in interior walls exposed to a mean relative humidity equal to or less than 75% shall comply with either Section 3.11.7.1, 3.11.7.2 or Section 3.11.7.3.

3.11.6.1 **Hot-dipped galvanized.** Apply a hot-dipped galvanized coating after fabrication as follows:

1. For joint reinforcement, wall ties, anchors and inserts, apply a minimum coating of 460 g/m^2 complying with the requirements of ASTM A 153, Class B.
2. For sheet metal ties and sheet metal anchors, comply with the requirements of ASTM A 153, Class B.
3. For steel plates and bars, comply with the requirements of either ASTM A 123 or ASTM A 153, Class B.

3.11.6.2 **Epoxy coatings.** Carbon steel accessories shall be epoxy coated as follows:

1. For joint reinforcement, comply with the requirements of ASTM A 884 Class B, Type 2 – 460 μm .

2. For wire ties and anchors, comply with the requirements of ASTM A 899 Class C – 510 μm .

3. For sheet metal ties and anchors, provide a minimum thickness of 510 μm or in accordance with the manufacturer's specification.

3.11.6.3 **Mill galvanized.** Apply a mill galvanized coating as follows:

1. For joint reinforcement, wall ties, anchors and inserts, apply a minimum coating of 30 g/m^2 complying with the requirements of ASTM A 641.
2. For sheet metal ties and sheet metal anchors, apply a minimum coating complying with Coating Designation G-60 according to the requirements of ASTM A 653.
3. For anchor bolts, steel plates or bars not exposed to the earth, weather or a mean relative humidity exceeding 75 percent, a coating is not required.

3.11.7 **Tests.** Where unidentified reinforcement is approved for use, not less than three tension and three bending tests shall be made on representative specimens of the reinforcement from each shipment and grade of reinforcing steel proposed for use in the work.

SECTION 3.12
MATERIAL PROPERTIES

3.12.1 **General.** Unless otherwise determined by test, the following moduli and coefficients shall be used in determining the effects of elasticity, temperature, moisture expansion, shrinkage, and creep.

3.12.2 **Elastic moduli**

Steel reinforcement

$$E_s = 200{,}000 \text{ MPa} \quad \textbf{Eq. (3-1)}$$

Clay and concrete masonry

The design of clay and concrete masonry shall be based on the following moduli of elasticity values:

$$E_m = 700\, f_m' \text{ for clay masonry;} \quad \textbf{Eq. (3-2)}$$

$$E_m = 900\, f_m' \text{ for concrete masonry;} \quad \textbf{Eq. (3-3)}$$

or the chord modulus of elasticity taken between 0.05 and 0.33 of the maximum compressive strength of each prism determined by test in accordance with the prism test method, (Article 1.4 B.3 of AC1 530.1/ASCE 6/TMS 602, and ASTM E 111.)

$$E_v = 0.4\, E_m \quad \textbf{Eq. (3-4)}$$

Grout — Modulus of elasticity of grout shall be determined by the expression $500\, f_g'$.

3.12.3 **Thermal expansion coefficients**

Clay masonry

$$k_t = 7.2 \times 10^{-6} \text{ mm/mm/°C} \quad \textbf{Eq. (3-5)}$$

Concrete masonry

$$k_t = 8.1 \times 10^{-6} \text{ mm/mm/°C} \qquad \textbf{Eq.(3-6)}$$

3.12.4 Moisture expansion coefficients of clay masonry

$$k_e = 3 \times 10^{-4} \text{ mm/mm} \qquad \textbf{Eq. (3-7)}$$

3.12.5 Shrinkage coefficients of concrete masonry

Masonry made of moisture-controlled concrete masonry units:

$$k_m = 0.15\ S_l \qquad \textbf{Eq. (3-8)}$$

where S_l is not more than 6.5 mm x 10^{-4} mm/mm.

Masonry made of non-moisture-controlled concrete masonry units:

$$k_m = 0.5\ S_l \qquad \textbf{Eq. (3-9)}$$

3.12.6 Creep coefficients

Clay masonry

$$k_c = 0.1 \times 10^{-4}, \text{ per MPa} \qquad \textbf{Eq. (3-10)}$$

Concrete masonry

$$k_c = 0.36 \times 10^{-4}, \text{ per MPa} \qquad \textbf{Eq. (3-11)}$$

SECTION 3.13
SECTION PROPERTIES

3.13.1 Stress computations

3.13.1.1 Member design shall be computed using section properties based on the minimum net cross-sectional area of the member under consideration. Section properties shall be based on specified dimensions.

3.13.1.2 In members designed for composite action, stresses shall be computed using section properties based on the minimum transformed net cross-sectional area concept for elastic analysis, in which areas of dissimilar materials are transformed in accordance with relative elastic moduli ratios shall apply. Actual stresses shall be used to verify compliance with allowable stress requirements.

3.13.2 Stiffness. Determination of stiffness based on uncracked section is permissible. Use of the average net cross-sectional area of the member considered in stiffness computations is permitted.

3.13.3 Radius of gyration. Radius of gyration shall be computed using average net cross-sectional area of the member considered.

3.13.4 Intersecting walls

3.13.4.1 Wall intersections shall meet one of the following requirements:

(a) Design shall conform to the provisions of Section 3.13.4.2.

(b) Transfer of shear between walls shall be prevented.

3.13.4.2 Design of wall intersection

3.13.4.2.1 Masonry shall be in running bond.

3.13.4.2.2 Flanges shall be considered effective in resisting applied loads.

3.13.4.2.3 The width of flange considered effective on each side of the web shall be the lesser of 6 times the flange thickness or the actual flange on either side of the web wall.

3.13.4.2.4 Design for shear, including the transfer of shear at interfaces, shall conform to the requirements of Section 7.2.5 or 7.3.5.

3.13.4.2.5 The connection of intersecting walls shall conform to one of the following requirements:

(a) Fifty percent of the masonry units at the interface shall interlock.

(b) Walls shall be anchored by steel connectors grouted into the wall and meeting the following requirements:

(1) Minimum size: 6.5 mm x 38 mm x 700 mm including 50 mm long 90 degree bend at each end to form a U or Z shape.

(2) Maximum spacing: 1.20 m.

(c) Intersecting bond beams shall be provided in intersecting walls at a maximum spacing of 1.20 m on centers. Bond beams shall be reinforced and the area of reinforcement shall not be less than 200 mm^2/m of wall. Reinforcement shall be developed on each side of the intersections.

CHAPTER 4
CONSTRUCTION

SECTION 4.1
MASONRY CONSTRUCTION

4.1.0 Masonry construction shall comply with the requirements of Section 4.1.1 through Section 4.8.6 and with ACI 530.1/ASCE 6/TMS 602.

4.1.1 **Tolerances.** Masonry, except masonry veneer, shall be constructed within the tolerances specified in ACI 530.1/ASCE 6/TMS 602.

4.1.2 **Placing mortar and units.** Placement of mortar and units shall comply with Sections 4.1.2.1 through 4.1.2.5.

4.1.2.1 **Bed and head joints.** Unless otherwise required or indicated on the construction documents, head and bed joints shall be 10 mm thick, except that the thickness of the bed joint of the starting course placed over foundations shall not be less than 6 mm and not more than 20 mm.

4.1.2.1.1 **Open-end units.** Open-end units with beveled ends shall be fully grouted. Head joints of open-end units with beveled ends need not be mortared. The beveled ends shall form a grout key that permits grouts within 15 mm of the face of the unit. The units shall be tightly butted to prevent leakage of the grout.

4.1.2.2 **Hollow units.** Hollow units shall be placed such that face shells of bed joints are fully mortared. Webs shall be fully mortared in all courses of piers, columns, pilasters, in the starting course on foundations where adjacent cells or cavities are to be grouted, and where otherwise required. Head joints shall be mortared a minimum distance from each face equal to the face shell thickness of the unit.

4.1.2.3 **Solid units.** Unless otherwise required or indicated on the construction documents, solid units shall be placed in fully mortared bed and head joints. The ends of the units shall be completely buttered. Head joints shall not be filled by slushing with mortar. Head joints shall be constructed by shoving mortar tight against the adjoining unit. Bed joints shall not be furrowed deep enough to produce voids.

4.1.2.4 **Glass unit masonry.** Glass units shall be placed so head and bed joints are filled solidly. Mortar shall not be furrowed.

Unless otherwise required, head and bed joints of glass unit masonry shall be 6 mm thick, except that vertical joint thickness of radial panels shall not be less than 3 mm. The bed joint thickness tolerance shall be minus 2 mm and plus 3 mm. The head joint thickness tolerance shall be plus or minus 3 mm.

4.1.2.5 **All units.** Units shall be placed while the mortar is soft and plastic. Any unit disturbed to the extent that the initial bond is broken after initial positioning shall be removed and re-laid in fresh mortar.

4.1.2.6 **Adhered veneer.** Tap the veneer unit into place, completely filling the space between the veneer unit and the backing. Sufficient mortar shall be used to create a slight excess to be forced out between the edges of the veneer units. The resulting thickness of the mortar in back of the veneer unit shall not be less than 10 mm nor more than 30 mm.

4.1.3 **Installation of wall ties.** The ends of wall ties shall be embedded in mortar joints. Wall tie ends shall engage outer face shells of hollow units by at least 12 mm.

Wire wall ties shall be embedded at least 38 mm into the mortar bed of solid masonry units or solid-grouted hollow units. Wall ties shall not be bent after being embedded in grout or mortar.

4.1.4 **Chases and recesses.** Chases and recesses shall be constructed as masonry units are laid. Masonry directly above chases or recesses wider than 300 mm shall be supported on lintels.

4.1.5 Deflection

4.1.5.1 **Deflection of beams and lintels.** Deflection of beams and lintels due to dead plus live loads shall not exceed the lesser of 8 mm when providing vertical support to masonry designed in accordance with Chapter 7 or Chapter 8. Minimum length of end support shall be 100 mm.

4.1.5.2 **Connection to structural frames.** Masonry walls shall not be connected to structural frames unless the connections and walls are designed to resist design interconnecting forces and to accommodate calculated deflections.

4.1.6 **Support on wood.** Masonry shall not be supported on wood girders or other forms of wood construction.

4.1.7 **Masonry protection.** The top of unfinished masonry work shall be covered to protect the masonry from the weather.

4.1.8 **Weep holes.** Weep holes provided in the outside wythe of masonry walls shall be at a maximum spacing of 850 mm on center. Weep holes shall not be less than 5 mm in diameter.

4.1.9 Stack bond masonry

For masonry in other than running bond, the minimum area of horizontal reinforcement shall be 0.00028 times the gross vertical cross-sectional area of the wall using specified dimensions. Horizontal reinforcement shall be placed in horizontal joints or in bond beams spaced not more than 1200 mm on center.

SECTION 4.2
CORBELED MASONRY

4.2.0 The maximum corbeled projection beyond the face of the wall shall not be more than one-half of the wall thickness nor one-half the wythe thickness for hollow walls. The maximum projection of one unit shall neither exceed one-half the height of the unit nor one-third the thickness at right angles to the wall.

4.2.1 **Molded cornices.** Unless structural support and anchorage are provided to resist the overturning moment, the center of gravity of projecting masonry or molded cornices shall lie within the middle one-third of the supporting wall. Terra cotta and metal cornices shall be provided with a structural frame of approved noncombustible material anchored in an approved manner.

SECTION 4.3
COLD WEATHER CONSTRUCTION

4.3.0 The cold weather construction provisions of ACI 530.1/ASCE 6/TMS 602, Article 1.8 C, or the following procedures shall be implemented when either the ambient temperature falls below 4°C or the temperature of masonry units is below 4°C.

4.3.1 **Preparation.**

1. Temperatures of masonry units shall not be less than -7°C when laid in the masonry. Masonry units containing frozen moisture, visible ice or snow on their surface shall not be laid.
2. Visible ice and snow shall be removed from the top surface of existing foundations and masonry to receive new construction. These surfaces shall be heated to above freezing, using methods that do not result in damage.

4.3.2 **Construction.** The following requirements shall apply to work in progress and shall be based on ambient temperature.

4.3.2.1 **Construction requirements for temperatures between 4°C and 0°C.** The following construction requirements shall be met when the ambient temperature is between 4°C and 0°C:

1. Glass unit masonry shall not be laid.
2. Water and aggregates used in mortar and grout shall not be heated above 60°C.
3. Mortar sand or mixing water shall be heated to produce mortar temperatures between 4°C and 49°C at the time of mixing. When water and aggregates for grout are below 0°C, they shall be heated.

4.3.2.2 **Construction requirements for temperatures between 0°C and -4°C.** The requirements of Section 4.3.2.1 and the following construction requirements shall be met when the ambient temperature is between 0°C and -4°C:

1. The mortar temperature shall be maintained above freezing until used in masonry.
2. Aggregates and mixing water for grout shall be heated to produce grout temperature between 21°C and 49°C at the time of mixing. Grout temperature shall be maintained above 21°C at the time of grout placement.

4.3.2.3 **Construction requirements for temperatures between -4°C and -7°C.** The requirements of Section 4.3.2.1 and 4.3.2.2 and the following construction requirements shall be met when the ambient temperature is between -4°C and -7°C:

1. Masonry surfaces under construction shall be heated to 4°C.
2. Wind breaks or enclosures shall be provided when the wind velocity exceeds 24 km/h.
3. Prior to grouting, masonry shall be heated to a minimum of 4°C.

4.3.2.4 **Construction requirements for temperatures below -7 ° C.** The requirements of Section 4.3.2.1, 4.3.2.2 and 4.3.2.3 and the following construction requirement shall be met when the ambient temperature is below -7 ° C. Enclosures and auxiliary heat shall be provided to maintain air temperature within the enclosure to above 0 ° C.

4.3.3 **Protection.** The requirements of this section and Section 4.3.3.1 through 4.3.3.4 apply after the masonry is placed and shall be based on anticipated minimum daily temperature for grouted masonry and anticipated mean daily temperature for ungrouted masonry.

4.3.3.1 **Glass unit masonry.** The temperature of glass unit masonry shall be maintained above 4 ° C for 48 hours after construction.

4.3.3.2 **Protection requirements for temperatures between 4 ° C and -4 ° C.** When the temperature is between 4 ° C and -4 ° C, newly constructed masonry shall be covered with a weather-resistive membrane for 24 hours after being completed.

4.3.3.3 **Protection requirements for temperatures between -4 ° C and -7 ° C.** When the temperature is between -4 ° C and -7 ° C, newly constructed masonry shall be completely covered with weather-resistive insulating blankets, or equal protection, for 24 hours after being completed. The time period shall be extended to 48 hours for grouted masonry, unless the only cement in the grout is Type III Portland cement.

4.3.3.4 **Protection requirements for temperatures below -7 ° C.** When the temperature is below -7 ° C, newly constructed masonry shall be maintained at a temperature above 0 ° C for at least 24 hours after being completed by using heated enclosures, electric heating blankets, infrared lamps or other acceptable methods. The time period shall be extended to 48 hours for grouted masonry, unless the only cement in the grout is Type III Portland cement.

SECTION 4.4
HOT WEATHER CONSTRUCTION

4.4.0 The hot weather construction provisions of ACI 530.1/ASCE 6/TMS 602, Article 1.8 D, or the following procedures shall be implemented when the temperature or the temperature and wind-velocity limits of this section are exceeded.

4.4.1 **Preparation.** The following requirements shall be met prior to conducting masonry work.

4.4.1.1 **Temperature.** When the ambient temperature exceeds 38 ° C, or exceeds 32 ° C with a wind velocity greater than 13 km/h:

1. Necessary conditions and equipment shall be provided to produce mortar having a temperature below 49 ° C.
2. Sand piles shall be maintained in a damp, loose condition.

4.4.1.2 **Special conditions.** When the ambient temperature exceeds 46 ° C, or 40 ° C with a wind velocity greater than 13 km/h, the requirements of Section 4.4.1.1 shall be implemented, and materials and mixing equipment shall be shaded from direct sunlight.

4.4.2 **Construction.** The following requirements shall be met while masonry work is in progress.

4.4.2.1 **Temperature.** When the ambient temperature exceeds 38 ° C, or exceeds 32 ° C with a wind velocity greater than 13 km/h:

1. The temperature of mortar and grout shall be maintained below 49 ° C.
2. Mixers, mortar transport containers and mortar boards shall be flushed with cool water before they come into contact with mortar ingredients or mortar.
3. Mortar consistency shall be maintained by retempering with cool water.
4. Mortar shall be used within 2 hours of initial mixing.

4.4.2.2 **Special conditions.** When the ambient temperature exceeds 46 ° C, or exceeds 40 ° C with a wind velocity greater than 13 km/h, the requirements of Section 4.4.2.1 shall be implemented and cool mixing water shall be used for mortar and grout. The use of ice shall be permitted in the mixing water prior to use. Ice shall not be permitted in the mixing water when added to the other mortar or grout materials.

4.4.3 **Protection.** When the mean daily temperature exceeds 38 ° C, or exceeds 32 ° C with a wind velocity greater than 13 km/h, newly constructed masonry shall be fog sprayed until damp at least three times a day until the masonry is three days old.

SECTION 4.5
WETTING OF BRICK

4.5.0 Brick (clay or shale) at the time of laying shall require wetting if the unit's initial rate of water absorption exceeds 30 grams per 20,000 mm^2 per minute or 1 g/645 mm^2, as determined by ASTM C 67.

SECTION 4.6
GROUTING, MINIMUM SPACES

4.6.0 The minimum dimensions of spaces provided for the placement of grout shall be in accordance with Table 4.6.1. Higher group pours, higher grout lifts, smaller cavity widths, or smaller cell sizes than those shown in Table 4.6.1 are permitted if the results of a grout demonstration panel show that the grout spaces are filled and adequately consolidated. In that case, the procedures used in constructing the grout demonstration panel shall be the minimum acceptable standard for grouting, and the quality assurance program shall include inspection during construction to verify grout placement.

SECTION 4.7
EMBEDDED CONDUITS, PIPES, AND SLEEVES

4.7.0 Conduits pipes and sleeves of any material to be embedded in masonry shall be compatible with masonry and shall comply with the following requirements.

4.7.1 Design shall not consider conduits, pipes, or sleeves as structurally replacing the displaced masonry.

4.7.2 Design shall consider the structural effects resulting from the removal of masonry to allow for the placement of pipes or conduits.

4.7.3 Conduits, pipes, and sleeves in masonry shall be no closer than 3 diameters on center.

4.7.4 Maximum area of vertical conduits, pipes, or sleeves placed in masonry columns or pilasters shall not displace more than 2% of the net cross section.

4.7.5 Pipes shall not be embedded in masonry when:

(a) Containing liquid, gas, or vapors at temperature higher than 66 $^{\circ}$C.

(b) Under pressure in excess of 380 kPa.

(c) Containing water or other liquids subject to freezing.

TABLE 4.6.1
Grout Space Requirements

Grout type[1]	Maximum grout pour height, m	Minimum width of grout space, [2,3] mm	Minimum grout space dimensions for grouting cells of hollow units, [3,4] mm x mm
Fine	0.3	20	38 x 51
Fine	1.5	50	50 x 76
Fine	3.7	60	63 x 76
Fine	7.3	75	76 x 76
Coarse	0.3	38	38 x 76
Coarse	1.5	50	63 x 76
Coarse	3.7	60	76 x 76
Coarse	7.3	75	76 x 102

1 Fine and coarse grouts are defined in ASTM C 476.
2 For grouting between masonry wythes.
3 Grout space dimension is the clear dimension between any masonry protrusion and shall be increased by the diameters of the horizontal bars within the cross section of the grout space.
4 Area of vertical reinforcement shall not exceed 6 percent of the area of the grout space.

SECTION 4.8
REINFORCEMENT

4.8.1 **Embedment**
Reinforcing bars shall be embedded in grout.

4.8.2 **Size of reinforcement**

4.8.2.1 The maximum size of reinforcement used in masonry shall be (Dia 36).

4.8.2.2 The diameter of reinforcement shall not exceed one-half the least clear dimension of the cell, bond beam, or collar joint in which it is placed.

4.8.2.3 Longitudinal and cross wires of joint reinforcement shall have a minimum wire size of (WD 4) and a maximum wire size of one half the joint thickness.

4.8.3 Placement of reinforcement

4.8.3.1 The clear distance between parallel bars shall not be less than the nominal diameter of the bars, nor less than 25 mm.

4.8.3.2 In columns and pilasters, the clear distance between vertical bars shall not be less than one and one-half times the nominal bar diameter, nor less than 40 mm.

4.8.3.3 The clear distance limitations between bars required in Section 4.8.3.1 and 4.8.3.2 shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

4.8.3.4 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to two in any one bundle. Individual bars in a bundle cut off within the span of a member shall terminate at points at least 40 bar diameters apart.

4.8.3.5 Reinforcement embedded in grout shall have a thickness of grout between the reinforcement and masonry units not less than 6 mm for fine grout or 13 mm for coarse grout.

4.8.4 Protection of reinforcement

4.8.4.1 Reinforcing bars shall have a masonry cover not less than the following:

- **(a)** Masonry face exposed to earth or weather: 50 mm for bars larger than (Dia 16); 40 mm for (Dia 16) bars or smaller.
- **(b)** Masonry not exposed to earth or weather: 40 mm.

4.8.4.2 Longitudinal wires of joint reinforcement shall be fully embedded in mortar or grout with a minimum cover of 16 mm when exposed to earth or weather and 13 mm when not exposed to earth or weather. Joint reinforcement shall be stainless steel or protected from corrosion by hot-dipped galvanized coating or epoxy coating when used in masonry exposed to earth or weather and in interior walls exposed to a mean relative humidity exceeding 75%. All other joint reinforcement shall be mill galvanized, hot-dip galvanized, or stainless steel.

4.8.4.3 Wall ties, sheet metal anchors, steel plates and bars, and inserts exposed to earth or weather, or exposed to a mean relative humidity exceeding 75% shall be stainless steel or protected from corrosion by hot-dip galvanized coating or epoxy coating. Wall ties, anchors, and inserts shall be mill galvanized, hot-dip galvanized, or stainless steel for all other cases. Anchor bolts, steel plates, and bars not exposed to earth, weather, nor exposed to a mean relative humidity exceeding 75%, need not be coated.

4.8.5 Standard hooks

- **(a)** A 180 degree turn plus extension of at least 4 bar diameters but not less than 64 mm at free end of bar.
- **(b)** A 90 degree turn plus extension of at least 12 bar diameters at free end of bar.
- **(c)** For stirrup and tie anchorage only, either a 90 degree or a 135 degree turn plus an extension of at least 6 bar diameters at the free end of the bar.

4.8.6 Minimum bend diameter for reinforcing bars

The diameter of bend measured on the inside of reinforcing bars, other than for stirrups and ties, shall not be less than values specified in Table 4.8.1.

Table 4.8.1
Minimum diameters of bend

Bar size and type	Minimum diameter
(Dia 10) through (Dia 22) Grade 300	5 bar diameters
(Dia 10) through (Dia 25) Grade 350 or 420	6 bar diameters
(Dia 28), (Dia 32), and (Dia 36) Grade 350 or 420	8 bar diameters

CHAPTER 5
QUALITY ASSURANCE

SECTION 5.1
GENERAL

5.1.0 A quality assurance program shall be used to ensure that the constructed masonry is in compliance with the construction documents.

The minimum quality assurance program shall comply with the requirements of this chapter and Table 5.1.1. The quality assurance program shall itemize the methods used to verify conformance of material composition, quality, storage, handling, preparation, and placement with the requirements of ACI 530.1/ASCE 6/TMS 602.

5.1.1 The quality assurance program shall define the qualifications for testing laboratories and for inspection agencies and shall comply with the inspection and testing requirements of SBC 302.

TABLE 5.1.1
QUALITY ASSURANCE

MINIMUM TESTS AND SUBMITTALS	MINIMUM INSPECTION
Certificates for materials used in masonry construction indicating compliance with the contract documents. Verification of f'_m • Prior to construction • Every 500 m^2 during construction Verification of proportions of materials in mortar and grout as delivered to the site.	From the beginning of masonry construction and continuously during construction of masonry, verify the following are in compliance: • proportions of site-mixed mortar, grout, and prestressing grout for bonded tendons • grade and size of reinforcement, prestressing tendons and anchorages • placement of masonry units and construction of mortar joints • placement of reinforcement, connectors, and prestressing tendons and anchorages • grout space prior to grouting • placement of grout and prestressing grout for bonded tendons Observe preparation of grout specimens, mortars specimens, and/or prisms. Verify compliance with the required inspection provisions of the contract documents and the approved submittals.

SECTION 5.2
ACCEPTANCE RELATIVE TO STRENGTH REQUIREMENTS

5.2.1 **Compliance with f'_m.** Compressive strength of masonry shall be considered satisfactory if the compressive strength of each masonry wythe and grouted collar joint equals or exceeds the value of f'_m.

5.2.2 **Determination of compressive strength.** The compressive strength for each wythe shall be determined by the unit strength method or by the prism test method as specified herein.

5.2.2.1 Unit strength method.

5.2.2.1.1 Clay masonry. The compressive strength of masonry shall be determined based on the strength of the units and the type of mortar specified using Table 5.2.1, provided:

1. Units conform to ASTM C 62, ASTM C 216 or ASTM C 652 and are sampled and tested in accordance with ASTM C 67.
2. Thickness of bed joints does not exceed 16 mm.
3. For grouted masonry, the grout meets one of the following requirements:
    a. Grout conforms to ASTM C 476.
    b. Minimum grout compressive strength equals $f_m^{'}$ but not less than 14 MPa. The compressive strength of grout shall be determined in accordance with ASTM C 1019.

TABLE 5.2.1
COMPRESSIVE STRENGTH OF CLAY MASONRY

NET AREA COMPRESSIVE STRENGTH OF CLAY MASONRY UNITS (MPa)		NET AREA COMPRESSIVE STRENGTH OF MASONRY (MPa)
Type M or S mortar	Type N mortar	
12	14.5	7
23	29	10.5
34	43	14
46	57	17.5
57	71	21
68	–	24.5
91	–	28

5.2.2.1.2 Concrete masonry. The compressive strength of masonry shall be determined based on the strength of the unit and type of mortar specified using Table 5.2.2, provided:

1. Units conform to ASTM C 55, ASTM C 90 and are sampled and tested in accordance with ASTM C 140.
2. Thickness of bed joints does not exceed 16 mm.
3. For grouted masonry, the grout meets one of the following requirements:
    a. Grout conforms to ASTM C 476.
    b. Minimum grout compressive strength equals $f_m^{'}$ but not less than 14 MPa. The compressive strength of grout shall be determined in accordance with ASTM C 1019.

5.2.2.2 Prism test method.

5.2.2.2.1 General. The compressive strength of masonry shall be determined by the prism test method:

1. Where specified in the construction documents.
2. Where masonry does not meet the requirements for application of the unit strength method in Section 5.2.2.1.

5.2.2.2.2 **Number of prisms per test.** A prism test shall consist of three prisms constructed and tested in accordance with ASTM C 1314.

TABLE 5.2.2
COMPRESSIVE STRENGTH OF CONCRETE MASONRY

NET AREA COMPRESSIVE STRENGTH OF CLAY MASONRY UNITS (MPa)		NET AREA COMPRESSIVE STRENGTH OF MASONRY (MPa)[a]
Type M or S mortar	Type N mortar	
9	9	7
13	15	10.5
19	21	14
26	28	17.5
33	36	21

a. For units of less than 100 mm in height, 85 percent of the values listed.

SECTION 5.3
TESTING PRISMS FROM CONSTRUCTED MASONRY

5.3.0 When approved by the building official, acceptance of masonry that does not meet the requirements of Section 5.2.2.1 or 5.2.2.2 shall be permitted to be based on tests of prisms cut from the masonry construction in accordance with Section 5.3.1, 5.3.2 and 5.3.3.

5.3.1 **Prism sampling and removal.** A set of three masonry prisms that are at least 28 days old shall be saw cut from the masonry for each 500 m^2 of the wall area that is in question but not less than one set of three masonry prisms for the project. The length, width and height dimensions of the prisms shall comply with the requirements of ASTM C 1314. Transporting, preparation and testing of prisms shall be in accordance with ASTM C 1314.

5.3.2 **Compressive strength calculations.** The compressive strength of prisms shall be the value calculated in accordance ASTM C 1314, except that the net cross-sectional area of the prism shall be based on the net mortar bedded area.

5.3.3 **Compliance.** Compliance with the requirement for the specified compressive strength of masonry, f_m' shall be considered satisfied provided the modified compressive strength equals or exceeds the specified f_m'. Additional testing of specimens cut from locations in question shall be permitted.

CHAPTER 6
SEISMIC DESIGN

SECTION 6.1
SEISMIC DESIGN REQUIREMENTS FOR MASONRY

6.1.0 Masonry structures and components shall comply with the requirements in Section 6.1.1, 6.1.2, 6.1.3, 6.1.4, 6.1.5 depending on the structure's seismic design category as determined in SBC 301. All masonry walls, unless isolated on three edges from in-plane motion of the basic structures systems, shall be considered to be part of the seismic-force-resisting system.

6.1.1 **Basic seismic-force-resisting system –** Buildings relying on masonry shear walls as part of the Basic seismic-force-resisting system shall have shear walls that comply with the requirements of Sections 6.1.1.1, 6.1.1.2, 6.1.1.3, 6.1.1.4 and 6.1.1.5.

Exception: Buildings assigned to Seismic Design Category A shall be permitted to have shear walls complying with Section 9.2.

6.1.1.1 **Ordinary plain (unreinforced) masonry shear walls** – Design of ordinary plain (unreinforced) masonry shear walls shall comply with the requirements of Section 7.2 and 8.3.

6.1.1.2 **Detailed plain (unreinforced) masonry shear walls** – Design of detailed plain (unreinforced) masonry shear walls shall comply with the requirements of Section 7.2 or 8.3, and shall comply with the requirements of Section 6.1.1.2.1 and 6.1.1.2.2.

6.1.1.2.1 **Minimum reinforcement requirements** – Vertical reinforcement of at least 130 mm^2 in cross-sectional area shall be provided at corners, within 400 mm of each side of openings, within 200 mm of each side of movement joints, within 200 mm of the ends of walls, and at a maximum spacing of 3.0 m on center.

Reinforcement adjacent to openings need not be provided for opening smaller than 400 mm in either the horizontal or vertical direction, unless the spacing of distributed reinforcement is interrupted by such openings.

Horizontal joint reinforcement shall consist of at least two wires of WD 4 spaced not more than 400 mm; or bond beam reinforcement shall be provided of at least 130 mm^2 in cross-sectional area spaced not more than 3.0 m. Horizontal reinforcement shall also be provided at the bottom and top of wall openings and shall extend not less than 600 mm nor less than 40 bar diameters past the opening; continuously at structurally connected roof and floor levels; and within 400 mm of the top of walls.

6.1.1.2.2 **Connections** – Connectors shall be provided to transfer forces between masonry walls and horizontal elements in accordance with the requirements of Section 7.1.8. Connectors shall be designed to transfer horizontal design forces acting either perpendicular or parallel to the wall, but not less than 3000 N per lineal m of wall. The maximum spacing between connectors shall be 1.20 m.

6.1.1.3 **Ordinary reinforced masonry shear walls** – Design of ordinary reinforced masonry shear walls shall comply with the requirements of reinforced Masonry in Section 7.3 or 8.2, and shall comply with the requirements of Section 6.1.1.2.1 and 6.1.1.2.2.

6.1.1.4 **Intermediate reinforced masonry shear walls** – Design of intermediate reinforced masonry shear walls shall comply with the requirements of reinforced Masonry in Section 7.3 or 8.2. Design shall also comply with the requirements of Section 6.1.1.2.1 and 6.1.1.2.2, except that the spacing of vertical reinforcement in intermediate reinforced masonry shear walls shall not exceed 1200 mm.

6.1.1.5 **Special reinforced masonry shear walls** – Design of special reinforced masonry shear walls shall comply with the requirements of reinforced Masonry in Section 7.3 or 8.2. Design shall also comply with the requirements of Sections 6.1.1.2.1, 6.1.1.2.2, 6.1.5.3, and the following:

- **(a)** The maximum spacing of vertical and horizontal reinforcement shall be the smaller of; one-third the length of the shear wall; one-third the height of the shear wall; or 1200 mm.
- **(b)** The minimum cross-sectional area of vertical reinforcement shall be one-third of the required shear reinforcement.
- **(c)** Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook.

6.1.2 Seismic Design Category A

6.1.2.1 Structure in Seismic design category A shall comply with the requirements of Chapter 7, Chapter 8 or Chapter 9.

6.1.2.2 **Drifts limits –** The calculated story drift of masonry structures due to the combination of design seismic forces and gravity loads shall not exceed 0.007 times the story height.

6.1.3 Seismic Design Category B

6.1.3.1 Structures in Seismic Design category B shall comply with the requirements of Seismic design Category A and to the additional requirements in Section 6.1.3.2 and 6.1.3.3.

6.1.3.2 **Design of elements that are part of basic seismic-force-resisting system.** The lateral force-resisting system shall be designed to comply with the requirements of Chapter 7, Chapter 8 and Chapter 9. Masonry shear walls shall comply with the requirements of ordinary plain (unreinforced) masonry shear walls, detailed plain (unreinforced) masonry shear walls, ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, or special reinforced masonry shear walls.

6.1.3.3 **Masonry walls not part of the basic seismic-force-resisting system**. Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass. Shall be isolated from the structure so that the vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.

6.1.4 Seismic Design Category C

6.1.4.1 Structures in Seismic Design Category C shall comply with the requirements of Seismic Design Category B and to the additional requirements in Section 6.1.4.2 and 6.1.4.3.

6.1.4.2 Design of elements that are not part of basic seismic-force-resisting system

6.1.4.2.1 Load-bearing frames or columns that are not part of the lateral force-resisting system shall be analyzed as to their effect on the response of the system. Such frames or columns shall be adequate for vertical load carrying capacity and induced moment due to the design story drift.

6.1.4.2.2 Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.

6.1.4.2.3 **Reinforcement requirements.** Masonry elements listed in Section 6.1.4.2.2 shall be reinforced in either the horizontal or vertical direction in accordance with the following:

(a) Horizontal reinforcement. Horizontal joint reinforcement shall consist of at least two longitudinal WD 4 wires spaced not more than 400 mm for walls greater than 100 mm in width and at least one longitudinal WD 4 wire spaced not more 400 mm for walls not exceeding 100 mm in width; or at least one (Dia 12) bar spaced not more than 1200 mm. Where two longitudinal wires of joint reinforcement are used, the space between these wires shall be the widest that the mortar joint will accommodate. Horizontal reinforcement shall be provided within 400 mm of the top and bottom of these masonry walls.

(b) Vertical reinforcement. Vertical reinforcement shall consist of at least one (Dia 12) bar spaced not more than 1200 mm. Vertical reinforcement shall be located within 400 mm of the ends of masonry walls.

6.1.4.3 Design of elements that are part of the Basic seismic-force-resisting system

6.1.4.3.1 **Connections to masonry columns.** Connectors shall be provided to transfer forces between masonry columns and horizontal elements in accordance with the requirements of Section 7.1.8. Where anchor bolts are used to connect horizontal elements to the tops of columns, anchor bolts shall be placed within lateral ties. Lateral ties shall enclose both the vertical bars in the column and the anchor bolts. There shall be a minimum of two (Dia 12) lateral ties provided in the top 130 mm of the column.

6.1.4.3.2 **Masonry shear walls.** Masonry shear walls shall comply with the requirements for ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, or special reinforced masonry shear walls.

6.1.4.4 **Design of discontinuous members that are part of the lateral-force-resisting system.** Columns and pilasters that are part of the lateral-force-resisting system and that support reactions from discontinuous stiff members such as walls shall be provided with transverse reinforcement spaced at no more than one-fourth of the least nominal dimension of the column or pilaster. The minimum transverse reinforcement ratio shall be 0.0015. Beams supporting reactions from discontinuous walls or frames shall be provided with transverse reinforcement spaced at no more than one-half of the nominal depth of the beam. The minimum transverse reinforcement ratio shall be 0.0015.

6.1.5 Seismic design category D

6.1.5.1 Structures in Seismic design Category D shall comply with the requirements of Seismic Design Category C and to the additional requirements in Section 6.1.5.2 to 6.1.5.9.

6.1.5.2 **Design requirements** – Masonry elements, other than those covered by Section 6.1.4.2.2, shall be designed in accordance with the requirements of Sections 7.1 and 7.3 or Chapter 8.

6.1.5.3 **Minimum reinforcement requirements for masonry walls** – Masonry walls other than those covered by section 6.1.4.2.3 shall be reinforced in both the vertical and horizontal direction. The sum of the cross-sectional area of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall, and the minimum cross-sectional area in each direction shall be not less than 0.0007 times the gross cross-sectional area of the wall, using specified dimensions. Reinforcement shall be uniformly distributed. The maximum spacing of reinforcement shall be 1200 mm except for stack bond masonry. Wythes of stack bond masonry shall be constructed of fully grouted hollow open-end units, fully grouted hollow units laid with full head joints or solid units. Maximum spacing of reinforcement for walls with stack bond masonry shall be 600 mm.

6.1.5.4 **Masonry shear walls** – Masonry shear walls shall comply with the requirements for special reinforced masonry shear walls.

6.1.5.5 **Minimum reinforcement for masonry columns** – Lateral ties in masonry columns shall be spaced not more than 200 mm on center and shall be at least 10 mm diameter. Lateral ties shall be embedded in grout.

6.1.5.6 **Material requirements** – Neither Type N mortar nor masonry cement shall be used as part of the lateral force-resisting system.

6.1.5.7 **Lateral tie anchorage** – Standard hooks for lateral tie anchorage shall be either a 135 degree standard hook or a 180 degree standard hook.

6.1.5.8 **Loads for shear walls designed by the working stress design method** – When calculating shear or diagonal tension stresses by the working stress design method. Shear walls that resist seismic forces shall be designed to resist 1.5 times the forces required by SBC 301. The multiplier need not be applied to the overturning moment.

6.1.5.9 **Shear walls shear strength** – For all shear walls whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.

For all cross sections within a region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall, the nominal shear strength shall be determined by Equation (6-1).

$$V_n = A_n \rho_n f_y \qquad \textbf{Eq. (6-1)}$$

The required shear strength for this region shall be calculated at a distance $L_w/2$ above the base of the shear wall, but not to exceed one-half story height.

For the other region, the nominal shear strength of the shear wall shall be determined from Chapter 8.

SECTION 6.2
ANCHORAGE OF MASONRY WALLS

6.2.1 Masonry walls shall be anchored to the roof and floors that provide lateral support for the wall in accordance with Section 10.11 SBC 301.

CHAPTER 7
WORKING STRESS DESIGN

SECTION 7.1
GENERAL

7.1.1 **Scope.** This chapter provides minimum requirements for allowable stress design of masonry. Masonry designed by working stress design method shall comply with the requirements of Section 7.1 and either Section 7.2 or Section 7.3.

7.1.2 **Load combinations:** Loads and Load Combinations are in accordance with SBC 301. The allowable stresses and allowable loads are permitted to be increased by one third when considering load combinations of Section 2.4, SBC 301.

7.1.3 **Design strength**

7.1.3.1 Project drawings shall show the specified compressive strength of masonry, f_m', for each part of the structure.

7.1.3.2 Each portion of the structure shall be designed based on the specified compressive strength of masonry, f_m', for that part of the work.

7.1.4 **Anchor bolts solidly grouted in masonry**

7.1.4.1 **Test design requirements** – Except as provided in Section 7.1.4.2, anchor bolts shall be designed based on the following provisions.

7.1.4.1.1 Anchors shall be tested in accordance with ASTM E 488 under stresses and conditions representing intended use, except that a minimum of five tests shall be performed.

7.1.4.1.2 Allowable loads shall not exceed 20% of the average tested strength.

7.1.4.2 **Plate, headed, and bent bar anchor bolts** – The allowable loads for plate anchors, headed anchor bolts, and bent bar anchor bolts (J or L type) embedded in masonry shall be determined in accordance with the provisions of Section 7.1.4.2.1 through Section 7.1.4.2.4.

7.1.4.2.1 The minimum effective embedment length shall be 4 bolt diameters, but not less than 50 mm.

7.1.4.2.2 The allowable load in tension shall be the lesser of that given by Eq. (7-1) or Eq. (7-2).

$$B_a = 0.042\, A_p \sqrt{f'_m} \qquad \textbf{Eq. (7-1)}$$

$$B_a = 0.2\, A_b f_y \qquad \textbf{Eq. (7-2)}$$

7.1.4.2.2.1 The area A_p shall be the lesser of Eq. (7-3) or Eq. (7-4). Where the projected areas of adjacent anchor bolts overlap, A_p of each bolt shall be reduced by on-half of the overlapping area. That portion of the projected area falling in an open cell or core shall be deducted from the value of A_p calculated using Eq. (7-3) or (7-4).

$$A_p = \pi l_b^2 \qquad \textbf{Eq. (7-3)}$$

$$A_p = \pi l_{be}^2 \qquad \textbf{Eq. (7-4)}$$

7.1.4.2.2.2 The effective embedment length of plate or headed bolts, l_b, shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the plate or head of the anchor bolt.

7.1.4.2.2.3 The effective embedment length of bent anchors, l_b, shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the bent end minus one anchor bolt diameter.

7.1.4.2.3 The allowable load in shear, where l_{be} equals or exceeds 12 bolt diameters, shall be the lesser of that given by Eq. (7-5) or Eq. (7-6).

$$B_v = 1072\sqrt[4]{f'_m\ A_b} \qquad \textbf{Eq. (7-5)}$$

$$B_v = 0.12\,A_b f_y \qquad \textbf{Eq. (7-6)}$$

Where l_{be} is less than 12 bolt diameters, the value of B_v in Eq. (7-5) shall be reduced by linear interpolation to zero at an l_{be} distance of 25 mm.

7.1.4.2.4 **Combined shear and tension** – Anchors in Section 7.1.4.2 subjected to combined shear and tension shall be designed to satisfy Eq. (7-7).

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \leq 1 \qquad \textbf{Eq. (7-7)}$$

7.1.5 Multiwythe walls

7.1.5.1 Design of walls composed of more than one wythe shall comply with the provisions of Section 7.1.5.

7.1.5.2 Composite action

7.1.5.2.1 Multiwythe walls designed for composite action shall have collar joints either:

(a) crossed by connecting headers, or

(b) filled with mortar or grout and connected by wall ties.

7.1.5.2.2 Shear stresses developed in the planes of interfaces between wythes and collar joints or within headers shall not exceed the following:

(a) mortared collar joints, 35 kPa.

(b) grouted collar joints, 70 kPa.

(c) headers, square root of unit compressive strength of header, MPa (over net area of header).

7.1.5.2.3 Headers of wythes bonded by headers shall meet the requirements of Section 7.1.5.2.2 and shall be provided as follows:

(a) Headers shall be uniformly distributed and the sum of their cross-sectional areas shall be at least 4% of the wall surface area.

(b) Headers connecting adjacent wythes shall be embedded a minimum of 80 mm in each wythe.

7.1.5.2.4 Wythes not bonded by headers shall meet the requirements of Section 7.1.5.2.2 and shall be bonded by wall ties provided as follows:

Wire size	Minimum number of wall ties required
WD 4	one per 0.25 m^2 of wall
WD 5	one per 0.40 m^2 of wall

The maximum spacing between ties shall be 900 mm horizontally and 600 mm vertically.

The use of rectangular wall ties to tie walls made with any type of masonry units is permitted. The use of Z wall ties to tie walls made with other than hollow masonry units is permitted. Cross wires of joint reinforcement are permitted to be used in lieu of wall ties.

7.1.5.3 **Noncomposite action**. Masonry designed for noncomposite action shall comply to the following provisions:

7.1.5.3.1 Each wythe shall be designed to resist individually the effects of loads imposed on it.

Unless a more detailed analysis is performed, the following requirements shall be satisfied:

(a) Collar joints shall not contain headers, grout, or mortar.

(b) Gravity loads from supported horizontal members shall be resisted by the wythe nearest to the center of span of the supported member. Any resulting bending moment about the weak axis of the wall shall be distributed to each wythe in proportion to its relative stiffness.

(c) Loads acting parallel to the plane of a wall shall be carried only by the wythe on which they are applied. Transfer of stresses from such loads between wythes shall be neglected.

(d) Loads acting transverse to the plane of a wall shall be resisted by all wythes in proportion to their relative flexural stiffnesses.

(e) Specified distances between wythes shall not exceed a width of 100 mm unless a detailed wall tie analysis is performed.

7.1.5.3.2 Wythes of walls designed for noncomposite action shall be connected by wall ties meeting the requirements of Section 7.1.5.2.4 or by adjustable ties. Where the cross wires of joint reinforcement are used as ties, the joint reinforcement shall be ladder-type. Wall ties shall be without cavity drips.

Adjustable ties shall meet the following requirements:

(a) One tie shall be provided for each 0.16 m^2 of wall area.

(b) Horizontal and vertical spacing shall not exceed 400 mm.

(c) Adjustable ties shall not be used when the misalignment of bed joints from one wythe to the other exceeds 32 mm.

(d) Maximum clearance between connecting parts for the tie shall be 1.5 mm.

(e) Pintle ties shall have at least two pintle legs of wire size (WD 5).

7.1.6 **Columns.** Design of columns shall meet the general requirements of this section.

7.1.6.1 Minimum side dimension shall be 200 mm nominal.

7.1.6.2 The ratio between the effective height and least nominal dimension shall not exceed 25.

7.1.6.3 Columns shall be designed to resist applied loads. As a minimum, columns shall be designed to resist loads with an eccentricity equal to 0.1 times each side dimension. Consider each axis independently.

7.1.6.4 Vertical column reinforcement shall not be less than 0.0025 A_n nor exceed 0.04 A_n. The minimum number of bars shall be four.

7.1.6.5 **Lateral ties.** Lateral ties shall conform to the following:

a) Longitudinal reinforcement shall be enclosed by lateral ties at least 10 mm in diameter.

b) Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 lateral tie bar or wire diameters, or least cross-sectional dimension of the member.

c) Lateral ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a lateral tie with an included angle of not more than 135 degrees. No bar shall be farther than 150 mm clear on each side along the lateral tie from such a laterally supported bar. Lateral ties shall be placed in either a mortar joint or in grout. Where longitudinal bars are located around the perimeter of a circle, a complete circular lateral is permitted. Lap length for circular ties shall be 48 tie diameters.

d) Lateral ties shall be located vertically not more than one-half lateral tie spacing above the top of footing or slab in any story, and shall be spaced as provide herein to not more than one-half a lateral tie spacing below the lowest horizontal reinforcement in beam, girder, slab, or drop panel above.

e) Where beams or brackets frame into a column from four directions, lateral ties may be terminated not more than 76 mm below the lowest reinforcement in the shallowest of such beams or brackets.

7.1.6.6 Masonry columns used only to support light-frame roofs of car-ports, porches, sheds or similar structures with a maximum area of 42 m^2 assigned to Seismic Design Category A, B or C are permitted to be designed and constructed as follows:

1. Concrete masonry materials shall be in accordance with Section 3.1. Clay or shale masonry units shall be in accordance with Section 3.2.

2. The nominal cross-sectional dimension of columns shall not be less than 200 mm.

3. Columns shall be reinforced with not less than one Dia 12 mm bar centered in each cell of the column.

4. Columns shall be grouted solid.

5. Columns shall not exceed 3.65 m in height.

6. Roofs shall be anchored to the columns. Such anchorage shall be capable of resisting the design load specified in SBC 301.

7. Where such columns are required to resist uplift loads, the columns shall be anchored to their footings with two Dia 12 mm bars extending a minimum of 600 mm into the columns and bent horizontally a minimum of 400 mm in opposite directions into the footings. One of these bars is permitted to be the reinforcing bar specified in Item 3 above. The total weight of a column and its footing shall not be less than 1.5 times the design uplift load.

7.1.7 Pilasters

7.1.7.1 Walls interfacing with pilasters shall not be considered as flanges unless the provisions of Section 3.13.4.2 are met.

7.1.7.2 Where vertical reinforcement is provided to resist axial compressive stress, lateral ties shall meet all applicable requirements of Section 7.1.6.5.

7.1.8 Load transfer at horizontal connections

7.1.8.1 Walls, columns, and pilasters shall be designed to resist all loads, moments, and shears applied at intersections with horizontal members.

7.1.8.2 Effect of lateral deflection and translation of members providing lateral support shall be considered.

7.1.8.3 Devices used for transferring lateral support from members that intersect walls, columns, or pilasters shall be designed to resist the forces involved. For columns, a force of not less than 4500 N shall be used.

7.1.9 Concentrated loads

7.1.9.1 For computing compressive stress f_a for walls laid in running bond, concentrated loads shall not be distributed over the length of supporting wall in excess of the length of wall equal to the width of bearing areas plus four times the thickness of the supporting wall, but not to exceed the center-to-center distance between concentrated loads.

7.1.9.2 Bearing stresses shall be computed by distributing the bearing load over an area determined as follows:

(a) The direct bearing area A_1, or

(b) $A_1\sqrt{A_2 / A_1}$ but not more than $2A_1$, where A_2 is the supporting surface wider than A_1 on all sides, or A_2 is the area of the lower base of the largest frustum of a right pyramid or cone having A_1 as upper base sloping at 45 degrees from the horizontal and wholly contained within the support. For walls in other than running bond, area A_2, shall terminate at head joints.

7.1.9.3 Bearing stresses shall not exceed 0.25 $f_m^{'}$.

7.1.10 Development of reinforcement embedded in grout

7.1.10.1 **General.** The calculated tension or compression in the reinforcement at each section shall be developed on each side of the section by embedment length, hook or mechanical device, or a combination thereof. Hooks shall not be used to develop bars in compression.

7.1.10.2 **Embedment of bars and wires in tension.** The embedment length of bars and wire shall be determined by Eq. (7-8), but shall not be less than 300 mm for bars and 150 mm for wire.

$$l_d = 0.22\, d_b\, F_s \qquad \textbf{Eq. (7-8)}$$

When epoxy-coated bars or wires are used, development length determined by Eq. (7-8) shall be increased by 50 percent.

7.1.10.3 Embedment of flexural reinforcement

7.1.10.3.1 General

7.1.10.3.1.1 Tension reinforcement is permitted to be developed by bending across the neutral axis of the member to be anchored or made continuous with reinforcement on the opposite face of the member.

7.1.10.3.1.2 Critical sections for development of reinforcement in flexural members are at points of maximum steel stress and at points within the span where adjacent reinforcement terminates or is bent.

7.1.10.3.1.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member or $12d_b$, whichever is greater, except at supports of simple spans and at the free end of cantilevers.

7.1.10.3.1.4 Continuing reinforcement shall extend a distance l_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure as required by Section 7.1.10.2.

7.1.10.3.1.5 Flexural reinforcement shall not be terminated in a tension zone unless one of the following conditions is satisfied:

(a) Shear at the cutoff point does not exceed two-thirds of the allowable shear at the section considered.

(b) Stirrup area in excess of that required for shear is provided along each terminated bar or wire over a distance from the termination point equal to three-fourths the effective depth of the member. Excess stirrup area, Av, shall not be less than 60 $b_w s/f_y$. Spacing s shall not exceed d/(8 β b).

(c) Continuous reinforcement provides double the area required for flexure at the cut-off point and shear does not exceed three-fourths the allowable shear at the section considered.

7.1.10.3.1.6 Anchorage complying with Section 7.1.10.2 shall be provided for tension reinforcement in corbels, deep flexural members, variable-depth arches, members where flexural reinforcement is not parallel with the compression face, and in other cases where the stress in flexural reinforcement does not vary linearly in proportion to the moment.

7.1.10.3.2 **Development of positive moment reinforcement** When a wall or other flexural member is part of a primary lateral resisting system, at least 25 percent of the positive moment reinforcement shall extend into the support and be anchored to develop a stress equal to the F_s in tension.

7.1.10.3.3 Development of negative moment reinforcement

7.1.10.3.3.1 Negative moment reinforcement in a continuous, restrained, or cantilever member shall be anchored in or through the supporting member in accordance with the provisions of Section 7.1.10.1.

7.1.10.3.3.2 At least one-third of the total reinforcement provided for moment at a support shall extend beyond the point of inflection the greater distance of the effective depth of the member or one-sixteenth of the span.

7.1.10.4 **Hooks**

7.1.10.4.1 Standard hooks in tension shall be considered to develop an equivalent embedment length, l_e, equal to 11.25 d_b.

7.1.10.4.2 The effect of hooks for bars in compression shall be neglected in design computations.

7.1.10.5 **Development of Shear reinforcement**

7.1.10.5.1 Bar and wire reinforcement

7.1.10.5.1.1 Shear reinforcement shall extend to a distance d from the extreme compression face and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its calculated stress.

7.1.10.5.1.2 The ends of single leg or U-stirrups shall be anchored by one of the following means:

(a) A standard hook plus an effective embedment of 0.5 l_d. The effective embedment of a stirrup leg shall be taken as the distance between mid depth of the member d/2 and the start of the hook (point of tangency).

(b) For bar (Dia 16) and (WD 16.0) wire and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of 0.33 l_d. The 0.33 l_d embedment of a stirrup leg shall be taken as the distance between mid depth of member d/2 and start of hook (point of tangency).

7.1.10.5.1.3 Between the anchored ends, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

7.1.10.5.1.4 Longitudinal bars bent to act as shear reinforcement, where extended into a region of tension, shall be continuous with longitudinal reinforcement and, where extended into a region of compression, shall be developed beyond mid depth of the member d/2.

7.1.10.5.1.5 Pairs of U-stirrups or ties placed to form a closed unit shall be considered properly spliced when length of laps are 1.7 l_d. In grout at least 450 mm deep, such splices with A_v f_y not more than 40000 N per leg may be considered adequate if legs extend the full available depth of grout.

7.1.10.5.2 **Welded wire fabric**

7.1.10.5.2.1 For each leg of welded wire fabric forming simple U-stirrups, there shall be either:

(a) Two longitudinal wires at a 50 mm spacing along the member at the top of the U, or

(b) One longitudinal wire located not more than d/4 from the compression face and a second wire closer to the compression face and spaced not less than 50 mm from the first wire. The second wire shall be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than 8 d_b.

7.1.10.5.2.2 For each end of a single leg stirrup of welded smooth or deformed wire fabric, there shall be two longitudinal wires spaced minimum of 50 mm with the inner wire placed at a distance at least d/4 or 50 mm from middepth of member d/2. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

7.1.10.6 **Splices of reinforcement** – Lap splices, welded splices, or mechanical connections are permitted in accordance with the provisions of this section. All welding shall conform to AWS D 1.4.

7.1.10.6.1 **Lap Splices**

7.1.10.6.1.1 The minimum length of lap splices for reinforcing bars in tension or compression l_{ld}, shall be calculated by Eq (7-10), but shall not be less than 400 mm.

$$l_{ld} = \frac{1.95 d_b^2 f_y \gamma}{K\sqrt{f'_m}} \qquad \textbf{Eq. (7-10)}$$

Where:

d_b = Diameter of reinforcement, mm.

f_y = Specified yield stress of the reinforcement or the anchor bolt. MPa.

f'_m = Specified compressive strength of masonry at age of 28 days. MPa.

l_{ld} = Minimum lap splice length, mm.

K = The lesser of the masonry cover, clear spacing between adjacent reinforcement or five times d_b, mm.

γ = 1.0 for Dia 10 through Dia 16 mm reinforcing bars. 1.4 for Dia 20 and Dia 22 mm reinforcing bars. 1.5 for Dia 25 and Dia 28 mm reinforcing bars.

7.1.10.6.1.2 Bars spliced by noncontact lap splices shall not be spaced transversely farther apart than one-fifth the required length of lap nor more than 200 mm.

7.1.10.6.1.3 Splices for large bars. Reinforcing bars larger than Dia 28 mm in size shall be spliced using mechanical connectors in accordance with Section 7.1.10.6.3.

7.1.10.6.2 **Welded splices** - Welded splices shall have the bars butted and welded to develop in tension at least 125% of the specified yield strength of the bar.

7.1.10.6.3 **Mechanical connections** - Mechanical connections shall have the bars connected to develop in tension or compression, as required, at least 125% of the specified yield strength of the bar.

7.1.10.6.4 **End-bearing splices**

7.1.10.6.4.1 In bars required for compression only, the transmission of compressive stress by bearing of square cut ends held in concentric contact by a suitable device is permitted.

7.1.10.6.4.2 Bar ends shall terminate in flat surfaces within 1½ degree of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly.

7.1.10.6.4.3 End-bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

7.1.11 **Maximum bar size** The bar diameter shall not exceed one-eighth of the nominal wall thickness and shall not exceed one-quarter of the least dimension of the cell, course or collar joint in which it is placed

7.1.12 **Maximum reinforcement percentage** Special reinforced masonry shear walls having a shear span ratio M/Vd. Equal to or greater than 1.0 and having an axial load, P greater than 0.05 $f'_m A_n$ which are subjected to in-plane forces, shall have a maximum reinforcement ratio, ρ_{max}, not greater than that computed as follows:

$$\rho_{max} = \frac{nf'_m}{2f_y\left(n + \frac{f_y}{f'_m}\right)} \qquad \textbf{Eq. (7-9)}$$

The maximum reinforcement ratio does not apply in the out-of-plane direction.

7.1.13 Special inspection during construction shall be provided as set forth in SBC 302.

SECTION 7.2
UNREINFORCED MASONRY

7.2.1 **Scope.** This section provides requirements for unreinforced masonry as defined in Chapter 2, except as otherwise indicated in Section 7.2.4.

7.2.2 **Stress in reinforcement.** The effect of stresses in reinforcement shall be neglected.

7.2.3 **Axial compression and flexure**

7.2.3.1 Members subjected to axial compression, flexure, or to combined axial compression and flexure shall be designed to satisfy Eq. (7-11) and Eq. (7-12).

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \qquad \textbf{Eq. (7-11)}$$

$$P \leq (^1/_4)\, P_e \qquad \textbf{Eq. (7-12)}$$

Where:

a) For members having an h/r ratio not greater than 99:

$$F_a = (^1/_4)\ f'_m \left[1 - \left(\frac{h}{140r}\right)^2\right] \qquad \textbf{Eq. (7-13)}$$

b) For members having an h/r ratio greater than 99:

$$F_a = (^1/_4)\ f'_m \left(\frac{70r}{h}\right)^2 \qquad \textbf{Eq. (7-14)}$$

c) $$F_b = (^1/_3)\ f'_m \qquad \textbf{Eq. (7-15)}$$

d) $$P_e = \frac{\pi^2 E_m l_n}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3 \qquad \textbf{Eq. (7-16)}$$

7.2.3.2 Allowable tensile stresses due to flexure transverse to the plane of masonry member shall be in accordance with the values in Table 7.1.1.

7.2.4 **Axial tension.** The tensile strength of masonry shall be neglected in design when the masonry is subjected to axial tension forces.

7.2.5 **Shear**

7.2.5.1 Shear stresses due to forces acting in the direction considered shall be computed in accordance with Section 3.13.1 and determined by Eq. (7-17).

$$f_v = VQ/I_n b \qquad \textbf{Eq.(7-17)}$$

7.2.5.2 In-plane shear stresses shall not exceed any of:

(a) $0.125\sqrt{f'_m}$

(b) 827 kPa

(c) $\nu + 0.45\, N_v / A_n$

where ν :

= 255 kPa for masonry in running bond that is not grouted solid, or

= 255 kPa for masonry in other than running bond with open end units that are grouted solid, or

= 414 kPa for masonry in running bond that is grouted solid.

(d) 100 kPa for masonry in other than running bond with other than open end units that are grouted solid.

Table 7.1.1 Allowable flexural tension for clay and concrete masonry, kPa

Direction of flexural tensile Stress and masonry type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained Portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints Solid units	270	200	160	100
Hollow units,# Ungrouted	170	130	100	60
Fully grouted	445	430	420	400
Parallel to bed joints in running bond Solid units	550	400	330	200
Hollow Units – Ungrouted and partially grouted	345	260	200	130
Fully grouted	550	410	330	200

For partially grouted masonry, allowable stresses shall be determined on the basis of linear interpolation between hollow units that are fully grouted and ungrouted hollow units based on amount of grouting.

SECTION 7.3
REINFORCED MASONRY

7.3.1 **Scope.** This section provides requirements for the design of structures neglecting the contribution of tensile strength of masonry, except as provided in Section 7.3.5.

7.3.2 **Steel reinforcement.** Allowable stresses

7.3.2.1 **Tension.** Tensile stress in reinforcement shall not exceed the following:

(a) Grade 40 or Grade 50 reinforcement…… 138 MPa

(b) Grade 60 reinforcement ………..……….. 165 MPa

(c) Wire joint reinforcement………………..... 200 MPa

7.3.2.2 Compression

7.3.2.2.1 The compressive resistance of steel reinforcement shall be neglected unless lateral reinforcement is provided in compliance with the requirement of Section 7.1.6.5.

7.3.2.2.2 Compressive stress in reinforcement shall not exceed the lesser of $0.4f_y$ or 165 MPa.

7.3.3 Axial compression and flexure

7.3.3.1 Members subjected to axial compression, flexure, or combined axial compression and flexure shall be designed in compliance with Section 7.3.3.2 through Section 7.3.3.4.

7.3.3.2 Allowable forces and stresses

7.3.3.2.1 The compressive force in reinforced masonry due to axial load only shall not exceed that given by Eq. (7-18) or Eq. (7-19):

(a) For members having an h/r ratio not greater than 99:

$$P_a = (0.25\ f'_m\ A_n + 0.65\ A_{st}\ F_s)\left[1-\left(\frac{h}{140r}\right)^2\right] \qquad \textbf{Eq. (7-18)}$$

(b) For members having an h/r ratio greater than 99:

$$P_a = (0.25\ f'_m A_n + 0.65\ A_{st}\ F_s)\left(\frac{70r}{h}\right)^2 \qquad \textbf{Eq. (7-19)}$$

7.3.3.2.2 The compressive stress in masonry due to flexure or due to flexure in combination with axial load shall not exceed (1/3) f'_m provided the calculated compressive stress due to the axial load component, f_a, does not exceed the allowable stress, F_a, in Section 7.2.3.1.

7.3.3.3 Effective compressive width per bar

7.3.3.3.1 In running bond masonry and masonry in other than running bond with bond beams spaced not more than 1200 mm center-to-center, the width of the compression area used in stress calculations shall not exceed the least of:

(a) Center-to-center bar spacing.

(b) Six times the wall thickness.

(c) 1800 mm.

7.3.3.3.2 In masonry in other than running bond, with bond beams spaced more than. 1200 mm center-to-center, the width of the compression area used in stress calculations shall not exceed the length of the masonry unit.

7.3.3.4 Beams

7.3.3.4.1 Span length of members not built integrally with supports shall be taken as the clear span plus depth of member, but need to exceed the distance between centers of supports.

7.3.3.4.2 In analysis of members that are continuous over supports for determination of moments, span length shall be taken as the distance between centers of supports.

7.3.3.4.3 Length of bearing of beams on their supports shall be a minimum of 100 mm in the direction of span.

7.3.3.4.4 The compression face of beams shall be laterally supported at a maximum spacing of 32 times the beam thickness.

7.3.3.4.5 Beams shall be designed to meet the deflection requirements of Section 4.1.5.1.

7.3.4 **Axial tension and flexural tension.** Axial tension and flexural tension shall be resisted entirely by steel reinforcement.

7.3.5 **Shear**

7.3.5.1 Members that are not subjected to flexural tension shall be designed in accordance with the requirements of Section 7.2.5 or shall be designed in accordance with the following:

7.3.5.1.1 Reinforcement shall be provided in accordance with the requirements of Section 7.3.5.3.

7.3.5.1.2 The calculated shear stress, f_v, shall not exceed F_v, where F_v is determined in accordance with Section 7.3.5.2.3.

7.3.5.2 Members subjected to flexural tension shall be reinforced to resist the tension and shall be designed in accordance with the following:

7.3.5.2.1 Calculated shear stress in the masonry shall be determined by the relationship:

$$f_v = V/bd \qquad \textbf{Eq. (7-20)}$$

7.3.5.2.2 Where reinforcement is not provided to resist all of the calculated shear, f_v shall not exceed F_v, where:

(a) For flexural members:

$$F_v = 0.083\sqrt{f'_m} \qquad \textbf{Eq. (7-21)}$$

but shall not exceed 345 kPa.

(b) For shear walls,
where M/Vd < 1

$$F_v = 0.028\,[4-(M/V\text{d})]\,\sqrt{f'_m} \qquad \textbf{Eq. (7-22)}$$

but shall not exceed (0.55-0.31 (M/Vd))

where M/Vd $\geq$ 1,

$$F_v = 0.083\sqrt{f'_m} \qquad \textbf{Eq. (7-23)}$$

but shall not exceed 240kPa

7.3.5.2.3 Where shear reinforcement is provided in accordance with Section 7.3.5.3 to resist all of the calculated shear, f_v shall not exceed F_v, where:

(a) For flexural members:

$$F_v = 0.25\sqrt{f'_m} \qquad \textbf{Eq. (7-24)}$$

but shall not exceed 1000 kPa

(b) For shear walls,

where $M/V\text{d} < 1$

$$F_v = 0.042\ [4 - (M/V\text{d})]\ \sqrt{f'_m} \qquad \textbf{Eq. (7-25)}$$

but shall not exceed (0.82-0.31 (M/Vd))

where $M/V\text{d} \geq 1$

$$F_v = 0.125\ \sqrt{f'_m} \qquad \textbf{Eq. (7-26)}$$

but shall not exceed 500 kPa

7.3.5.2.4 The ratio M/Vd shall always be taken as a positive number.

7.3.5.3 Minimum area of shear reinforcement required by Section 7.3.5.1 or 7.3.5.2.3 shall be determined by the following:

$$A_v = \frac{V\ s}{F_s\ d} \qquad \textbf{Eq. (7-27)}$$

7.3.5.3.1 Shear reinforcement shall be provided parallel to the direction of applied shear force. Spacing of shear reinforcement shall not exceed the lesser of $d/2$ or 1200 mm.

7.3.5.3.2 Reinforcement shall be provided perpendicular to the shear reinforcement and shall be at least equal to one-third A_v. The reinforcement shall be uniformly distributed and shall not exceed a spacing of 2.50 m.

7.3.5.4 In composite masonry walls, shear stresses developed in the planes of interfaces between wythes and filled collar joints or between wythes and headers shall meet the requirements of Section 7.1.5.2.2.

7.3.5.5 In cantilever beams, the maximum shear shall be used. In noncantilever beams, the maximum shear shall be used except that sections located within a distance $d/2$ from the face of support shall be designed for the same shear as that computed at a distance $d/2$ from the face of support when the following conditions are met:

(a) Support reaction, in direction of applied shear force, introduces compression into the end regions of member, and

(b) No concentrated load occurs between face of support and a distance $d/2$ from face.

CHAPTER 8
STRENGTH DESIGN OF MASONRY

SECTION 8.1
GENERAL

8.1.1 **Scope.** This Chapter provides minimum requirements for strength design of masonry. Masonry design by the strength design method shall comply with the requirements of Section 8.1 and either Section 8.2 or Section 8.3.

The minimum nominal thickness for hollow clay masonry in accordance with Section 8.2.2 shall be 100 mm.

8.1.2 **Required strength.** Required strength shall be determined in accordance with the strength design load combinations of SBC 301. Members subject to compressive axial load shall be designed for the maximum design moment accompanying the axial load. The factored moment, M_u, shall include the moment induced by relative lateral displacement.

8.1.3 **Design strength.** Masonry members shall be proportioned such that the design strength equals or exceeds the required strength. Design strength is the nominal strength multiplied by the strength reduction factor, ϕ, as specified in Section 8.1.4.

The design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the member, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u.

8.1.3.1 **Seismic design provisions.** At each story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls. Along each column line at a particular story level, at least 80% of the lateral stiffness shall be provided by lateral-force-resisting walls.

Exception: Where seismic loads are determined based on a seismic response modification factor, R, not greater than 1.5, piers and columns are permitted to be used to provide seismic load resistance.

8.1.4 **Strength reduction factors**

8.1.4.1 **Combinations of flexure and axial load in reinforced masonry.** The value of ϕ shall be taken as 0.90 for reinforced masonry subjected to flexure, axial load, or combinations thereof.

8.1.4.2 **Combinations of flexure and axial load in unreinforced masonry.** The value of ϕ shall be taken as 0.60 for unreinforced masonry subjected to flexure, axial load, or combinations thereof.

8.1.4.3 **Shear.** The value of ϕ shall be taken as 0.80 for masonry subjected to shear.

8.1.4.4 **Anchor bolts.** For cases where the nominal strength of an anchor bolt is controlled by masonry breakout, ϕ shall be taken as 0.50. For cases where the nominal strength of an anchor bolt is controlled by anchor bolt steel, ϕ shall be taken as 0.90. For cases where the nominal strength of an anchor bolt is controlled by anchor pullout, ϕ shall be taken as 0.65.

8.1.4.5 **Development and splices of reinforcement.** For development and splicing of reinforcement, ϕ shall be taken as 0.80.

8.1.4.6 **Bearing.** For cases where bearing on masonry, ϕ shall be taken as 0.60.

8.1.5 Deformation requirements

8.1.5.1 **Drift limits.** Under loading combinations that include earthquake, masonry structures shall be designed so the calculated story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from SBC 301.

For determining drift, the calculated deflection shall be multiplied by C_d as indicated in SBC 301.

8.1.5.2 **Deflection of unreinforced (plain) masonry.** Deflection calculations for unreinforced (plain) masonry members shall be based on uncracked section properties.

8.1.5.3 **Deflection of reinforced masonry.** Deflection calculations for reinforced masonry members shall be based on cracked section properties. The flexural and shear stiffness properties assumed for deflection calculations shall not exceed one half of the gross section properties unless a cracked-section analysis is performed.

8.1.6 **Headed and bent-bar anchor bolts.** All embedded bolts shall be grouted in place with at least 13 mm of grout between the bolt and the masonry, except that 6.5 mm diameter bolts are permitted to be placed in bed joints that are at least 13 mm in thickness.

8.1.6.1 **Nominal axial tensile strength of headed anchor bolts.** The nominal axial tensile strength, $B_{an,}$ of headed anchor bolts embedded in masonry shall be computed by Eq. (8-1) (strength governed by masonry breakout) and Eq. (8-2) (strength governed by steel). In computing the capacity, the smaller of the design strengths shall be used.

$$B_{an} = 0.332\, A_{\mathrm{pt}} \sqrt{f_m'} \qquad \textbf{Eq. (8-1)}$$

$$B_{an} = A_b f_y \qquad \textbf{Eq. (8-2)}$$

8.1.6.1.1 **Projected area of masonry for headed anchor bolts.** The projected area, A_{pt}, in Eq. (8-1) shall be determined by Eq. (8-3).

$$A_{pt} = \pi\ l_b^2 \qquad \textbf{Eq. (8-3)}$$

Where the projected areas, A_{pt}, of adjacent headed anchor bolts overlap, the projected area, A_{pt}, of each bolt shall be reduced by one-half of the overlapping area. The portion of the projected area overlapping an open cell, open head joint, or that is outside the wall shall be deducted from the value of A_{pt} calculated using Eq. (8-3).

8.1.6.1.2 **Effective embedment length for headed anchor bolts.** The effective embedment length for a headed anchor bolt, l_b, shall be the length of the embedment measured perpendicular from the masonry surface to the bearing surface of the anchor head. The minimum effective embedment length for headed anchor bolts resisting axial forces shall be 4 bolt diameters or 50 mm, whichever is greater.

8.1.6.2 **Nominal axial tensile strength of bent-bar anchor bolts.** The nominal axial tensile strength, B_{an}, for bent-bar anchor bolts (J- or L-bolts) embedded in masonry shall be computed by Eq. (8-4) (strength governed by masonry breakout), Eq. (8-5) (strength governed by steel), and Eq. (8-6) (strength governed by anchor pullout). In computing the capacity, the smaller of the design strengths shall be used.

$$B_{an} = 0.332 A_{pt} \sqrt{f'_m} \qquad \textbf{Eq. (8-4)}$$

$$B_{an} = A_b f_y \qquad \textbf{Eq. (8-5)}$$

$$B_{an} = 1.5\ f'_m e_b d_b + [3.07\pi\ (l_b + e_b + d_b) d_b] \qquad \textbf{Eq. (8-6)}$$

The second term in Eq. (8-6) shall be included only if the specified quality assurance program includes verification that shanks of J- and L-bolts are free of debris, oil, and grease when installed.

8.1.6.2.1 **Projected area of masonry for bent-bar anchor bolts.** The projected area, A_{pt} in Eq. (8-4) shall be determined by Eq. (8-7).

That portion of the projected area overlapping an open cell, open head joint, or that is outside the wall shall be deducted from the value of A_{pt} calculated using Eq. (8-7).

$$A_{pt} = \pi\ l_b^2 \qquad \textbf{Eq. (8-7)}$$

Where the projected areas, A_{pt}, of adjacent bent – bar anchor bolts overlap, the projected area, $A_{pt,}$ of each bolt shall be one-half the overlapping area.

That portion of the projected area overlapping an open cell, open head joint, or that is outside the wall shall be deducted from value of A_{pt} calculated using Eq. (8-7).

8.1.6.2.2 **Effective embedment length of bent-bar anchor bolts.** The effective embedment for a bent-bar anchor bolt, l_b, shall be the length of embedment measured perpendicular from the masonry surface to the bearing surface of the bent end, minus one anchor bolt diameter. The minimum effective embedment length for bent-bar anchor bolts resisting axial forces shall be 4 bolt diameters or 50 mm, whichever is greater.

8.1.6.3 **Nominal shear strength of headed and bent-bar anchor bolts.** The nominal shear strength, B_{vn}, shall be computed by Eq. (8-8) (strength governed by masonry breakout) and Eq. (8-9) (strength governed by steel). In computing the capacity, the smaller of the design strengths shall be used.

$$B_{vn} = 0.332\ A_{pv} \sqrt{f'_m} \qquad \textbf{Eq. (8-8)}$$

$$B_{vn} = 0.6\ A_b f_y \qquad \textbf{Eq. (8-9)}$$

8.1.6.3.1 **Projected area of masonry.** The area A_{pv}, in Eq. (8-8) shall be determined from Eq. (8-10).

$$A_{pv} = \frac{\pi\ l_{be}^{\ 2}}{2} \qquad \textbf{Eq. (8-10)}$$

8.1.6.3.2 **Minimum effective embedment length.** The minimum effective embedment length for headed or bent-bar anchor bolts resisting shear forces shall be 4 bolt diameters, or 50 mm, whichever is greater.

8.1.6.4 **Combined axial and shear strength of anchor bolts.** Anchor bolts subjected to combined shear and tension shall be designed to satisfy Eq. (8-11).

$$\frac{b_{af}}{\phi\ B_{an}} + \frac{b_{vf}}{\phi\ B_{vn}} \leq 1 \qquad \textbf{Eq. (8-11)}$$

ϕB_{an} and ϕB_{vn}, used in Eq. (8-11) shall be the governing design tensile and shear strengths, respectively.

8.1.7 Material properties

8.1.7.1 Compressive strength

8.1.7.1.1 **Masonry compressive strength.** The specified compressive strength of masonry, f_m' shall equal or exceed 10.5 MPa. The value of f_m' used to determine nominal strength values in this chapter shall not exceed 28 MPa for concrete masonry and shall not exceed 42 MPa for clay masonry.

8.1.7.1.2 **Grout compressive strength.** For concrete masonry, the specified compressive strength of grout, f_g', shall equal or exceed the specified compressive strength of masonry, f_m', but shall not exceed 35 MPa. For clay masonry, the specified compressive strength of grout, f_g', shall not exceed 42 MPa.

8.1.7.2 Masonry modulus of rupture

8.1.7.2.1 **Out-of-plane bending.** The modulus of rupture, f_r, for masonry elements subjected to out-of-plane bending shall be taken from Table 8.1.1.

8.1.7.2.2 **In-plane bending.** For masonry subjected to in-plane loads, the modulus of rupture, f_r, normal to the bed joints shall be taken as 1.75 MPa. The modulus of rupture used for masonry parallel to the bed joints shall be taken as 1.75 MPa. For grouted stack bond masonry, tension parallel to the bed joints shall be assumed to be resisted only by the continuous horizontal grout section.

8.1.7.3 **Reinforcement strength.** Masonry design shall be based on a reinforcement strength equal to the specified yield strength of reinforcement, f_y, which shall not exceed 420 MPa. The actual yield strength shall not exceed 1.3 times the specified yield strength. The compressive resistance of steel reinforcement shall be neglected unless lateral reinforcement is provided in compliance with the requirements of lateral ties.

Table 8.1.1 — Modulus of rupture *(f_r), kPa*

Direction of flexural tensile stress and masonry type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained Portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints in running or stack bond				
Solid units	700	525	420	266
Hollow units[1]				
Ungrouted	441	336	266	158
Fully grouted	1190	1015	721	511
Parallel to bed joints in running or stack bond	1400	1050	840	525
Solid units				
Hollow units				
Ungrouted and partially grouted	875	665	525	336
Fully grouted	1400	1050	840	525
Parallel to bed joints in stack bond	0	0	0	0

[1] For partially grouted masonry, modulus of rupture values shall be determined on the basis of linear interpolation between hollow units that are fully grouted and ungrouted based on amount (percentage) of grouting.

SECTION 8.2
REINFORCED MASONRY

8.2.1 **Scope.** The requirements of this section are in addition to the requirements of Section 8.1 and govern masonry design in which reinforcement is used to resist tensile forces.

8.2.2 **Design assumptions.** The following assumptions apply to the design of reinforced masonry:

(a) There is strain continuity between the reinforcement, grout, and masonry such that all applicable loads are resisted in a composite manner.

(b) The nominal strength of reinforced masonry cross-sections for combined flexure and axial load shall be based on applicable conditions of equilibrium.

(c) The maximum usable strain, ε_{mu} at the extreme masonry compression fiber shall be assumed to be 0.0035 for clay masonry and 0.0025 for concrete masonry.

(d) Strain in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis.

(e) Reinforcement stress below specified yield strength, f_y, shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be taken equal to f_y.

(f) The tensile strength of masonry shall be neglected in calculating flexural strength but shall be considered in calculating deflection.

(g) The relationship between masonry compressive stress and masonry strain shall be assumed to be defined by the following:

Masonry stress of $0.80 f_m^{'}$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance a = 0.80 c, from the fiber of maximum compressive strain. The distance, c, from the fiber of maximum strain to the neutral axis shall be measured perpendicular to that axis. For out-of-plane bending, the width of the equivalent stress block shall not be taken greater than six times the nominal thickness of the masonry wall or the spacing between reinforcement, whichever is less. For in-plane bending of flanged walls, the effective flange width shall not exceed six times the thickness of the flange.

8.2.3 Reinforcement requirements and details

8.2.3.1 Reinforcing bar size limitations. Reinforcing bars used in masonry shall not be larger than (Dia 28). The nominal bar diameter shall not exceed one-eighth of the nominal member thickness and shall not exceed one-quarter of the least clear dimension of the cell, course, or collar joint in which it is placed. The area of reinforcing bars placed in a cell or in a course of hollow unit construction shall not exceed 4 percent of the cell area.

8.2.3.2 Standard hooks. The equivalent embedment length to develop standard hooks in tension, l_e, shall be determined by Eq. (8-12):

$$l_e = 13 d_b \qquad \textbf{Eq. (8-12)}$$

8.2.3.3 Development. The required tension or compression reinforcement shall be developed in accordance with the following provisions:

The required development length of reinforcement shall be determined by Eq. (8-13), but shall not be less than 300 mm.

$$l_d = \frac{l_{de}}{\phi} \qquad \textbf{Eq. (8-13)}$$

where:

$$l_d = \frac{1.5 d_b^2 f_y \gamma}{K\sqrt{f_m^{'}}} \qquad \textbf{Eq. (8-14)}$$

K = shall not exceed the lesser of the masonry cover, clear spacing between adjacent reinforcement, nor 5 times d_b.

γ = 1.0 for (Dia 10) through (Dia 16) bars;

γ = 1.4 for (Dia 20) and (Dia 22) bars; and

γ = 1.5 for (Dia 25) and (Dia 28) bars.

8.2.3.3.1 Development of shear reinforcement. Shear reinforcement shall extend the depth of the member less cover distances.

8.2.3.3.1.1 Except at wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements of Section 8.2.4.1.2 shall be bent around the edge vertical reinforcing bar with a 180-degree hook. The ends of single leg or U-stirrups shall be anchored by one of the following means:

(a) A standard hook plus an effective embedment of $l_d/2$. The effective embedment of a stirrup leg shall be taken as the distance between the mid-depth of the member, $d/2$, and the start of the hook (point of tangency).

(b) For (Dia 16) bars and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of $l_d/3$. The $l_d/3$ embedment of a stirrup leg shall be taken as the distance between mid-depth of the member, *d/2*, and the start of the hook (point of tangency).

(c) Between the anchored ends, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

8.2.3.3.1.2 At wall intersections, horizontal reinforcing bars needed to satisfy shear strength requirements of Section 8.2.4.1.2 shall be bent around the edge vertical reinforcing bar with a 90-degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.

8.2.3.4 **Splices.** Reinforcement splices shall comply with one of the following:

(a) The minimum length of lap for bars shall be 300 mm or the length determined by Eq. (8-15), whichever is greater.

$$l_d = \frac{l_{de}}{\phi} \quad \textbf{Eq. (8-15)}$$

(b) A welded splice shall have the bars butted and welded to develop at least 125% of the yield strength, f_y, of the bar in tension or compression, as required. Welded splices shall be of ASTM A 706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls or special moment frames of masonry.

(c) Mechanical splices shall be classified as Type 1 or 2 according to SBC 304. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-column joint of intermediate or special reinforced masonry shear walls or special moment frames. Type 2 mechanical splices are permitted in any location within a member.

8.2.3.5 **Maximum reinforcement percentages**

8.2.3.5.1 For structures designed using an *R* value greater than 1.5, the ratio of reinforcement, ρ, shall not exceed the lesser of the values required to satisfy the following two critical strain conditions:

(a) For walls subjected to in-plane forces, for columns, and for beam, a strain of 5 times yield in the extreme tension reinforcement and a maximum masonry strain defined by Section 8.2.2(c).

(b) For walls subjected to out-of-plane forces, a strain of 1.3 times yield in the extreme tension reinforcement and a maximum masonry strain defined by Section 8.2.2(c).

In calculating the maximum reinforcement ratio for each case, equilibrium shall include unfactored gravity axial loads. The stress in the tension reinforcement shall be assumed to be 1.25 f_y. Tension in the masonry shall be neglected. The strength of the compression zone shall be calculated as 80 percent of f_m' times 80 percent of the area of the compressive zone. Stress in reinforcement in the compression zone shall be based on a linear strain distribution.

8.2.3.5.2 For structures designed using an *R* value less than or equal to 1.5, the ratio of reinforcement, ρ, shall not exceed the ratio as calculated using the following critical strain condition:

A strain of 2 times yield in the extreme tension reinforcement and a maximum masonry strain defined by Section 8.2.2(c). In calculating the maximum reinforcement ratio, equilibrium shall include unfactored gravity axial loads. The stress in the tension reinforcement shall be assumed to be 1.25 f_y. Tension in the masonry shall be neglected. The strength of the compression zone shall be calculated as 80% of f'_m times 80 percent of the area of the compressive zone. Stress in reinforcement in the compression zone shall be based on a linear strain distribution.

8.2.3.6 **Bundling of reinforcing bars.** Reinforcing bars shall not be bundled.

8.2.4 Design of beams, piers, and columns

Member design forces shall be based on an analysis that considers the relative stiffness of structural members. The calculation of lateral stiffness shall include the contribution of all beam, piers, and columns. The effects of cracking on member stiffness shall be considered.

8.2.4.1 Nominal strength

8.2.4.1.1 **Nominal axial and flexural strength**. The nominal axial strength, P_n, and the nominal flexural strength, M_n, of a cross section shall be determined in accordance with the design assumptions of Section 8.2.2 and the provisions of Section 8.2.4.1. Using the slenderness-dependent modification factors of Eq. (8-16) $(1-(h/140r)^2)$ and Eq. (8-17) $(70r/h)^2$, as appropriate, the nominal axial strength shall be modified for the effects of slenderness. The nominal flexural strength at any section along a member shall not be less than one fourth of the maximum nominal flexural strength at the critical section.

The nominal axial compressive strength shall not exceed Eq. (8-16) or Eq. (8-17), as appropriate.

(a) For members having an h/r ratio not greater than 99:

$$P_n = 0.80[0.80\ f'_m\ (A_n - A_s) + f_y\ A_s\ ] \left(1-\left(\frac{h}{140r}\right)^2\right) \qquad \textbf{Eq. (8-16)}$$

(b) For members having an h/r ratio greater than 99:

$$P_n = 0.80[0.80\ f'_m\ (A_n - A_s) + f_y\ A_s\ ] \left(\frac{70r}{h}\right)^2 \qquad \textbf{Eq. (8-17)}$$

8.2.4.1.2 **Nominal shear strength.** Nominal shear strength, V_n, shall be computed using Eq. (8-18) and either Eq. (8-19) or Eq. (8-20), as appropriate.

$$V_n = V_m + V_s \qquad \textbf{Eq. (8-18)}$$

where V_n shall not exceed the following:

(a) Where $M/Vd_v \leq 0.25$:

$$V_n \leq 6A_n\sqrt{f'_m} \qquad \textbf{Eq. (8-19)}$$

(b) Where $M/Vd_v \geq 1.00$

$$V_n \leq 4A_n\sqrt{f'_m} \qquad \textbf{Eq. (8-20)}$$

(c) The maximum value of V_n for M/Vd_v between 0.25 and 1.0 may be interpolated.

8.2.4.1.2.1 **Nominal masonry shear strength** — Shear strength provided by the masonry, V_m, shall be computed using Eq. (8-21):

$$V_m = 0.83 \left[4.0 - 1.75 \left(\frac{M}{Vd_v} \right) \right] A_n \sqrt{f'_m} + 0.25\, P \qquad \textbf{Eq. (8-21)}$$

M/Vd_v need not be taken greater than 1.0.

8.2.4.1.2.2 **Nominal shear strength provided by reinforcement** — Nominal shear strength provided by reinforcement, V_s, shall be computed as follows:

$$V_s = 0.5 \left(\frac{A_v}{s} \right) f_y\, d_v \qquad \textbf{Eq. (8-22)}$$

8.2.4.2 **Beams**

8.2.4.2.1 Members designed primarily to resist flexure shall comply with the requirements of Section 8.2.4.2. The factored axial compressive force on a beam shall not exceed $0.05 A_n f'_m$.

8.2.4.2.2 **Longitudinal reinforcement**

8.2.4.2.2.1 The variation in longitudinal reinforcing bars in a beam shall not be greater than one bar size. Not more than two bar sizes shall be used in a beam.

8.2.4.2.2.2 The nominal flexural strength of a beam shall not be less than 1.3 times the nominal cracking moment strength of the beam, M_{cr}. The modulus of rupture, f_r, for this calculation shall be determined in accordance with Section 8.1.7.2.

8.2.4.2.3 **Transverse reinforcement.** Transverse reinforcement shall be provided where V_u exceeds ϕV_m. The factored shear, V_u, shall include the effects of lateral load. When transverse reinforcement is required, the following provisions shall apply:

(a) Transverse reinforcement shall be a single bar with a 180-degree hook at each end.

(b) Transverse reinforcement shall be hooked around the longitudinal reinforcement.

(c) The minimum area of transverse reinforcement shall be 0.0007 bd_v.

(d) The first transverse bar shall not be located more than one fourth of the beam depth, d_v, from the end of the beam.

(e) The maximum spacing shall not exceed one-half the depth of the beam nor 1200 mm.

8.2.4.2.4 **Construction.** Beams shall be grouted solid.

8.2.4.2.5 **Dimensional limits.** Dimensions shall be in accordance with the following:

(a) The clear distance between locations of lateral bracing of the compression side of the beam shall not exceed 32 times the least width of the compression area.

(b) The nominal depth of a beam shall not be less than 200 mm.

8.2.4.3 **Piers**

8.2.4.3.1 The factored axial compression force on the piers shall not exceed $0.3 A_n f'_m$.

8.2.4.3.2 **Longitudinal reinforcement.** A pier subjected to in-plane stress reversals shall be reinforced symmetrically about the neutral axis of the pier. The longitudinal reinforcement of all piers shall comply with the following:

(a) One bar shall be provided in the end cells.

(b) The minimum area of longitudinal reinforcement shall be 0.0007 bd.

(c) Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.

8.2.4.3.3 **Dimensional limits.** Dimensions shall be in accordance with the following:

(a) The nominal thickness of a pier shall not be less than 150 mm and shall not exceed 400 mm.

(b) The distance between lateral supports of a pier shall not exceed 25 times the nominal thickness of a pier except as provided for in Section 8.2.4.3.3(c).

(c) When the distance between lateral supports of a pier exceeds 25 times the nominal thickness of the pier, design shall be based on the provisions of Section 8.2.5.

(d) The nominal length of a pier shall not be less than three times its nominal thickness nor greater than six times its nominal thickness. The clear height of a pier shall not exceed five times its nominal length.

Exception: When the factored axial force at the location of maximum moment is less than 0.05 f'_m A_g, the length of a pier may be equal to the thickness of the pier.

8.2.4.4 Columns

8.2.4.4.1 **Longitudinal reinforcement.** Longitudinal reinforcement shall be a minimum of four bars, one in each corner of the column, and shall comply with the following:

(a) Maximum reinforcement area shall be determined in accordance with Section 8.2.3.5, but shall not exceed 0.04 A_n.

(b) Minimum reinforcement area shall be 0.0025 A_n.

(c) Longitudinal reinforcement shal1 be uniformly distributed throughout the depth of the element.

8.2.4.4.2 **Lateral ties.** Lateral ties shall be provided in accordance with Section 7.1.6.5.

8.2.4.4.3 **Construction.** Columns shall be solid grouted.

8.2.4.4.4 **Dimensional limits.** Dimensions shall be in accordance with the following:

(a) The nominal width of a column shall not be less than 200 mm.

(b) The distance between lateral supports of a column shall not exceed 30 times its nominal width.

(c) The nominal depth of a column shall not be less than 200 mm and not greater than three times its nominal width.

8.2.5 Wall design for out-of-plane loads

8.2.5.1 **General.** The requirements of Section 8.2.5 are for the design of walls for out-of-plane loads.

8.2.5.2 **Maximum reinforcement**. The maximum reinforcement ratio shall be determined by Section 8.2.3.5.

8.2.5.3 **Moment and deflection calculations.** All moment and deflection calculations in Section 8.2.5.4 are based on simple support conditions top and bottom. For other support and fixity conditions, moments and deflections shall be calculated using established principles of mechanics.

8.2.5.4 **Walls with factored axial stress of 0.05 $f_m^{'}$ or less.** The procedures set forth in this section shall be used when the factored axial load stress at the location of maximum moment satisfies the requirement computed by Eq. (8-23).

$$\left(\frac{P_u}{A_g}\right) \leq 0.05\, f_m^{'} \qquad \textbf{Eq. (8-23)}$$

Factored moment and axial force shall be determined at the midheight of the wall and shall be used for design. The factored moment, M_u, at the midheight of the wall shall be computed using Eq. (8-24).

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u \qquad \textbf{Eq. (8-24)}$$

where:

$$P_u = P_{uw} + P_{uf} \qquad \textbf{Eq. (8-25)}$$

The design strength for out-of-plane wall loading shall be in accordance with Eq. (8-26).

$$M_u \leq \phi\, M_n \qquad \textbf{Eq. (8-26)}$$

where:

$$M_n = (A_s f_y + P_u)\left(d - \frac{a}{2}\right) \qquad \textbf{Eq. (8-27)}$$

$$a = \frac{\left(P_u + A_s f_y\right)}{0.80 f'_m\, b} \qquad \textbf{Eq. (8-28)}$$

The nominal shear strength shall be determined by Section 8.2.4.1.2.

8.2.5.5 **Walls with factored axial stress greater than 0.05 $f_m^{'}$.** The procedures set forth in this section shall be used for the design of masonry walls when the factored axial load stress at the location of maximum moment exceeds $0.05\, f_m^{'}$. These provisions shall not be applied to walls with factored axial load stress equal to or exceeding $0.2\, f_m^{'}$ or slenderness ratios exceeding 30. Such walls shall be designed in accordance with the provisions of Section 8.2.5.4 and shall have a minimum nominal thickness of 150 mm.

The nominal shear strength shall be determined by Section 8.2.4.1.2.

8.2.5.6 **Deflection design**. The horizontal midheight deflection, δ_s, under service lateral and service axial loads (without load factors) shall be limited by the relation:

$$\delta_s \leq 0.007\, h \qquad \textbf{Eq. (8-29)}$$

P-delta effects shall be included in deflection calculation. The midheight deflection shall be computed using either Eq. (8-30) or Eq. (8-31), as applicable.

(a) Where $M_{ser} < M_{cr}$

$$\delta_s = \frac{5 M_{ser} h^2}{48 E_m I_g} \quad \text{For } M_{ser} \leq M_{cr} \qquad \textbf{Eq. (8-30)}$$

(b) Where $M_{cr} < M_{ser} < M_n$

$$\delta_s = \frac{5M_{cr}h^2}{48E_m l_g} + \frac{5(M_{ser} - M_{cr})h^2}{48E_m l_{cr}} \qquad \textbf{Eq. (8-31)}$$

The cracking moment strength of the wall shall be computed using Eq. (8-32):

$$M_{cr} = S_n f_r \qquad \textbf{Eq. (8-32)}$$

The modulus of rupture, f_r, shall be taken from Table 8.1.1.

8.2.6 Wall design for in-plane loads

8.2.6.1 Scope. The requirements of Section 8.2.6 are for the design of walls to resist in-plane loads.

8.2.6.2 Reinforcement. Reinforcement shall be in accordance with the following:

(a) The amount of vertical reinforcement shall not be less than one-half the horizontal reinforcement.

(b) The maximum reinforcement ratio shall be determined in accordance with Section 8.2.3.5.

8.2.6.3 Flexural and axial strength. The nominal flexural and axial strength shall be determined in accordance with Section 8.2.4.1.1.

8.2.6.4 Shear strength. The nominal shear strength shall be computed in accordance with Section 8.2.4.1.2.

SECTION 8.3
UNREINFORCED (PLAIN) MASONRY

8.3.1 Scope. The requirements of Section 8.3 are in addition to the requirements of Section 8.1 and govern masonry design in which masonry is used to resist tensile forces.

8.3.1.1 Strength for resisting loads. Unreinforced (plain) masonry members shall be designed using the strength of masonry units, mortar, and grout in resisting design loads.

8.3.1.2 Strength contribution from reinforcement. Stresses in reinforcement shall not be considered effective in resisting design loads.

8.3.1.3 Design criteria. Unreinforced (plain) masonry members shall be designed to remain uncracked.

8.3.2 Flexural strength of unreinforced (plain) masonry members. The following assumptions shall apply when determining the flexural strength of unreinforced (plain) masonry members:

(a) Strength design of members for factored flexure and axial load shall be in accordance with principles of engineering mechanics.

(b) Strain in masonry shall be directly proportional to the distance from the neutral axis.

(c) Flexural tension in masonry shall be assumed to be directly proportional to strain.

(d) Flexural compressive stress in combination with axial compressive stress in masonry shall be assumed directly proportional to strain. Nominal compressive strength shall not exceed a stress corresponding to $0.80\,f'_m$.

(e) The nominal flexural tensile strength of masonry shall be determined from Section 8.1.7.2.

8.3.3 **Nominal axial strength of unreinforced (plain) masonry members.** Nominal axial strength, P_n, shall be computed using Eq. (8-33) or Eq. (8-34).

(a) For members having an h/r ratio not greater than 99:

$$P_n = 0.80\left(0.80 A_n\ f'_m\left[1-\left(\frac{h}{140r}\right)^2\right]\right) \quad \text{for } \frac{h}{r} \leq 99 \qquad \textbf{Eq. (8-33)}$$

(b) For members having an h/r ratio greater than 99:

$$P_n = 0.80\left(0.80 A_n\ f'_m\left(\frac{70r}{h}\right)^2\right) \quad \text{for } \frac{h}{r} > 99 \qquad \textbf{Eq. (8-34)}$$

8.3.4 **Nominal shear strength** – Nominal shear strength, V_n, shall be the smallest of the following:

(a) $0.375\,A_n\sqrt{f'_m}$

(b) $0.83\,A_n$

(c) For running bond masonry not solidly grouted;

$0.26\,A_n + 0.3\,N_v$

(d) For stack bond masonry with open end units and grouted solid;

$0.26\,A_n + 0.3\,N_v$

(e) For running bond masonry grouted solid;

$0.414\,A_n + 0.3\,N_v$

(f) For stack bond other than open end units grouted solid.

$0.103 A_n$

CHAPTER 9
EMPIRICAL DESIGN OF MASONRY

SECTION 9.1
GENERAL

9.1.0 Empirically designed masonry shall conform to this chapter. The provisions of Chapter 1 through Chapter 5, excluding Sections 1.3(item 5), 1.3(item 6), 1.4, 3.12, 3.13, 5.2, shall apply to empirical design.

9.1.1 **Limitations.** Empirical masonry design shall not be utilized for any of the following conditions:

1. The design or construction of masonry in buildings assigned to Seismic Design Category D, as specified in SBC 301, and the design of the seismic-force-resisting system for buildings assigned to Seismic Design Category B or C.
2. The design or construction of masonry structures located in areas where the 3-second gust wind speed exceeds 145 km/hr.
3. Buildings more than 10700 mm in height which have masonry wall lateral-force-resisting systems.
4. Glass unit masonry.

In buildings that exceed one or more of the above limitations, masonry shall be designed in accordance with the engineered design provisions of Chapter 7 or Chapter 8, or the foundation wall provisions of Foundation Walls (SBC 303).

SECTION 9.2
LATERAL STABILITY

9.2.1 **Shear walls.** Where the structure depends upon masonry walls for lateral stability, shear walls shall be provided parallel to the direction of the lateral forces resisted.

9.2.1.1 **Shear wall thickness.** Minimum nominal thickness of masonry shear walls shall be 200 mm.

Exception: Shear walls of one-story buildings are permitted to be a minimum nominal thickness of 150 mm.

9.2.1.2 **Cumulative length of shear walls.** In each direction in which shear walls are required for lateral stability, shear walls shall be positioned in two separate planes. The minimum cumulative length of shear walls provided shall be 0.4 times the long dimension of the building. Cumulative length of shear walls shall not include openings or any element whose length is less than one-half its height.

9.2.1.3 **Maximum diaphragm ratio.** Masonry shear walls shall be spaced so that the length-to-width ratio of each diaphragm transferring lateral forces to the shear walls does not exceed the values given in Table 9.2.1.

TABLE 9.2.1
DIAPHRAGM LENGTH-TO-WIDTH RATIOS

FLOOR OR ROOF DIAPHRAGM CONSTRUCTION	MAXIMUM LENGTH-TO-WIDTH RATIO OF DIAPHRAGM PANEL*
Cast-in-place concrete	5:1
Precast concrete	4:1
Metal deck with concrete fill	3:1
Metal deck with no fill	2:1

* Diaphram Panel length: Dimension perpendicular to the resisting shear wall.
Diaphram Panel width: Dimension parallel to the resisting shear wall.

9.2.2 **Roofs.** The roof construction shall be designed so as not to impart out-of-plane lateral thrust to the walls under roof gravity load.

9.2.3 **Surface-bonded walls.** Dry-stacked, surface-bonded concrete masonry walls shall comply with the requirements of this code for masonry wall construction, except where otherwise noted.

9.2.3.1 **Strength.** Dry-stacked, surface-bonded concrete masonry walls shall be of adequate strength and proportions to support all superimposed loads without exceeding the allowable stresses listed in Table 9.2.2.

TABLE 9.2.2
ALLOWABLE STRESS GROSS CROSS-SECTIONAL AREA FOR DRY-STACKED, SURFACE-BONDED CONCRETE MASONRY WALLS

DESCRIPTION	MAXIMUM ALLOWABLE STRESS kPa
Compression standard block	310
Shear	70
Flexural tension	
Vertical span	124
Horizontal span	207

9.2.3.2 **Construction.** Construction of dry-stacked, surface-bonded masonry walls, including stacking and leveling of units, mixing and application of mortar and curing and protection shall comply with ASTM C 946.

SECTION 9.3
COMPRESSIVE STRESS REQUIREMENTS

9.3.1 **Calculations.** Compressive stresses in masonry due to vertical dead plus live loads, excluding wind or seismic loads, shall be determined in accordance with Section 9.3.2.

9.3.1.1 Dead and live loads shall be in accordance with SBC 301 with live load reductions as permitted in SBC 301.

9.3.2 **Allowable compressive stresses.** The compressive stresses in masonry shall not exceed the values given in Table 9.3.1. Stress shall be calculated based on specified rather than nominal dimensions.

TABLE 9.3.1
ALLOWABLE COMPRESSIVE STRESSES FOR EMPIRICAL DESIGN OF MASONRY

Construction; Compressive Strength of Unit Gross Area MPa	Allowable compressive stresses[a] gross cross-sectional area, MPa	
	Type M or S mortar	Type N mortar
Solid masonry of brick and other solid units of clay or shale; sand-lime or concrete brick:		
56 or greater	2.5	2
31	1.6	1.4
17.5	1.1	1.0
10.5	0.8	0.7
Grouted masonry, of clay or shale; sand-lime or concrete:		
31 or greater	1.6	1.4
17.5	1.1	1.0
10.5	0.8	0.7
Solid masonry of solid concrete masonry units:		
21 or greater	1.6	1.4
14	1.1	1.0
8.5	0.8	0.7
Masonry of hollow load-bearing units:		
14 or greater	1.0	0.8
10.5	0.8	0.7
7.0	0.5	0.5
5.0	0.4	0.4
Hollow walls (noncomposite masonry bonded)[b]		
Solid units:		
17.5 or greater	1.1	1.0
10.5	0.8	0.7
Hollow units	0.5	0.4
Stone ashlar masonry:		
Granite	5.0	4.5
Limestone or marble	3.0	2.8
Sandstone or cast stone	2.5	2.2
Rubble stone masonry		
Coursed, rough or random	0.8	0.7

a. Linear interpolation for determining allowable stresses for masonry units having compressive strengths which are intermediate between those given in the table is permitted.

b. Where floor and roof loads are carried upon one wythe, the gross cross-sectional area is that of the wythe under load; if both wythes are loaded, the gross cross-sectional area is that of the wall minus the area of the cavity between the wythes. Walls bonded with metal ties shall be considered as noncomposite walls unless collar joints are filled with mortar or grout.

9.3.2.1 **Calculated compressive stresses.** Calculated compressive stresses for single wythe walls and for multiwythe composite masonry walls shall be determined by dividing the design load by the gross cross-sectional area of the member. The area of openings, chases or recesses in walls shall not be included in the gross cross-sectional area of the wall.

9.3.2.2 **Multiwythe walls.** The allowable stress shall be as given in Table 9.3.1 for the weakest combination of the units used in each wythe.

SECTION 9.4
LATERAL SUPPORT

9.4.1 **Intervals.** Masonry walls shall be laterally supported in either the horizontal or vertical direction at intervals not exceeding those given in Table 9.4.1.

TABLE 9.4.1
WALL LATERAL SUPPORT REQUIREMENTS

Construction	Maximum Wall Length to Thickness or Wall Height to Thickness (Ratio)
Bearing walls	
Solid units or fully grouted	20
All others	18
Nonbearing walls	
Exterior	18
Interior	36

9.4.2 **Thickness.** Except for cavity walls and cantilever walls, the thickness of a wall shall be its nominal thickness measured perpendicular to the face of the wall. For cavity walls, the thickness shall be determined as the sum of the nominal thicknesses of the individual wythes. For cantilever walls, except for parapets, the ratio of height-to-nominal thickness shall not exceed six for solid masonry or four for hollow masonry. For parapets, see Section 9.5.5.

In computing the ratio for multiwythe walls, use the following thickness:

1. The nominal wall thickness for solid walls and for hollow walls bonded with masonry headers Section 9.6.2.
2. The sum of the nominal thicknesses of the wythes for non-composite walls connected with wall ties Section 9.6.3.

9.4.3 **Support elements.** Lateral support shall be provided by cross walls, pilasters, buttresses or structural frame members when the limiting distance is taken horizontally, or by floors, roofs acting as diaphragms or structural frame members when the limiting distance is taken vertically.

SECTION 9.5
THICKNESS OF MASONRY

9.5.0 Minimum thickness requirements shall be based on nominal dimensions of masonry.

9.5.1 **Thickness of walls.** The thickness of masonry walls shall conform to the requirements of Section 9.5.

9.5.2 **Minimum thickness.** The minimum thickness of masonry bearing walls more than one story high shall be 200 mm. Bearing walls of one-story buildings shall not be less than 150 mm thick.

9.5.3 **Rubble stone walls.** The minimum thickness of rough or random or coursed rubble stone walls shall be 400 mm.

9.5.4 **Change in thickness.** Where walls of masonry of hollow units or masonry bonded hollow walls are decreased in thickness, a course or courses of solid masonry shall be interposed between the wall below and the thinner wall above, or special units or construction shall be used to transmit the loads from face shells or wythes above to those below.

9.5.5 Parapet walls

9.5.5.1 **Minimum thickness.** Unreinforced parapet walls shall be at least 200 mm thick, and their height shall not exceed three times their thickness.

9.5.5.2 **Additional provisions.** Additional provisions for parapet walls are contained in Flashing and Coping (Roof Assemblies and Rooftop Structures).

9.5.5.3 **Reinforcement.** Parapet walls shall be reinforced. The reinforcement shall be secured to slabs.

9.5.6 **Foundation walls.** Foundation walls shall comply with the requirements of Sections 9.5.6.1 and 9.5.6.2.

9.5.6.1 **Minimum thickness.** Minimum thickness for foundation walls shall comply with the requirements of Table 9.5.1. The provisions of Table 9.5.1 are only applicable where the following conditions are met:

1. The foundation wall does not exceed 2400 mm in height between lateral supports,
2. The terrain surrounding foundation walls is graded to drain surface water away from foundation walls,
3. Backfill is drained to remove ground water away from foundation walls,
4. Lateral support is provided at the top of foundation walls prior to backfilling,
5. The length of foundation walls between perpendicular masonry walls or pilasters is a maximum of three times the basement wall height,
6. The backfill is granular and soil conditions in the area are nonexpansive, and
7. Masonry is laid in running bond using Type M or S mortar.

9.5.6.2 **Design requirements.** Where the requirements of Section 9.5.6.1 are not met, foundation walls shall be designed in accordance with SBC 303.

TABLE 9.5.1
FOUNDATION WALL CONSTRUCTION

Wall Construction	Nominal Wall Thickness mm	Maximum Depth of Unbalanced Backfill m
Hollow unit masonry	200 250 300	1.50 1.80 2.10
Solid unit masonry	200 250 300	1.50 2.10 2.10
Fully grouted masonry	200 250 300	2.10 2.50 2.50

SECTION 9.6
BOND

9.6.1 **General.** The facing and backing of multiwythe masonry walls shall be bonded in accordance with Section 9.6.2, 9.6.3 or 9.6.4.

9.6.2 **Bonding with masonry headers.**

9.6.2.1 **Solid units.** Where the facing and backing (adjacent wythes) of solid masonry construction are bonded by means of masonry headers, no less than 4 percent of the wall surface of each face shall be composed of headers extending not less than 75 mm into the backing. The distance between adjacent full-length headers shall not exceed 600 mm either vertically or horizontally. In walls in which a single header does not extend through the wall, headers from the opposite sides shall overlap at least 75 mm, or headers from opposite sides shall be covered with another header course overlapping the header below at least 75 mm.

9.6.2.2 **Hollow units.** Where two or more hollow units are used to make up the thickness of a wall, the stretcher courses shall be bonded at vertical intervals not exceeding 860 mm by lapping at least 75 mm over the unit below, or by lapping at vertical intervals not exceeding 400 mm with units that are at least 50 percent greater in thickness than the units below.

9.6.2.3 **Masonry bonded hollow walls.** In masonry bonded hollow walls, the facing and backing shall be bonded so that not less than 4 percent of the wall surface of each face is composed of masonry bonded units extending not less than 75 mm into the backing. The distance between adjacent bonders shall not exceed 600 mm either vertically or horizontally.

9.6.3 **Bonding with wall ties or joint reinforcement.**

9.6.3.1 **Bonding with wall ties.** Except as required by Section 9.6.3.1.1, where the facing and backing (adjacent wythes) of masonry walls are bonded with wire size (WD 5) wall ties or metal wire of equivalent stiffness embedded in the horizontal mortar joints, there shall be at least one metal tie for each 0.42 m^2 of wall area. The maximum vertical distance between ties shall not exceed 600 mm, and the maximum horizontal distance shall not exceed 900 mm. Rods or ties bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical. In other walls, the ends of ties shall be bent to 1.5 rad angles to provide hooks no less than 50 mm long. Wall ties shall be without drips. Additional bonding ties shall be provided at all openings spaced not more than 900 mm apart around the perimeter and within 300 mm of the opening.

9.6.3.1.1 **Bonding with adjustable wall ties.** Where the facing and backing (adjacent wythes) of masonry are bonded with adjustable wall ties, there shall be at least one tie for each 0.16 m^2 of wall area. Neither the vertical nor horizontal spacing of the adjustable wall ties shall exceed 400 mm. The maximum vertical offset of bed joints from one wythe to the other shall be 30 mm. The maximum clearance between connecting parts of the ties shall be 1.6 mm. When pintle legs are used, ties shall have at least two wire size (WD 5) legs.

9.6.3.2 **Bonding with prefabricated joint reinforcement.** Where the facing and backing (adjacent wythes) of masonry are bonded with prefabricated joint reinforcement, there shall be at least one cross wire serving as a tie for each 0.25 m^2 of wall area. The vertical spacing of the joint reinforcing shall not exceed 600 mm. Cross wires on prefabricated joint reinforcement shall not be less than (WD 4) and shall be without drips. The longitudinal wires shall be embedded in the mortar.

9.6.4 Bonding with natural or cast stone.

9.6.4.1 Ashlar masonry. In ashlar masonry, bonder units, uniformly distributed, shall be provided to the extent of not less than 10 percent of the wall area. Such bonder units shall extend not less than 100 mm into the backing wall.

9.6.4.2 Rubble stone masonry. Rubble stone masonry 600 mm or less in thickness shall have bonder units with a maximum spacing of 900 mm vertically and 900 mm horizontally, and if the masonry is of greater thickness than 600 mm, shall have one bonder unit for each 0.5 m^2 of wall surface on both sides.

9.6.5 Masonry bonding pattern.

9.6.5.1 Masonry laid in running bond. Each wythe of masonry shall be laid in running bond, head joints in successive courses shall be offset by not less than one-fourth the unit length or the masonry walls shall be reinforced longitudinally as required in Section 9.6.5.2.

9.6.5.2 Masonry laid in stack bond. Where unit masonry is laid with less head joint offset than in Section 9.6.5.1, the minimum area of horizontal reinforcement placed in mortar bed joints or in bond beams spaced not more than 1200 mm apart, shall be 0.0003 times the vertical cross-sectional area of the wall.

SECTION 9.7
ANCHORAGE

9.7.1 General. Masonry elements shall be anchored in accordance with Section 9.7.2 through 9.7.4.

9.7.2 Intersecting walls. Masonry walls depending upon one another for lateral support shall be anchored or bonded at locations where they meet or intersect by one of the methods indicated in Sections 9.7.2.1 through 9.7.2.5.

9.7.2.1 Bonding pattern. Fifty percent of the units at the intersection shall be laid in an overlapping masonry bonding pattern, with alternate units having a bearing of not less than 75 mm on the unit below.

9.7.2.2 Steel connectors. Walls shall be anchored by steel connectors having a minimum section of 6 mm by 40 mm, with ends bent up at least 50 mm or with cross pins to form anchorage. Such anchors shall be at least 600 mm long and the maximum spacing shall be 1200 mm.

9.7.2.3 Joint reinforcement. Walls shall be anchored by joint reinforcement spaced at a maximum distance of 200 mm. Longitudinal wires of such reinforcement shall be at least wire size (WD 4) and shall extend at least 800 mm in each direction at the intersection.

9.7.2.4 Interior nonload-bearing walls. Interior nonload-bearing walls shall be anchored at their intersection, at vertical intervals of not more than 400 mm with joint reinforcement or 6.5 mm mesh galvanized hardware cloth.

9.7.2.5 Ties, joint reinforcement or anchors. Other metal ties, joint reinforcement or anchors, if used, shall be spaced to provide equivalent area of anchorage to that required by this section.

9.7.3 **Floor and roof anchorage.** Floor and roof diaphragms providing lateral support to masonry shall comply with the live loads in Uniform Live Loads (Structural Design) and shall be connected to the masonry in accordance with Section 9.7.3.1 through 9.7.3.3.

9.7.3.1 **Steel floor joists.** Steel floor joists bearing on masonry walls shall be anchored to the wall with 10 mm round bars, or their equivalent, spaced not more than 1800 mm o.c. Where joists are parallel to the wall, anchors shall be located at joist bridging.

9.7.3.2 **Roof diaphragms.** Roof diaphragms shall be anchored to masonry walls with 13 mm bolts, 1800 mm o.c. or their equivalent. Bolts shall extend and be embedded at least 400 mm into the masonry, or be hooked or welded to not less than 130 mm^2 of bond beam reinforcement placed not less than 150 mm from the top of the wall.

9.7.4 **Walls adjoining structural framing.** Where walls are dependent upon the structural frame for lateral support, they shall be anchored to the structural members with metal anchors or otherwise keyed to the structural members. Metal anchors shall consist of 13 mm bolts spaced at 1200 mm o.c. embedded 100 mm into the masonry, or their equivalent area.

SECTION 9.8
ADOBE CONSTRUCTION

9.8.0 Adobe construction shall comply with this section and shall be subject to the requirements of this code.

9.8.1 **Unstabilized adobe.**

9.8.1.1 **Compressive strength.** Adobe units shall have an average compressive strength of 2000 kPa when tested in accordance with ASTM C 67. Five samples shall be tested and no individual unit is permitted to have a compressive strength of less than 1750 kPa.

9.8.1.2 **Modulus of rupture.** Adobe units shall have an average modulus of rupture of 350 kPa when tested in accordance with the following procedure. Five samples shall be tested and no individual unit shall have a modulus of rupture of less than 240 kPa.

9.8.1.2.1 **Support conditions.** A cured unit shall be simply supported by 50 mm cylindrical supports located 50 mm in from each end and extending the full width of the unit.

9.8.1.2.2 **Loading conditions.** A 50 mm cylinder shall be placed at midspan parallel to the supports.

9.8.1.2.3 **Testing procedure.** A vertical load shall be applied to the cylinder at the rate of 37 N/s until failure occurs.

9.8.1.2.4 **Modulus of rupture determination.** The modulus of rupture shall be determined by the equation:

$$f_r = 3WL_s / 2bt^2 \qquad \textbf{Eq. (9-1)}$$

where, for the purposes of this section only:

b = Width of the test specimen measured parallel to the loading cylinder, mm.

f_r = Modulus of rupture, MPa.

L_s = Distance between supports, mm.

t = Thickness of the test specimen measured parallel to the direction of load, mm.

W = The applied load at failure, N.

9.8.1.3 **Moisture content requirements.** Adobe units shall have a moisture content not exceeding 4 percent by weight.

9.8.1.4 **Shrinkage cracks.** Adobe units shall not contain more than three shrinkage cracks and any single shrinkage crack shall not exceed 75 mm in length or 3.0 mm in width.

9.8.2 **Stabilized adobe.**

9.8.2.1 **Material requirements.** Stabilized adobe shall comply with the material requirements of unstabilized adobe in addition to Section 9.8.2.1.1 and 9.8.2.1.2.

9.8.2.1.1 **Soil requirements.** Soil used for stabilized adobe units shall be chemically compatible with the stabilizing material.

9.8.2.1.2 **Absorption requirements.** A 100 mm cube, cut from a stabilized adobe unit dried to a constant weight in a ventilated oven at 100 °C to 115 °C, shall not absorb more than 2-1/2% moisture by weight when placed upon a constantly water-saturated, porous surface for seven days. A minimum of five specimens shall be tested and each specimen shall be cut from a separate unit.

9.8.3 **Working stress.** The allowable compressive stress based on gross cross-sectional area of adobe shall not exceed 200 kPa.

9.8.3.1 **Bolts.** Bolt values shall not exceed those set forth in Table 9.8.1.

TABLE 9.8.1
ALLOWABLE SHEAR ON BOLTS IN ADOBE MASONRY

Diameter of Bolts mm	Minimum Embedment mm	Shear N
14	–	–
16	300	900
20	400	1400
22	500	1800
26	550	2250
30	600	2700

9.8.4 **Construction.**

9.8.4.1 **General.**

9.8.4.1.1 **Height restrictions.** Adobe construction shall be limited to buildings not exceeding one story, except that two-story construction is allowed when designed by a registered design professional.

9.8.4.1.2 **Mortar restrictions.** Mortar for stabilized adobe units shall comply with this Code or adobe soil. Adobe soil used as mortar shall comply with material requirements for stabilized adobe. Mortar for unstabilized adobe shall be Portland cement mortar.

9.8.4.1.3 **Mortar joints.** Adobe units shall be laid with full head and bed joints and in full running bond.

9.8.4.1.4 **Parapet walls.** Parapet walls constructed of adobe units shall be waterproofed.

9.8.4.2 **Wall thickness.** The minimum thickness of exterior walls in one-story buildings shall be 250 mm. The walls shall be laterally supported at intervals not exceeding 7000 mm. The minimum thickness of interior load-bearing walls shall be 200 mm. In no case shall the unsupported height of any wall constructed of adobe units exceed 10 times the thickness of such wall.

9.8.4.3 **Foundations.**

9.8.4.3.1 **Foundation support.** Walls and partitions constructed of adobe units shall be supported by foundations or footings that extend not less than 150 mm above adjacent ground surfaces and are constructed of solid masonry (excluding adobe) or concrete. Footings and foundations shall comply with SBC 303.

9.8.4.3.2 **Lower course requirements.** Stabilized adobe units shall be used in adobe walls for the first 100 mm above the finished first-floor elevation.

9.8.4.4 **Isolated piers or columns.** Adobe units shall not be used for isolated piers or columns in a load-bearing capacity. Walls less than 600 mm in length shall be considered isolated piers or columns.

9.8.4.5 **Tie beams.** Exterior walls and interior load-bearing walls constructed of adobe units shall have a continuous tie beam at the level of the floor or roof bearing and meeting the following requirements.

9.8.4.5.1 **Concrete tie beams.** Concrete tie beams shall be a minimum depth of 150 mm and a minimum width of 250 mm. Concrete tie beams shall be continuously reinforced with a minimum of two Dia 12 mm reinforcing bars. The ultimate compressive strength of concrete shall be at least 20 MPa at 28 days.

9.8.4.6 **Exterior finish.** Exterior walls constructed of unstabilized adobe units shall have their exterior surface covered with a minimum of two coats of Portland cement plaster having a minimum thickness of 20 mm and conforming to ANSI A42.2. Lathing shall comply with ANSI A42.3. Fasteners shall be spaced at 400 mm o.c. maximum. Exposed wood surfaces shall be treated with an approved wood preservative or other protective coating prior to lath application.

SECTION 9.9
MISCELLANEOUS REQUIREMENT

9.9.1 **Chases and recesses.** Masonry directly above chases or recesses wider than 300 mm shall be supported on lintels.

9.9.2 **Lintels.** Lintels shall be considered structural members and shall be designed in accordance with the applicable provisions of SBC 301.

9.9.3 **Support on wood.** No masonry shall be supported on wood girders or other forms of wood construction.

9.9.4 **Corbelling.** Solid masonry units shall be used for corbelling. The maximum corbelled projection beyond the face of the wall shall be not more than one-half of the wall thickness or one-half the wythe thickness for hollow walls; the maximum projection of one unit shall neither exceed one-half the height of the unit nor one-third its thickness at right angles to the wall.

CHAPTER 10
GLASS UNIT MASONRY

SECTION 10.1

10.1.0 This section covers the empirical requirements for nonload-bearing glass unit masonry elements in exterior or interior walls.

The provisions of Chapter 1 through Chapter 4, excluding Section 1.3 (5 to 6), 3.12, 3.13 shall apply to design and construction of glass unit masonry, except as stated herein. Section (5.2) shall not apply to construction of glass unit masonry.

10.1.1 **Limitations.** Solid or hollow approved glass block shall not be used in fire walls, party walls, fire barriers or fire partitions, or for load-bearing construction. Such blocks shall be erected with mortar and reinforcement in metal channel-type frames, structural frames, masonry or concrete recesses, embedded panel anchors as provided for both exterior and interior walls or other approved joint materials. Wood strip framing shall not be used in walls required to have a fire-resistance rating by other provisions of SBC.

Exceptions:

1. Glass-block assemblies having a fire protection rating of not less than $3/4$ hour shall be permitted as opening protectives in accordance with Chapter 4 SBC 801 in barriers and fire partitions that have a required fire-resistance rating of 1 hour or less and do not enclose exit stairways or exit passageways.
2. Glass-block assemblies as permitted in Section 2.16.5, SBC 201.

SECTION 10.2
UNITS

10.2.0 Hollow or solid glass-block units shall be standard or thin units.

10.2.1 **Standard units.** The specified thickness of standard units shall be 100 mm.

10.2.2 **Thin units.** The specified thickness of thin units shall be 80 mm for hollow units or 75 mm for solid units.

SECTION 10.3
PANEL SIZE

10.3.1 **Exterior standard-unit panels.** The maximum area of each individual exterior standard-unit panel shall be 13 m^2 when the design wind pressure is 960 N/m^2. The maximum panel dimension between structural supports shall be 7500 mm in width or 6000 mm in height. The panel areas are permitted to be adjusted in accordance with Figure 10.3.1 for other wind pressures.

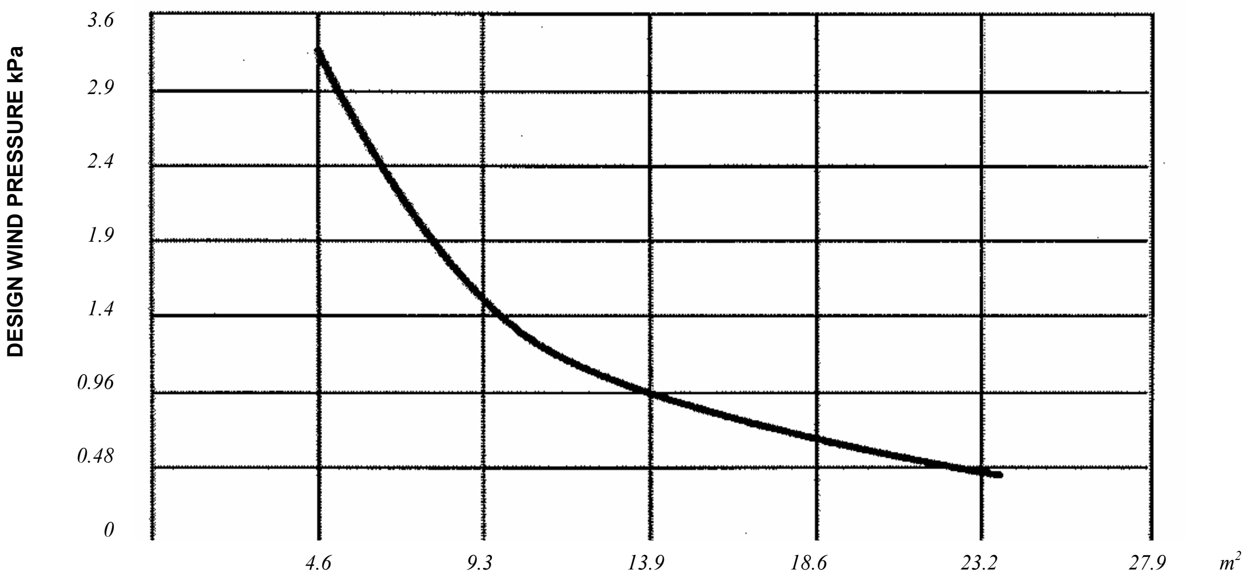

Figure 10.3.1
Glass Masonry Design Wind Load Resistance

10.3.2 **Exterior thin-unit panels.** The maximum area of each individual exterior thin-unit panel shall be 8 m^2. The maximum dimension between structural supports shall be 4500 mm in width or 3000 mm in eight. Thin units shall not be used in applications where the design wind pressure exceeds 950 N/m^2.

10.3.3 **Interior panels.** The maximum area of each individual standard-unit panel shall be 23 m^2. The maximum area of each thin-unit panel shall be 14 m^2. The maximum dimension between structural supports shall be 7600 mm in width or 6000 mm in height.

10.3.4 **Solid units.** The maximum area of solid glass-block wall panels in both exterior and interior walls shall not be more than 9 m^2.

10.3.5 **Curved panels.** The width of curved panels shall conform to the requirements of Section 10.3.1, 10.3.2 and 10.3.3, except additional structural supports shall be provided at locations where a curved section joins a straight section, and at inflection points in multi-curved walls.

SECTION 10.4
SUPPORT

10.4.1 **Isolation.** Glass unit masonry panels shall be isolated so that in-plane loads are not imparted to the panel.

10.4.2 **Vertical.** Maximum total deflection of structural members supporting glass unit masonry shall not exceed $l/600$.

10.4.3 **Lateral.** Glass unit masonry panels more than one unit wide or one unit high shall be laterally supported along their tops and sides. Lateral support shall be provided by panel anchors along the top and sides spaced not more than 400 mm o.c. or by channel-type restraints. Glass unit masonry panels shall be recessed at least 25 mm within channels and chases. Channel-type restraints shall be oversized to accommodate expansion material in the opening and packing and sealant between the framing restraints and the glass unit masonry perimeter units. Lateral supports for glass unit masonry panels shall be designed to resist applied loads, or a minimum of 3000 N/m (of panel), whichever is greater.

Exceptions:

1. Lateral support at the top of glass unit masonry panels that are no more than one unit wide shall not be required.
2. Lateral support at the sides of glass unit masonry panels that are no more than one unit high shall not be required.

10.4.3.1 **Single unit panels.** Single unit glass unit masonry panels shall conform to the requirements of Section 10.4.3, except lateral support shall not be provided by panel anchors.

SECTION 10.5
EXPANSION JOINTS

10.5.0 Glass unit masonry panels shall be provided with expansion joints along the top and sides at all structural supports. Expansion joints shall have sufficient thickness to accommodate displacements of the supporting structure, but shall not be less than 10 mm in thickness. Expansion joints shall be entirely free of mortar or other debris and shall be filled with resilient material. The sills of glass-block panels shall be coated with approved water-based asphaltic emulsion, or other elastic waterproofing material, prior to laying the first mortar course.

SECTION 10.6
MORTAR

10.6.0 Mortar for glass unit masonry shall comply with Section 3.7.

SECTION 10.7
REINFORCEMENT

10.7.0 Glass unit masonry panels shall have horizontal joint reinforcement spaced not more than 400 mm on center, located in the mortar bed joint, and extending the entire length of the panel but not across expansion joints. Longitudinal wires shall be lapped a minimum of 150 mm at splices. Joint reinforcement shall be placed in the bed joint immediately below and above openings in the panel. The reinforcement shall have not less than two parallel longitudinal wires of size (WD 4), and have welded cross wires of size (WD 4).

CHAPTER 11
MASONRY FIREPLACES

SECTION 11.1
DEFINITION

11.1.0 A masonry fireplace is a fireplace constructed of masonry. Masonry fireplaces shall be constructed in accordance with this section, Table 11.1.1 and Figure 11.1.1.

SECTION 11.2
FOOTINGS AND FOUNDATIONS

11.2.0 Footings for masonry fireplaces and their chimneys shall be constructed of concrete or solid masonry at least 300 mm thick and shall extend at least 150 mm beyond the face of the fireplace or foundation wall on all sides. Footings shall be founded on natural undisturbed earth or engineered fill. Footings shall be at least 300 mm below finished grade.

11.2.1 **Ash dump cleanout.** Cleanout openings, located within foundation walls below fireboxes, when provided, shall be equipped with ferrous metal or masonry doors and frames constructed to remain tightly closed, except when in use. Cleanouts shall be accessible and located so that ash removal will not create a hazard to combustible materials.

SECTION 11.3
SEISMIC REINFORCING

11.3.0 Masonry fireplaces shall be constructed, anchored, supported and reinforced as required in this chapter. In Seismic Design Category A, B or C, reinforcement and seismic anchorage is not required. In Seismic Design Category D, masonry and concrete fireplaces shall be reinforced and anchored as detailed in Section 11.3.1, 11.3.2, 11.4 and 11.4.1 for chimneys serving fireplaces.

11.3.1 **Vertical reinforcing.** For fireplaces with chimneys up to 1000 mm wide, four Dia 12 mm continuous vertical bars, anchored in the foundation, shall be placed in the concrete, between wythes of solid masonry or within the cells of hollow unit masonry and grouted in accordance with Section 3.10. For fireplaces with chimneys greater than 1000 mm wide, two additional Dia 12 mm vertical bars shall be provided for each additional 1000 mm in width or fraction thereof.

11.3.2 **Horizontal reinforcing.** Vertical reinforcement shall be placed enclosed within 6 mm ties or other reinforcing of equivalent net cross-sectional area, placed in the bed joints of unit masonry at a minimum of every 450 mm of vertical height. Two such ties shall be provided at each bend in the vertical bars.

SECTION 11.4
SEISMIC ANCHORAGE

11.4.0 Masonry and concrete chimneys in Seismic Design Category D shall be anchored at each floor, ceiling or roof line more than 1800 mm above grade, except where constructed completely within the exterior walls. Anchorage shall conform to the following requirements.

11.4.1 **Anchorage.** Two 5 mm by 25 mm straps shall be embedded a minimum of 300 mm into the chimney. Straps shall be hooked around the outer bars and extend 150 mm beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two 13 mm bolts.

SECTION 11.5
FIREBOX WALLS

11.5.0 Masonry fireboxes shall be constructed of solid masonry units, hollow masonry units grouted solid, stone or concrete. When a lining of firebrick at least 50 mm in thickness or other approved lining is provided, the minimum thickness of back and sidewalls shall each be 200 mm of solid masonry, including the lining. The width of joints between firebricks shall not be greater than 6.0 mm. When no lining is provided, the total minimum thickness of back and sidewalls shall be 250 mm of solid masonry. Firebrick shall conform to ASTM C 27 or ASTM C 1261 and shall be laid with medium-duty refractory mortar conforming to ASTM C 199.

11.5.1 **Steel fireplace units.** Steel fireplace units are permitted to be installed with solid masonry to form a masonry fireplace provided they are installed according to either the requirements of their listing or the requirements of this section. Steel fireplace units incorporating a steel firebox lining shall be constructed with steel not less than 6.0 mm in thickness, and an air-circulating chamber which is ducted to the interior of the building. The firebox lining shall be encased with solid masonry to provide a total thickness at the back and sides of not less than 200 mm, of which not less than 100 mm shall be of solid masonry or concrete. Circulating air ducts employed with steel fireplace units shall be constructed of metal or masonry.

SECTION 11.6
FIREBOX DIMENSIONS

11.6.0 The firebox of a concrete or masonry fireplace shall have a minimum depth of 500 mm. The throat shall not be less than 200 mm above the fireplace opening. The throat opening shall not be less than 100 mm in depth. The cross-sectional area of the passageway above the firebox, including the throat, damper and smoke chamber, shall not be less than the cross-sectional area of the flue.

Exception: Rumsford fireplaces shall be permitted provided that the depth of the fireplace is at least 300 mm and at least one-third of the width of the fireplace opening, and the throat is at least 300 mm above the lintel, and at least $1/20$ of the cross-sectional area of the fireplace opening.

TABLE 11.1.1
SUMMARY OF REQUIREMENTS FOR MASONRY FIREPLACES AND CHIMNEYS[a]

Item	Letter	Requirements	Section
Hearth and hearth extension thickness	A	100 mm minimum thickness for hearth, 50 mm minimum thickness for hearth extension.	11.9
Hearth extension (each side of opening)	B	200 mm for fireplace opening less than 0.6 m^2. 300 mm for fireplace opening greater than or equal to 0.6 m^2.	11.10
Hearth extension (front of opening)	C	400 mm for fireplace opening less than 0.6 m^2. 500 mm for fireplace opening greater than or equal to 0.6 m^2.	11.10
Firebox dimensions	–	500 mm minimum firebox depth. 300 mm minimum firebox depth for Rumford fireplaces.	11.6
Hearth and hearth extension reinforcing	D	Reinforced to carry its own weight and all imposed loads.	11.9
Thickness of wall of firebox	E	250 mm solid masonry or 200 mm where firebrick lining is used.	11.5
Distance from top of opening to throat	F	200 mm minimum.	11.7 11.7.1
Smoke chamber wall thickness dimensions	G	150 mm lined; 200 mm unlined. Not taller than opening width; walls not inclined more than 0.76 rad from vertical for prefabricated smoke chamber linings or 0.5 rad from vertical for corbeled masonry.	11.8
Chimney vertical reinforcing	H	Four Dia 12 mm full-length bars for chimney up to1000 mm wide. Add two Dia 12 mm bars for each additional 1000 mm or fraction of width, or for each additional flue.	11.3.1, 13.3.1
Chimney horizontal reinforcing	J	6 mm ties at each 450 mm, and two ties at each bend in vertical steel.	11.3.2 13.3.2
Fireplace lintel	L	Noncombustible material with 100 mm bearing length of each side of opening.	11.7
Chimney walls with flue lining	M	100 mm thick solid masonry with 16 mm fireclay liner or equivalent. 13 mm grout or airspace between fireclay liner and wall.	13.11.1
Effective flue area (based on area of fireplace opening and chimney)	P	See (13.16).	13.16
Clearances From chimney From fireplace From combustible trim or materials Above roof	R	 50 mm interior, 25 mm exterior or 300 mm from lining. 50 mm back or sides or 300 mm from lining. 150 mm from opening 900 mm above roof penetration, 600 mm above part of structure within 3000 mm.	 13.19 11.11 11.12 13.9
Anchorage strap Number required Embedment into chimney Fasten to Number of bolts	S	5 mm by 25 mm Two 300 mm hooked around outer bar with 150 mm extension. 4 joists Two 13 mm diameter.	 11.4 13.4.1
Footing Thickness Width	T	 300 mm minimum. 150 mm each side of fireplace wall.	 11.2

a. This table provides a summary of major requirements for the construction of masonry chimneys and fireplaces. Letter references are to Figure 11.1.1, which shows examples of typical construction. This table does not cover all requirements, nor does it cover all aspects of the indicated requirements. For the actual mandatory requirements of the code, see the indicated section of text.

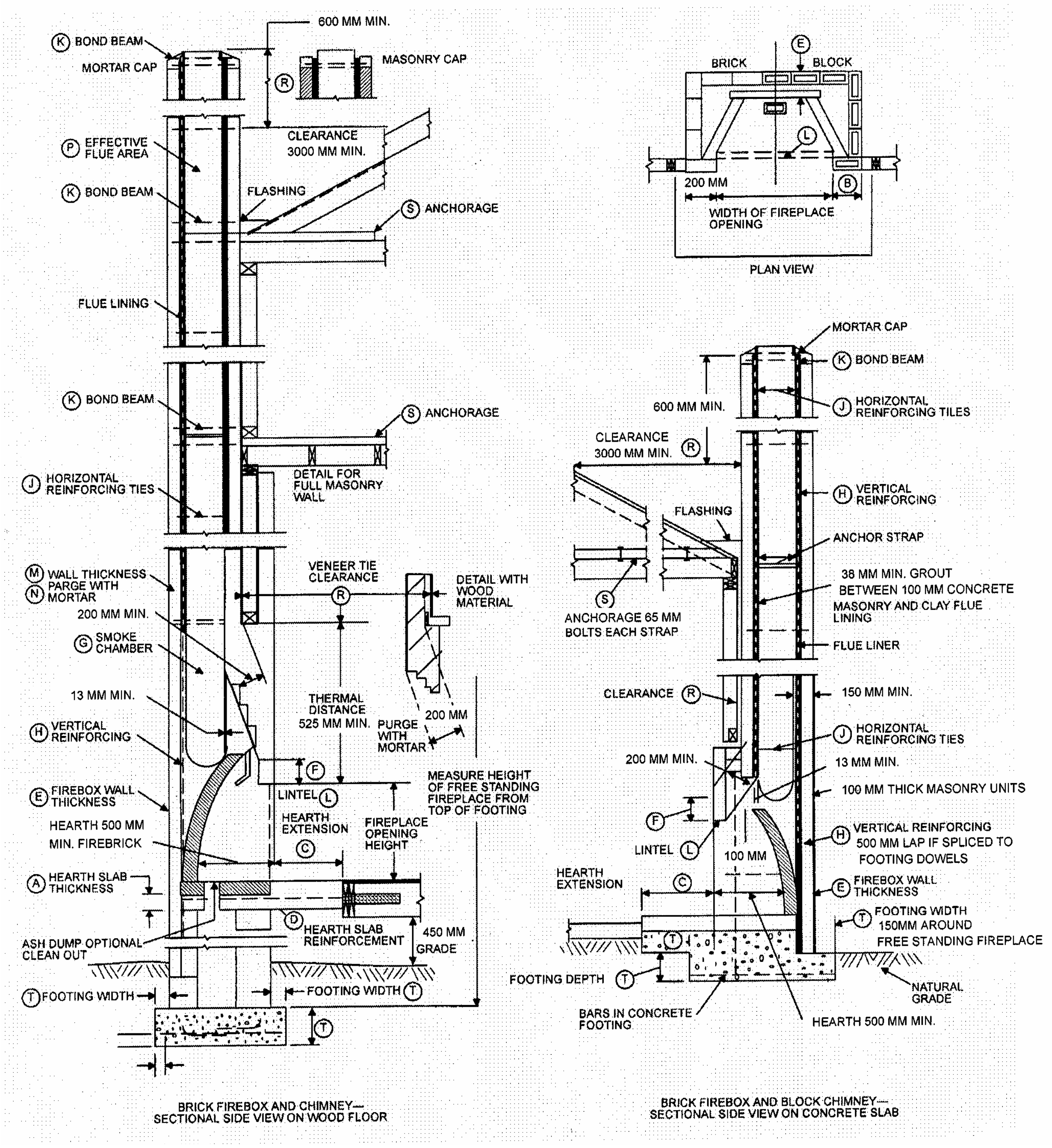

FIGURE 11.1.1

FIREPLACE AND CHIMNEY DETAILS

SECTION 11.7
LINTEL AND THROAT

11.7.0 Masonry over a fireplace opening shall be supported by a lintel of noncombustible material. The minimum required bearing length on each end of the fireplace opening shall be 100 mm. The fireplace throat or damper shall be located a minimum of 200 mm above the top of the fireplace opening.

11.7.1 **Damper.** Masonry fireplaces shall be equipped with a ferrous metal damper located at least 200 mm above the top of the fireplace opening. Dampers shall be installed in the fireplace or at the top of the flue venting the fireplace, and shall be operable from the room containing the fireplace. Damper controls shall be permitted to be located in the fireplace.

SECTION 11.8
SMOKE CHAMBER WALLS

11.8.0 Smoke chamber walls shall be constructed of solid masonry units, hollow masonry units grouted solid, stone or concrete. Corbeling of masonry units shall not leave unit cores exposed to the inside of the smoke chamber. The inside surface of corbeled masonry shall be parged smooth. Where no lining is provided, the total minimum thickness of front, back and sidewalls shall be 200 mm of solid masonry. When a lining of firebrick at least 50 mm thick, or a lining of vitrified clay at least 16 mm thick, is provided, the total minimum thickness of front, back and sidewalls shall be 150 mm of solid masonry, including the lining. Firebrick shall conform to ASTM C 27 or ASTM C 1261 and shall be laid with refractory mortar conforming to ASTM C 199.

11.8.1 **Smoke chamber dimensions.** The inside height of the smoke chamber from the fireplace throat to the beginning of the flue shall not be greater than the inside width of the fireplace opening. The inside surface of the smoke chamber shall not be inclined more than 0.76 rad from vertical when prefabricated smoke chamber linings are used or when the smoke chamber walls are rolled or sloped rather than corbeled. When the inside surface of the smoke chamber is formed by corbeled masonry, the walls shall not be corbeled more than 0.5 rad from vertical.

SECTION 11.9
HEARTH AND HEARTH EXTENSION

11.9.0 Masonry fireplace hearths and hearth extensions shall be constructed of concrete or masonry, supported by noncombustible materials, and reinforced to carry their own weight and all imposed loads. No combustible material shall remain against the underside of hearths or hearth extensions after construction.

11.9.1 **Hearth thickness.** The minimum thickness of fireplace hearths shall be 100 mm.

11.9.2 **Hearth extension thickness.** The minimum thickness of hearth extensions shall be 50 mm.
Exception: When the bottom of the firebox opening is raised at least 200 mm above the top of the hearth extension, a hearth extension of not less than 10 mm brick, concrete, stone, tile or other approved noncombustible material is permitted.

SECTION 11.10
HEARTH EXTENSION DIMENSIONS

11.10.0 Hearth extensions shall extend at least 400 mm in front of, and at least 200 mm beyond, each side of the fireplace opening Where the fireplace opening is 0.6 m^2 or larger, the hearth extension shall extend at least 500 mm in front of, and at least 300 mm beyond, each side of the fireplace opening.

SECTION 11.11
FIREPLACE CLEARANCE

11.11.0 Any portion of a masonry fireplace located in the interior of a building or within the exterior wall of a building shall have a clearance to combustibles of not less than 50 mm from the front faces and sides of masonry fireplaces and not less than 100 mm from the back faces of masonry fireplaces. The airspace shall not be filled, except to provide fireblocking in accordance with Section 11.13.

Exceptions:

1. Masonry fireplaces listed and labeled for use in contact with combustibles in accordance with UL 127, and installed in accordance with the manufacturer's installation instructions, are permitted to have combustible material in contact with their exterior surfaces.
2. When masonry fireplaces are constructed as part of masonry or concrete walls, combustible materials shall not be in contact with the masonry or concrete walls less than 300 mm from the inside surface of the nearest firebox lining.
3. Exposed combustible trim and the edges of sheathing materials, such as wood siding, flooring and drywall, are permitted to abut the masonry fireplace sidewalls and hearth extension, in accordance with Figure 11.11.1, provided such combustible trim or sheathing is a minimum of 300 mm from the inside surface of the nearest firebox lining.
4. Exposed combustible mantels or trim is permitted to be placed directly on the masonry fireplace front surrounding the fireplace opening provided such combustible materials shall not be placed within 150 mm of a fireplace opening. Combustible material within 300 mm of the fireplace opening shall not project more than 3 mm for each 25 mm distance from such opening.

SECTION 11.12
MANTEL AND TRIM

11.12.0 Woodwork or other combustible materials shall not be placed within 150 mm of a fireplace opening. Combustible material within 300 mm of the fireplace opening shall not project more than 3 mm for each 25 mm distance from such opening.

SECTION 11.13
FIREPLACE FIREBLOCKING

11.13.0 All spaces between fireplaces and floors and ceilings through which fireplaces pass shall be fireblocked with noncombustible material securely fastened in place. The fireblocking of spaces between wood joists, beams or headers shall be to a depth of 25 mm and shall only be placed on strips of metal or metal lath laid across the spaces between combustible material and the chimney.

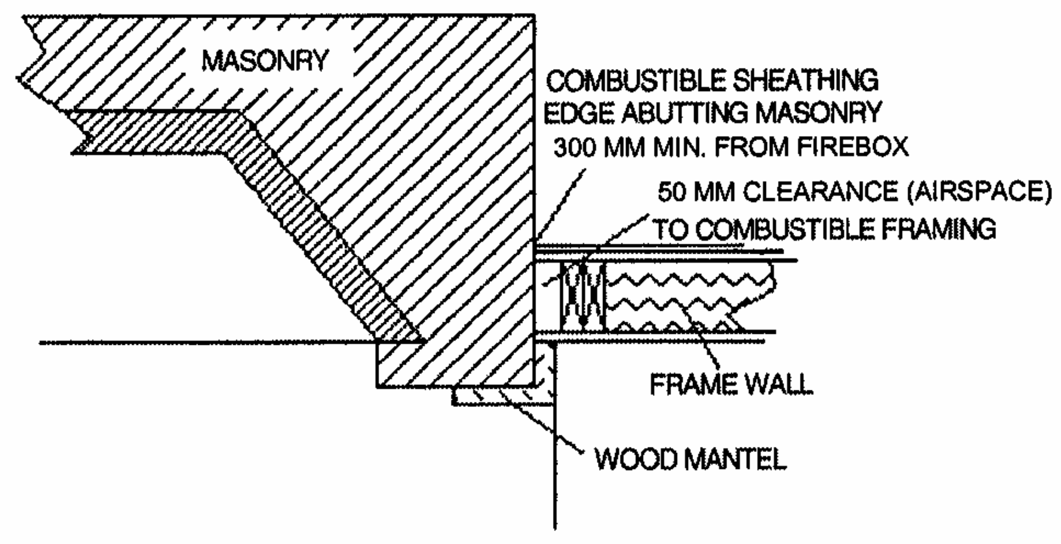

FIGURE 11.11.1
ILLUSTRATION OF EXCEPTION TO FIREPLACE CLEARANCE PROVISION

SECTION 11.14
EXTERIOR AIR

11.14.0 Factory-built or masonry fireplaces covered in this section shall be equipped with an exterior air supply to ensure proper fuel combustion unless the room is mechanically ventilated and controlled so that the indoor pressure is neutral or positive.

11.14.1 **Factory-built fireplaces.** Exterior combustion air ducts for factory-built fireplaces shall be listed components of the fireplace, and installed according to the fireplace manufacturer's instructions.

11.14.2 **Masonry fireplaces.** Listed combustion air ducts for masonry fireplaces shall be installed according to the terms of their listing and manufacturer's instructions.

11.14.3 **Exterior air intake.** The exterior air intake shall be capable of providing all combustion air from the exterior of the dwelling. The exterior air intake shall not be located within the garage, attic, basement or crawl space of the dwelling nor shall the air intake be located at an elevation higher than the firebox. The exterior air intake shall be covered with a corrosion-resistant screen of 6 mm mesh.

11.14.4 **Clearance.** Unlisted combustion air ducts shall be installed with a minimum 25 mm clearance to combustibles for all parts of the duct within 1500 mm of the duct outlet.

11.14.5 **Passageway.** The combustion air passageway shall be a minimum of 4000 mm^2 and not more than 0.04 m^2, except that combustion air systems for listed fireplaces or for fireplaces tested for emissions shall be constructed according to the fireplace manufacturer's instructions.

11.14.6 **Outlet.** The exterior air outlet is permitted to be located in the back or sides of the firebox chamber or within 600 mm of the firebox opening on or near the floor. The outlet shall be closable and designed to prevent burning material from dropping into concealed combustible spaces.

CHAPTER 12
MASONRY HEATERS

SECTION 12.1
DEFINITION

12.1.1 A masonry heater is a heating appliance constructed of concrete or solid masonry, hereinafter referred to as “masonry heater,” having a mass of at least 800 kg, excluding the chimney and foundation, which is designed to absorb and store heat from a solid fuel fire built in the firebox by routing the exhaust gases through internal heat exchange channels in which the flow path downstream of the firebox includes at least one 3.14 rad change in flow direction before entering the chimney, and that delivers heat by radiation from the masonry surface of the heater that shall not exceed 110° C except within 203 mm surrounding the fuel loading door(s).

SECTION 12.2
INSTALLATION

12.2.1 Masonry heaters shall be listed or installed in accordance with ASTM E 1602.

SECTION 12.3
SEISMIC REINFORCING

12.3.1 Seismic reinforcing shall not be required within the body of a masonry heater whose height is equal to or less than 2.5 times its body width and where the masonry chimney serving the heater is not supported by the body of the heater. Where the masonry chimney shares a common wall with the facing of the masonry heater, the chimney portion of the structure shall be reinforced in accordance with Section 13.3 and 13.4.

SECTION 12.4
MASONRY HEATER CLEARANCE

12.4.1 Wood or other combustible framing shall not be placed within 100 mm of the outside surface of a masonry heater, provided the wall thickness of the firebox is not less than 200 mm and the wall thickness of the heat exchange channels is not less than 120 mm. A clearance of at least 200 mm shall be provided between the gas-tight capping slab of the heater and a combustible ceiling. The required space between the heater and combustible material shall be fully vented to permit the free flow of air around all heater surfaces.

CHAPTER 13
MASONRY CHIMNEYS

SECTION 13.1
GENERAL

13.1.0 A masonry chimney is a chimney constructed of masonry, hereinafter referred to as "masonry chimney." Masonry chimneys shall be constructed, anchored, supported and reinforced as required in this section.

13.1.1 **Design requirements.** Masonry chimneys shall be designed as load bearing in accordance with requirements of Chapter 1 through Chapter 8.

SECTION 13.2
FOOTINGS AND FOUNDATIONS

13.2.0 Foundations for masonry chimneys shall be constructed of concrete or solid masonry at least 300 mm thick and shall extend at least 150 mm beyond the face of the foundation or support wall on all sides. Footings shall be founded on natural undisturbed earth or engineered fill. In areas not subjected to freezing, footings shall be at least 300 mm below finished grade.

13.2.1 **Design requirements.** Footings and foundations shall be designed in accordance with SBC 303.

SECTION 13.3
SEISMIC REINFORCING

13.3.0 Masonry chimneys shall be constructed, anchored, supported and reinforced as required in this chapter. In Seismic Design Category A, B or C, reinforcement and seismic anchorage is not required. In Seismic Design Category D, masonry chimneys shall be reinforced and anchored as detailed in Section 13.3.1 and 13.3.2.

13.3.1 **Vertical reinforcing.** For chimneys up to 1000 mm wide, four Dia 12 mm continuous vertical bars anchored in the foundation shall be placed, between wythes of solid masonry or within the cells of hollow unit masonry and grouted in accordance with Section 3.10. Grout shall be prevented from bonding with the flue liner so that the flue liner is free to move with thermal expansion. For chimneys greater than 1000 mm wide, two additional Dia 12 mm vertical bars shall be provided for each additional 1000 mm in width or fraction thereof.

13.3.2 **Horizontal reinforcing.** Vertical reinforcement shall be placed enclosed within 6 mm ties, or other reinforcing of equivalent net cross-sectional area, spaced not to exceed 450 mm o.c. in concrete, or placed in the bed joints of unit masonry, at a minimum of every 450 mm of vertical height. Two such ties shall be provided at each bend in the vertical bars.

SECTION 13.4
SEISMIC ANCHORAGE

13.4.0 Masonry and concrete chimneys and foundations in Seismic Design Category D shall be anchored at each floor, ceiling or roof line more than 1800 mm above grade, except where constructed completely within the exterior walls. Anchorage shall conform to the following requirements.

13.4.1 **Anchorage.** Two 5 mm by 25 mm straps shall be embedded a minimum of 300 mm into the chimney. Straps shall be hooked around the outer bars and extend 150 mm beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two 13 mm bolts.

SECTION 13.5
CORBELING

13.5.0 Masonry chimneys shall not be corbeled more than half of the chimney's wall thickness from a wall or foundation, nor shall a chimney be corbeled from a wall or foundation that is less than 300 mm in thickness unless it projects equally on each side of the wall, except that on the second story of a two-story dwelling, corbeling of chimneys on the exterior of the enclosing walls is permitted to equal the wall thickness. The projection of a single course shall not exceed one-half the unit height or one-third of the unit bed depth, whichever is less.

SECTION 13.6
CHANGES IN DIMENSION

13.6.0 The chimney wall or chimney flue lining shall not change in size or shape within 150 mm above or below where the chimney passes through floor components, ceiling components or roof components.

SECTION 13.7
OFFSETS

13.7.0 Where a masonry chimney is constructed with a fireclay flue liner surrounded by one wythe of masonry, the maximum offset shall be such that the centerline of the flue above the offset does not extend beyond the center of the chimney wall below the offset. Where the chimney offset is supported by masonry below the offset in an approved manner, the maximum offset limitations shall not apply. Each individual corbeled masonry course of the offset shall not exceed the projection limitations specified in Section 13.5.

SECTION 13.8
ADDITIONAL LOAD

13.8.0 Chimneys shall not support loads other than their own weight unless they are designed and constructed to support the additional load. Masonry chimneys are permitted to be constructed as part of the masonry walls of the building.

SECTION 13.9
TERMINATION

13.9.0 Chimneys shall extend at least 600 mm higher than any portion of the building within 3000 mm, but shall not be less than 900 mm above the highest point where the chimney passes through the roof.

13.9.1 **Spark arrestors.** Where a spark arrestor is installed on a masonry chimney, the spark arrestor shall meet all of the following requirements:

1. The net free area of the arrestor shall not be less than four times the net free area of the outlet of the chimney flue it serves.
2. The arrestor screen shall have heat and corrosion resistance equivalent to 19-gage galvanized steel or 24-gage stainless steel.
3. Openings shall not permit the passage of spheres having a diameter greater than 13 mm nor block the passage of spheres having a diameter less than 11 mm.
4. The spark arrestor shall be accessible for cleaning and the screen or chimney cap shall be removable to allow for cleaning of the chimney flue.

SECTION 13.10
WALL THICKNESS

13.10.0 Masonry chimney walls shall be constructed of concrete, solid masonry units or hollow masonry units grouted solid with not less than 100 mm nominal thickness.

SECTION 13.11
FLUE LINING (MATERIAL)

13.11.0 Masonry chimneys shall be lined. The lining material shall be appropriate for the type of appliance connected, according to the terms of the appliance listing and the manufacturer's instructions.

13.11.1 **Residential-type appliances (general).** Flue lining systems shall comply with one of the following:

1. Clay flue lining complying with the requirements of ASTM C 315, or equivalent.
2. Listed chimney lining systems complying with UL 1777.
3. Factory-built chimneys or chimney units listed for installation within masonry chimneys.
4. Other approved materials that will resist corrosion, erosion, softening or cracking from flue gases and condensate at temperatures up to 982 °C.

13.11.1.1 **Flue linings for specific appliances.** Flue linings other than those covered in Section 13.11.1 intended for use with specific appliances shall comply with Section 13.11.1.2 through 13.11.1.4, 13.11.2, 13.11.3.

13.11.1.2 **Gas appliances.** Flue lining systems for gas appliances shall be installed in an approved method.

13.11.1.3 **Pellet fuel-burning appliances.** Flue lining and vent systems for use in masonry chimneys with pellet fuel-burning appliances shall be limited to flue lining systems complying with Section 13.11.1 and pellet vents listed for installation within masonry chimneys (see 13.11.1.5 for marking).

13.11.1.4 **Oil-fired appliances approved for use with L-vent.** Flue lining and vent systems for use in masonry chimneys with oil-fired appliances approved for use with Type

L-vent shall be limited to flue lining systems complying with Section 13.11.1 and listed chimney liners complying with UL 641 see Section 13.11.1.5 for marking).

13.11.1.5 **Notice of usage.** When a flue is relined with a material not complying with Section 13.11.1, the chimney shall be plainly and permanently identified by a label attached to a wall, ceiling or other conspicuous location adjacent to where the connector enters the chimney. The label shall include the following message or equivalent language: “This chimney is for use only with (type or category of appliance) that burns (type of fuel). Do not connect other types of appliances.”

13.11.2 Concrete and masonry chimneys for medium-heat appliances.

13.11.2.1 **General.** Concrete and masonry chimneys for medium-heat appliances shall comply with Section 13.1 through 13.5.

13.11.2.2 **Construction.** Chimneys for medium-heat appliances shall be constructed of solid masonry units or of concrete with walls a minimum of 200 mm thick, or with stone masonry a minimum of 300 mm thick.

13.11.2.3 **Lining.** Concrete and masonry chimneys shall be lined with an approved medium-duty refractory brick a minimum of 110 mm thick laid on the 110 mm in an approved medium-duty refractory mortar. The lining shall start 600 mm or more below the lowest chimney connector entrance. Chimneys terminating 7500 mm or less above a chimney connector entrance shall be lined to the top.

13.11.2.4 **Multiple passageway.** Concrete and masonry chimneys containing more than one passageway shall have the liners separated by a minimum 100 mm concrete or solid masonry wall.

13.11.2.5 **Termination height.** Concrete and masonry chimneys for medium-heat appliances shall extend a minimum of 3000 mm higher than any portion of any building within 7500 mm.

13.11.2.6 **Clearance.** A minimum clearance of 100 mm shall be provided between the exterior surfaces of a concrete or masonry chimney for medium-heat appliances and combustible material.

13.11.3 Concrete and masonry chimneys for high-heat appliances.

13.11.3.1 **General.** Concrete and masonry chimneys for high-heat appliances shall comply with Section 13.1 through 13.5.

13.11.3.2 **Construction.** Chimneys for high-heat appliances shall be constructed with double walls of solid masonry units or of concrete, each wall to be a minimum of 200 mm thick with a minimum airspace of 50 mm between the walls.

13.11.3.3 **Lining.** The inside of the interior wall shall be lined with an approved high-duty refractory brick, a minimum 110 mm thick laid on the 110 mm bed in an approved high-duty refractory mortar. The lining shall start at the base of the chimney and extend continuously to the top.

13.11.3.4 **Termination height.** Concrete and masonry chimneys for high-heat appliances shall extend a minimum of 6000 mm higher than any portion of any building within 15000 mm.

13.11.3.5 **Clearance.** Concrete and masonry chimneys for high-heat appliances shall have approved clearance from buildings and structures to prevent overheating combustible materials, permit inspection and maintenance operations on the chimney and prevent danger of burns to persons.

SECTION 13.12
FLUE LINING (INSTALLATION)

13.12.1 Flue liners shall be installed in accordance with ASTM C 1283 and extend from a point not less than 200 mm below the lowest inlet or, in the case of fireplaces, from the top of the smoke chamber, to a point above the enclosing walls. The lining shall be carried up vertically, with a maximum slope no greater than 0.52 rad (30 degrees) from the vertical.

13.12.2 Fireclay flue liners shall be laid in medium-duty refractory mortar conforming to ASTM C 199, with tight mortar joints left smooth on the inside and installed to maintain an airspace or insulation not to exceed the thickness of the flue liner separating the flue liners from the interior face of the chimney masonry walls. Flue lining shall be supported on all sides. Only enough mortar shall be placed to make the joint and hold the liners in position.

SECTION 13.13
ADDITIONAL REQUIREMENTS

13.13.1 **Listed materials.** Listed materials used as flue linings shall be installed in accordance with the terms of their listings and the manufacturer's instructions.

13.13.2 **Space around lining.** The space surrounding a chimney lining system or vent installed within a masonry chimney shall not be used to vent any other appliance.

Exception: This shall not prevent the installation of a separate flue lining in accordance with the manufacturer's instructions.

SECTION 13.14
MULTIPLE FLUES

13.14.1 When two or more flues are located in the same chimney, masonry wythes shall be built between adjacent flue linings. The masonry wythes shall be at least 100 mm thick and bonded into the walls of the chimney.

Exception: When venting only one appliance, two flues are permitted to adjoin each other in the same chimney with only the flue lining separation between them. The joints of the adjacent flue linings shall be staggered at least 100 mm.

SECTION 13.15
FLUE AREA (APPLIANCE)

13.15.1 Chimney flues shall not be smaller in area than the area of the connector from the appliance. Chimney flues connected to more than one appliance shall not be less than the area of the largest connector plus 50% of the areas of additional chimney connectors.

Exceptions:

1. Chimney flues serving gas and oil-fired appliances sized in accordance with NFPA 31.

SECTION 13.16
FLUE AREA (MASONRY FIREPLACE)

13.16.0 Flue sizing for chimneys serving fireplaces shall be in accordance with Section 13.16.1 or 13.16.2.

13.16.1 **Minimum area.** Round chimney flues shall have a minimum net cross-sectional area of at least $1/12$ of the fireplace opening. Square chimney flues shall have a minimum net cross-sectional area of at least $1/10$ of the fireplace opening. Rectangular chimney flues with an aspect ratio less than 2 to 1 shall have a minimum net cross-sectional area of at least $1/10$ of the fireplace opening. Rectangular chimney flues with an aspect ratio of 2 to 1 or more shall have a minimum net cross-sectional area of at least $1/8$ of the fireplace opening.

13.16.2 **Determination of minimum area.** The minimum net cross-sectional area of the flue shall be determined in accordance with Figure 13.16.1. A flue size providing at least the equivalent net cross-sectional area shall be used. Cross-sectional areas of clay flue linings are as provided in Tables 13.16.1 and 13.16.2 or as provided by the manufacturer or as measured in the field. The height of the chimney shall be measured from the firebox floor to the top of the chimney flue.

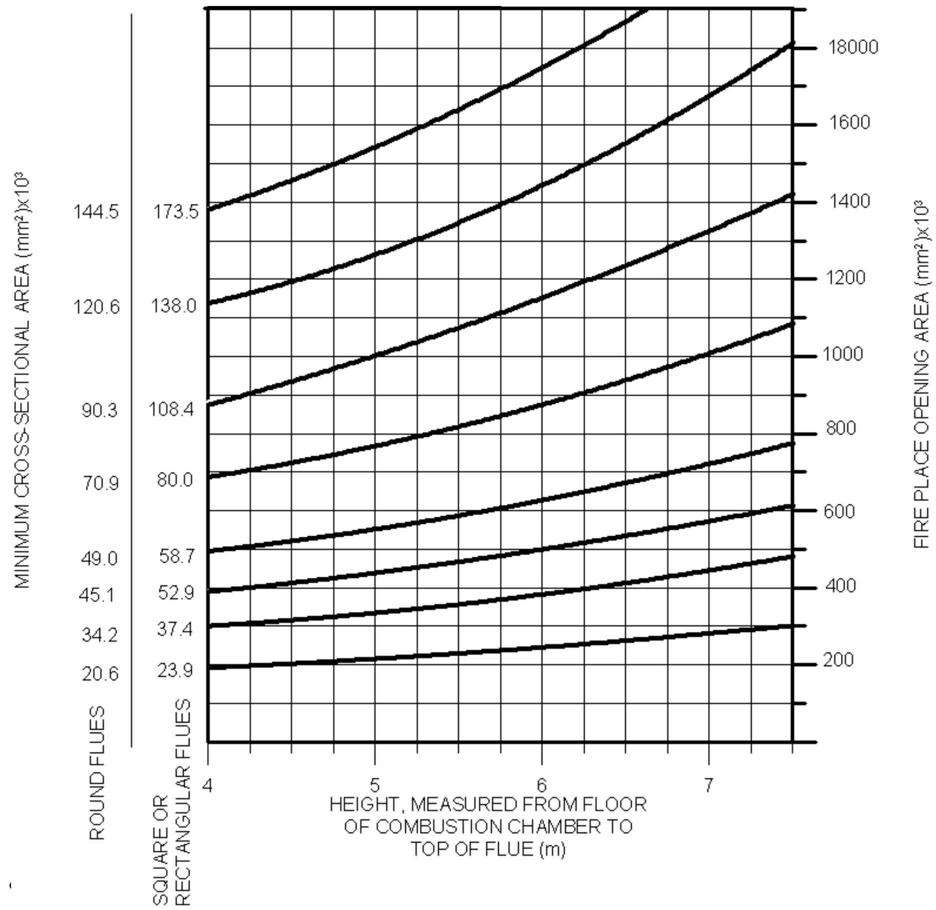

FIGURE 13.16.1
FLUE SIZES FOR MASONRY CHIMNEYS

TABLE 13.16.1
NET CROSS-SECTIONAL AREA OF ROUND FLUE SIZES[a]

FLUE SIZE, INSIDE DIAMETER (mm)	CROSS-SECTIONAL AREA (square mm)
150	17670
175	24050
200	31415
250	49090
269	56830
300	70685
375	110445
450	159045

a. Flue sizes are based on ASTM C 315.

TABLE 13.16.2
NET CROSS-SECTIONAL AREA OF SQUARE AND RECTANGULAR FLUE SIZES[a]

FLUE SIZE (mm)	CROSS-SECTIONAL AREA (square mm)
110 x 325	21250
190 x 190	23125
210 x 210	29375
190 x 290	36250
210 x 325	46250
190 x 390	51250
290 x 290	56875
210 x 440	63125
325 x 325	76250
290 x 390	77500
325 x 440	103125
390 x 390	105000
390 x 490	133750
440 x 440	141250
490 x 490	168125
500 x 500	178750

a. Flue sizes are based on ASTM C 315.

SECTION 13.17
INLET

13.17.1 Inlets to masonry chimneys shall enter from the side. Inlets shall have a thimble of fireclay, rigid refractory material or metal that will prevent the connector from pulling out of the inlet or from extending beyond the wall of the liner.

SECTION 13.18
MASONRY CHIMNEY CLEANOUT OPENINGS

13.18.1 Cleanout openings shall be provided within 150 mm of the base of each flue within every masonry chimney. The upper edge of the cleanout shall be located at least 150 mm below the lowest chimney inlet opening. The height of the opening shall be at least 150 mm. The cleanout shall be provided with a noncombustible cover.

Exception: Chimney flues serving masonry fireplaces, where cleaning is possible through the fireplace opening.

SECTION 13.19
CHIMNEY CLEARANCES

13.19.1 Any portion of a masonry chimney located in the interior of the building or within the exterior wall of the building shall have a minimum airspace clearance to combustibles of 50 mm. Chimneys minimum airspace clearance of 25 mm. The airspace shall not be filled, except to provide fireblocking in accordance with Section 13.20.

Exceptions:

1. Masonry chimneys equipped with a chimney lining system listed and labeled for use in chimneys in contact with combustibles in accordance with UL 1777, and installed in accordance with the manufacturer's instructions, are permitted to have combustible material in contact with their exterior surfaces.
2. Where masonry chimneys are constructed as part of masonry or concrete walls, combustible materials shall not be in contact with the masonry or concrete wall less than 300 mm from the inside surface of the nearest flue lining.
3. Exposed combustible trim and the edges of sheathing materials, such as wood siding, are permitted to abut the masonry chimney sidewalls, in accordance with Figure 13.19.1, provided such combustible trim or sheathing is a minimum of 300 mm from the inside surface of the nearest flue lining. Combustible material and trim shall not overlap the corners of the chimney by more than 25 mm.

SECTION 13.20
CHIMNEY FIREBLOCKING

13.20.1 All spaces between chimneys and floors and ceilings through which chimneys pass shall be fireblocked with noncombustible material securely fastened in place. The fireblocking of spaces between wood joists, beams or headers shall be to a depth of 25 mm and shall only be placed on strips of metal or metal lath laid across the spaces between combustible material and the chimney.

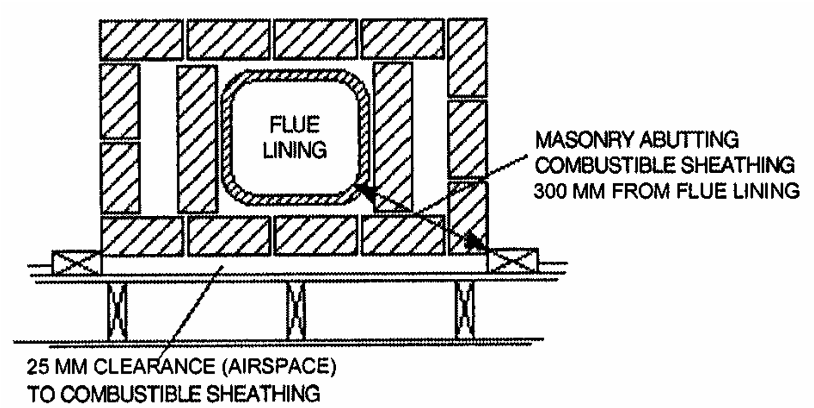

FIGURE 13.19.1
ILLUSTRATION OF EXCEPTION TO CHIMNEY CLEARANCE PROVISION

CHAPTER 14
MASONRY VENEER

SECTION 14.1
GENERAL

14.1.1 **Scope.** This section provides requirements for design and detailing of anchored masonry veneer and adhered masonry veneer.

14.1.1.1 The provisions of Chapter 1 to Chapter 5, excluding Section 1.3(item 6), 1.4, 3.13, 4.13 and 5.2 shall apply to design of anchored and adhered veneer.

14.1.1.2 Section 4.1.2 and 4.1.9 shall not apply to adhered veneer.

14.1.2 **Design of anchored veneer.** Anchored veneer shall meet the requirements of Section 14.1.5 and shall be designed rationally by Section 14.2.1 or detailed by the prescriptive requirements of Section 14.2.2.

14.1.3 **Design of adhered veneer.** Adhered veneer shall meet the requirements of Section 14.1.5 and shall be designed rationally by Section 14.3.1 or detailed by the prescriptive requirements of Section 14.3.2.

14.1.4 **Dimension stone.** Dimension stone veneer is not covered under this Code. Any such system shall be considered a Special System and submitted to the Building Official in accordance with Section 1.3.5.

14.1.5 **General design requirements**

14.1.5.1 Design and detail the backing system of exterior veneer to resist water penetration. Exterior sheathing shall be covered with a water-resistant membrane unless the sheathing is water resistant and the joints are sealed.

14.1.5.2 Design and detail flashing and weep holes in exterior veneer wall systems to resist water penetration into the building interior. Weepholes shall be at least 5 mm in diameter and spaced less than 800 mm on center.

14.1.5.3 Design and detail the veneer to accommodate differential movement.

SECTION 14.2
ANCHORED VENEER

14.2.1 **Alternative design of anchored masonry veneer.** The alternative design of anchored veneer, which is permitted under Section1.3.5, shall satisfy the following conditions:

- **(a)** Loads shall be distributed through the veneer to the anchors and the backing using principles of mechanics.
- **(b)** Out-of-plane deflection of the backing shall be limited to maintain veneer stability.
- **(c)** All masonry, other than veneer, shall meet the provisions of Section 1.2, excluding Section 1.2.5 and 1.2.6.
- **(d)** The veneer is not subject to the flexural tensile stress provisions of Section 7.2.

14.2.2 Prescriptive requirements for anchored masonry veneer

14.2.2.1 Prescriptive requirements for anchored masonry veneer shall not be used in areas where the basic wind speed exceeds 145 km/h as given in SBC 301.

14.2.2.2 Connect anchored veneer to the backing with anchors that comply with Section 14.2.2.5 and 3.11.

14.2.2.3 Vertical support of anchored masonry veneer

14.2.2.3.1 The weight of anchored veneer shall be supported vertically on concrete or masonry foundations or other noncombustible structural supports.

14.2.2.3.1.1 If anchored veneer with a backing of cold-formed steel framing exceeds the height above the noncombustible foundation given in Table 14.2.1, the weight of the veneer shall be supported by noncombustible construction for each story above the height limit given in Table 14.2.1.

Table 14.2.1 – Height limit from foundation

Height at plate, m	Height at gable, m
9.00	11.00

14.2.2.3.2 When anchored veneer is supported by floor construction, the floor shall be designed to limit deflection as required in Section 4.1.5.1.

14.2.2.3.3 Provide noncombustible lintels or supports attached to noncombustible framing over all openings where the anchored veneer is not self-supporting. The deflection of such lintels or supports shall conform to the requirements of Section 4.1.5.1.

14.2.2.4 **Masonry units** - Masonry units shall be at least 70 mm in actual thickness.

14.2.2.5 Anchor requirements

14.2.2.5.1 Corrugated sheet metal anchors

14.2.2.5.1.1 Corrugated sheet metal anchors shall be at least 22 mm wide, have a base metal thickness of at least 0.8 mm, and shall have corrugations with a wavelength of 7.5 to 12 mm and an amplitude of 1.5 to 2.5 mm.

14.2.2.5.1.2 Corrugated sheet metal anchors shall be placed as follows:

(a) With solid units, embed anchors in the mortar joint and extend into the veneer a minimum of 38 mm, with at least 16 mm mortar cover to the outside face.

(b) With hollow units, embed anchors in mortar or grout and extend into the veneer a minimum of 38 mm, with at least 16 mm mortar or grout cover to the outside face.

14.2.2.5.2 Sheet metal anchors

14.2.2.5.2.1 Sheet metal anchors shall be at least 23 mm wide, have a base metal thickness of at least 1.5 mm and shall:

(a) Have corrugations as given in Section14.2.2.5.1.1, or

(b) Be bent, notched, or punched to provide equivalent performance in pull-out or push-through.

14.2.2.5.2.2 Sheet metal anchors shall be placed as follows:

(a) With solid units, embed anchors in the mortar joint and extend into the veneer a minimum of 38 mm, with at least 16 mm mortar cover to the outside face.

(b) With hollow units, embed anchors in mortar or grout and extend into the veneer a minimum of 38 mm, with at least 16 mm mortar or grout cover to the outside face.

14.2.2.5.3 Wire anchors

14.2.2.5.3.1 Wire anchors shall be at least wire size WD 4 and have ends bent to form an extension from the bend at least 50.0 mm long.

14.2.2.5.3.2 Wire anchors shall be placed as follows:

(a) With solid units, embed anchors in the mortar joint and extend into the veneer a minimum of 40 mm, with at least 16 mm mortar cover to the outside face.

(b) With hollow units, embed anchors in mortar or grout and extend into the veneer a minimum of 40 mm, with at least 16 mm mortar or grout cover to the outside face.

14.2.2.5.4 Joint reinforcement

14.2.2.5.4.1 Ladder-type or tab-type joint reinforcement is permitted. Cross wires used to anchor masonry veneer shall be at least wire size WD 4 and shall be spaced at a maximum of 400 mm on center. Cross wires shall be welded to longitudinal wires, which shall be at least wire size WD 4.

14.2.2.5.4.2 Embed longitudinal wires of joint reinforcement in the mortar joint with at least 16 mm mortar cover on each side.

14.2.2.5.5 Adjustable anchors

14.2.2.5.5.1 Sheet metal and wire components of adjustable anchors shall conform to the requirements of Section 14.2.2.5.1, 14.2.2.5.2, or 14.2.2.5.3. Adjustable anchors with joint reinforcement shall also meet the requirements of Section 14.2.2.5.4.

14.2.2.5.5.2 Maximum clearance between connecting parts of the tie shall be 1.5 mm.

14.2.2.5.5.3 Adjustable anchors shall be detailed to prevent disengagement.

14.2.2.5.5.4 Pintle anchors shall have at least two pintle legs of wire size WD 5 each and shall have an offset not exceeding 30 mm.

14.2.2.5.5.5 Adjustable anchors of equivalent strength and stiffness to those specified in Sections 14.2.2.5.5.1 through 14.2.2.5.5.5 are permitted.

14.2.2.5.6 Anchor spacing

14.2.2.5.6.1 For adjustable two-piece anchors, anchors of wire size WD 4, and 1 mm corrugated sheet metal anchors, provide at least one anchor for each 0.25 m^2 of wall area.

14.2.2.5.6.2 For all other anchors, provide at least one anchor for each 0.3 m^2 of wall area.

14.2.2.5.6.3 Space anchors at a maximum of 810 mm horizontally and 450 mm vertically.

14.2.2.5.6.4 Provide additional anchors around all openings larger than 400 mm in either dimension. Space anchors around perimeter of opening at a maximum of 1 m on center. Place anchors within 300 mm of openings.

14.2.2.5.7 **Joint thickness for anchors** – Mortar bed joint thickness shall be at least twice the thickness of the embedded anchor.

14.2.2.6 **Masonry veneer anchored to steel backing**

14.2.2.6.1 Attach veneer with adjustable anchors.

14.2.2.6.2 Attach each anchor to steel framing with corrosion-resistant screws that have a minimum nominal shank diameter of 5 mm.

14.2.2.6.3 Cold-formed steel framing shall be corrosion resistant and have a minimum base metal thickness of 1 mm.

14.2.2.6.4 Maintain a 110 mm maximum distance between the inside face of the veneer and the steel framing. Maintain a 25 mm minimum air space.

14.2.2.7 **Masonry veneer anchored to masonry or concrete backing**

14.2.2.7.1 Attach veneer to masonry backing with wire anchors, adjustable anchors, or joint reinforcement. Attach veneer to concrete backing with adjustable anchors.

14.2.2.7.2 Maintain a 110 mm maximum distance between the inside face of the veneer and the outside face of the masonry or concrete backing. Maintain a 25 mm minimum air space.

14.2.2.8 **Veneer laid in other than running bond**

Anchored veneer laid in other than running bond shall have joint reinforcement of at least one wire, of size WD 4, spaced at a maximum of 450 mm on center vertically.

14.2.2.9 **Requirements in seismic areas**

14.2.2.9.1 **Seismic Design Category C**

14.2.2.9.1.1 The requirements of this section apply to anchored veneer for buildings in Seismic Design Category C.

14.2.2.9.1.2 Isolate the sides and top of anchored veneer from the structure so that vertical and lateral seismic forces resisted by the structure are not imparted to the veneer.

14.2.2.9.2 **Seismic Design Category D**

14.2.2.9.2.1 The requirements for Seismic Design Category C and the requirements of this section apply to anchored veneer for buildings in Seismic Design Category D.

14.2.2.9.2.2 Support the weight of anchored veneer for each story independent of other stories.

14.2.2.9.2.3 Reduce the maximum wall area supported by each anchor to 75 % of that required in Sections 14.2.2.5.6.1 and 14.2.2.5.6.2. Maximum horizontal and vertical spacing are unchanged.

14.2.2.9.2.4 Provide continuous, single-wire joint reinforcement of minimum wire size WD 4 at a maximum spacing of 450 on center vertically.

SECTION 14.3
ADHERED VENEER

14.3.1 **Alternative design of adhered masonry veneer**. The alternative design of adhered veneer, which is permitted under Section 1.3.5, shall satisfy the following conditions:

(a) Loads shall be distributed through the veneer to the backing using principles of mechanics.

(b) Out-of-plane curvature shall be limited to prevent veneer unit separation from the backing.

(c) All masonry, other than veneer, shall meet the provisions of Section 1.2, excluding Sections 1.2.5 and 1.2.6.

(d) The veneer is not subject to the flexural tensile stress provisions of Section 7.2.

14.3.2 **Prescriptive requirements for adhered masonry veneer**

14.3.2.1 **Unit sizes —** Adhered veneer units shall not exceed 65 mm in specified thickness, 910 mm in any face dimension, nor more that 0.45 m^2 in total face area, and shall not weigh more than 710 Pa .

14.3.2.2 **Wall area limitations —** The height length and area of adhered veneer shall not be limited except as required to control restrained differential movement stresses between veneer and backing.

14.3.2.3 **Backing —** Backing shall provide a continuous, moisture-resistant surface to receive the adhered veneer. Backing is permitted to be masonry, concrete, or metal lath and Portland cement plaster applied to masonry, concrete or steel framing.

14.3.2.4 Adhesion developed between adhered veneer units and backing shall have a shear strength of at least 0.35 MPa based on gross unit surface area when tested in accordance with ASTM C 482, or shall be adhered in compliance with Section 4.1.2.6.

REFERENCED STANDARDS

These are the standards referenced within SBC 305. The standards are listed herein by the promulgating agency of the standard, the standard identification, the effective date and title. The application of the referenced standards shall be as specified in SBC.

1. ACI, 530/ASCE 5/TMS 402, Building Code Requirements for Masonry Structures and Commentary and Specification for Masonry Structures and Commentary, American Concrete Institute (ACI), 38800 Country Club Dr., Farmington Hills, MI 48331, U.S.A.
2. ACI, 530.1/ASCE 6/TMS 602, Building Code Requirements for Masonry Structures and Commentary and Specification for Masonry Structures and Commentary, American Concrete Institute (ACI), 38800 Country Club Dr., Farmington Hills, MI 48331, U.S.A.
3. ANSI, A42.2, Portland Cement and Portland Cement-lime Plastering, Exterior (STUCCO) and Interior, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
4. ANSI, A42.3, Lathing and Furring for Portland Cement-lime Plastering, Exterior (STUCCO) and interior, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
5. ANSI, A108.1A, Specifications for Installation of Ceramic Tile in the Wet-set Method, with Portland Cement Mortar, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
6. ANSI, A108.1B, Specifications for Installation of Ceramic Tile on a Cured Portland Cement Mortar Setting Bed with Dry-set or Latex Portland Cement Mortar, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
7. ANSI, A108.4, Specifications for Installation of Ceramic Tile with Organic Adhesives or Water Cleanable Tile Setting Epoxy Adhesive, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
8. ANSI, A108.5, Ceramic Tile, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
9. ANSI, A108.6, Specifications for Installation of Ceramic Tile with Tile with Chemical Resistant, Water Cleanable Tile Setting and Grouting Epoxy, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
10. ANSI, A108.7, Ceramic Tile, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
11. ANSI, A108.8, Specifications for Installation of Ceramic Tile with Chemical Resistant Furan Mortar and Grout, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
12. ANSI, A108.9, Ceramic Tile, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
13. ANSI, A108.10, Specifications for Installation of Grout in Tilework, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
14. ANSI, A118.1, Specifications for Dry-Set Portland Cement Mortar, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
15. ANSI, A118.2, Specifications for Conductive Dry-Set Portland Cement Mortar, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
16. ANSI, A118.3, Specifications for Chemical Resistant, Water Cleanable Tile-Setting and Grouting Epoxy and Water, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.

17. ANSI, A118.4, Specifications for Latex Portland Cement Mortar, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
18. ANSI, A118.5, Specifications for Chemical Resistant Furan Resin Mortars and Grouts for Tile Installation, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
19. ANSI, A118.6, Specifications for Ceramic Tile Grouts, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
20. ANSI, A118.8, Specifications for Modified Epoxy Emulsion Mortar Grout, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
21. ANSI, A136.1, Ceramic Tile, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
22. ANSI, A137.1, American National Standard Specifications for Ceramic Tile, American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036, U.S.A.
23. ASCE, 7, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Structural Engineering Institute, 1801 Alexander Bell Drive, Reston, VA 20191-4400, U.S.A.
24. ASTM, C 27, Standard Classification of Fireclay and High-Alumina Refractory Brick, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
25. ASTM, C 34, Standard Specification for Structural Clay Load Bearing-Wall Tile, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
26. ASTM, A 36, Standard Specification for Carbon Structural Steel, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
27. ASTM, C 55, Standard Specification for Concrete Building Brick, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
28. ASTM, C 56, Standard Specification for Structural Clay Non-Load-Bearing Tile, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
29. ASTM, C 62, Standard Specification for Building Brick (Solid Masonry Units) Made from Clay or Shale, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
30. ASTM, C 67, Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
31. ASTM, C 73, Standard Specification for Calcium Silicate Brick (Sand-Lime Brick), ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
32. ASTM, A 82, Standard Specification for Steel Wire, Plain, for Concrete Reinforcement, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
33. ASTM, C 90, Standard Specification for Loadbearing Concrete Masonry Units, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
34. ASTM, C 91, Standard Specification for Masonry Cement, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
35. ASTM, E 111, Standard Test Methods for Young's Modulus, Tangent Modulus, and Chord Modulus, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.

36. ASTM, E 119, Standard Specification for Fire Tests of Building Construction and Materials, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
37. ASTM, A 123, Standard Specification for Apparatus for Determination of Water by Distillation, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
38. ASTM, C 126, Standard Specification for Ceramic Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
39. ASTM, C 140, Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units, ASTM International, 100 Bar Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959, U.S.A.
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Gratitude

In appreciation and gratitude
to The Custodian of the Two Holy Mosques
King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence
& Aviation and Inspector General

For their continuous support and gracious consideration,
the Saudi Building Code National Committee (SBCNC)
is honored to present the first issue of
the Saudi Building Code (SBC).

Saudi Building Code Requirements

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PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11th June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom.

The Saudi Building Code Structural Requirements for Steel Structures (SBC 306) were developed based on ICC code in addition to American Institute of Steel Construction Inc. (AISC). AISC grants a limited license to the Saudi Building Code National Committee (SBCNC) to utilize the following AISC publications in development of building codes and similar construction standards Specification for Structural Steel Buildings (ANSI / AISC 360-05), Seismic Provisions for Structural Steel Buildings (ANSI / AISC 341-05), Code of Standard Practice for Structural Steel Buildings and Bridges (AISC 303-05) and Specification for Safety – Related Steel Structures for Nuclear Facilities (ANSI / AISC N690-06).

The development process of SBC 306 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made on AISC-LRFD, 1999, such as merging the Appendices into the main text of the Code and deleting parts or paragraphs of the Appendices and the Commentaries that are irrelevant to the Saudi Building Code. Only SI units are used through out the Code.

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SBC 306 was reviewed and significantly revised by special technical committee during the period Aug.-Sept. 2007. The committee members are:

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Dr. Saleh I. Al-Deghaiter,
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(Civil Eng. Dept., King Saud Univ.)

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CHAPTER 1
GENERAL PROVISIONS

SECTION 1.1
SCOPE

The Saudi Building Code for Steel Structures referred to as SBC 306, provides minimum requirements for design and construction of Steel Structures. SBC 306 shall govern the design, fabrication, and erection of steel-framed buildings.

- Seismic design of buildings shall comply with the *AISC Seismic Provisions for Structural Steel Buildings, Seismic Provision* supplement No. 1 and with this Code.
- Single angle members shall comply with the AISC specification for *Load and Resistance Factor Design of Single-Angle Members and* with this Code.
- Hollow structural sections (HSS) shall comply with the AISC *Specification for the Design of Steel Hollow Structural Sections* and with this Code.

As used in this code, the term *structural steel* refers to the steel elements of the structural steel frame essential to the support of the required loads.

SECTION 1.2
TYPES OF CONSTRUCTION

Two basic types of construction and associated design assumptions shall be permitted under the conditions stated herein, and each will govern in a specific manner the strength of members and the types and strength of their connections.

Type FR (fully restrained), commonly designated as “rigid-frame” (continuous frame), assumes that connections have sufficient stiffness to maintain the angles between intersecting members.

Type PR (partially restrained) assumes that connections have insufficient stiffness to maintain the angles between intersecting members. When connection restraint is considered, use of Type PR construction under this code requires that the strength, stiffness and ductility characteristics of the connections be incorporated in the analysis and design. These characteristics shall be documented in the technical literature or established by analytical or experimental means.

When connection restraint is ignored, commonly designated “simple framing,” it is assumed that for the transmission of gravity loads the ends of the beams and girders are connected for shear only and are free to rotate. For “simple framing” the following requirements apply:

(1) The connections and connected members shall be adequate to resist the factored gravity loads as “simple beams.”

(2) The connections and connected members shall be adequate to resist the factored lateral loads.

(3) The connections shall have sufficient inelastic rotation capacity to avoid overload of fasteners or welds under combined factored gravity and lateral loading. The type of construction assumed in the design shall be indicated on the design documents. The design of all connections shall be consistent with the assumption.

SECTION 1.3
MATERIAL

1.3.1 Structural Steel

1.3.1.1 **ASTM Designations**. Material conforming one of the following standard specifications is approved for use under this code:

Carbon Structural Steel, ASTM A36/A36M

Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless ASTM A53/A53M, Gr. B

High-Strength Low-Alloy Structural Steel, ASTM A242/A242M

Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes, ASTM A500

Hot-Formed Welded and Seamless Carbon Steel Structural Tubing, ASTM 501

High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding, ASTM A514/A514M

High-Strength Carbon-Manganese Steel of Structural Quality, ASTM A529/A529M

Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality, ASTM A570/ A570M, Gr. 275, 310, and 345

High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality, ASTM A572/A572M

High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100 mm Thick, ASTM A588/A588M

Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance, ASTM A606

Steel, Sheet and Strip, High-Strength, Low-Alloy, Columbium or Vanadium, or Both, Hot-Rolled and Cold-Rolled, ASTM A607

Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing, ASTM A618

Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges, ASTM A709/A709M

Quenched and Tempered Low-Alloy Structural Steel Plate with 485 MPa Minimum Yield Strength to 100 mm Thick, ASTM A852/A852M

High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST), ASTM A913/A913M Steel for Structural Shapes for Use in Building Framing, ASTM A992/A992M

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M, Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling or A568/A568M, Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for, as applicable, shall constitute sufficient evidence of conformity with one of the above ASTM standards. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

***Note**: Materials with other international designations (e.g. JIS, EN) considered equivalent to ASTM are also approved for use under this code.*

1.3.1.2 **Unidentified Steel.** Unidentified steel, if surface conditions are acceptable according to criteria contained in ASTM A6/A6M, is permitted to be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

1.3.1.3 **Heavy Shapes.** For ASTM A6/A6M Group 4 and 5 rolled shapes to be used as members subject to primary tensile stresses due to tension or flexure, toughness need not be specified if splices are made by bolting. If such members are spliced using complete-joint-penetration groove welds, the steel shall be specified in the contract documents to be supplied with Charpy V-notch (CVN) impact testing in accordance with ASTM A6/A6M, Supplementary Requirement S5. The impact test shall meet a minimum average value of 27 J absorbed energy at +21°C and shall be conducted in accordance with ASTM A673/A673M, with the following exceptions:

1. The center longitudinal axis of the specimens shall be located as near as practical to midway between the inner flange surface and the center of the flange thickness at the intersection with the web mid-thickness.
2. Tests shall be conducted by the producer on material selected from a location representing the top of each ingot or part of an ingot used to produce the product represented by these tests.

For plates exceeding 50 mm thick used for built-up cross-sections with bolted splices and subject to primary tensile stresses due to tension or flexure, material toughness need not be specified. If such cross-sections are spliced using complete-joint-penetration welds, the steel shall be specified in the contract documents to be supplied with Charpy V-notch testing in accordance with ASTM A6/A6M, Supplementary Requirement S5. The impact test shall be conducted by the producer in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 27 J absorbed energy at +21°C.

The above supplementary requirements also apply when complete-joint-penetration welded joints through the thickness of ASTM A6/A6M Group 4 and 5 shapes and built-up cross sections with thickness exceeding 50 mm are used in connections subjected to primary tensile stress due to tension or flexure of such members. The requirements need not apply to ASTM A6/A6M Group 4 and 5 shapes and built-up members with thickness exceeding 50 mm to which members other than ASTM A6/A6M Group 4 and 5 shapes and built-up members are connected by complete-joint- penetration welded joints through the thickness of the thinner material to the face of the heavy material.

Additional requirements for joints in heavy rolled and built-up members are given in Sections 10.1.5, 10.2.8 and 13.2.2.

***Note**: Materials with other international designations (e.g. JIS, EN) considered equivalent to ASTM are also approved for use under this code.*

1.3.2 **Steel Castings and Forgings.** Cast steel shall conform to one of the following standard specifications:

Steel Castings, Carbon, for General Application, ASTM A27/A27M, Gr. 450-240

Steel Castings, High Strength, for Structural Purposes, ASTM A148/148M Gr.

550-345

Steel forgings shall conform to the following standard specification:

Steel Forgings Carbon and Alloy, for General Industrial Use, ASTM A668/ A668M

Certified test reports shall constitute sufficient evidence of conformity with standards.

***Note**: Materials with other international designations (e.g. JIS, EN) considered equivalent to ASTM are also approved for use under this code.*

1.3.3 **Bolts, Washers, and Nuts.** Steel bolts, washers, and nuts shall conform to one of the following standard specifications:

Carbon and Alloy Steel Nuts for Bolts for High-Pressure or High-Temperature Service, or Both, ASTM A194/A194M

Carbon Steel Bolts and Studs, 410 MPa Tensile Strength, ASTM A307

Structural Bolts, Steel, Heat Treated, 830/720 MPa Minimum Tensile Strength, ASTM A325

High-Strength Bolts for Structural Steel Joints [Metric], ASTM A325M Quenched and Tempered Steel Bolts and Studs, ASTM A449

Heat-Treated Steel Structural Bolts, 1030 MPa Minimum Tensile Strength, ASTM A490

High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric], ASTM A490M

Carbon and Alloy Steel Nuts, ASTM A563

Carbon and Alloy Steel Nuts [Metric], ASTM A563M

Hardened Steel Washers, ASTM F436

Hardened Steel Washers [Metric], ASTM F436M

Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners, ASTM F959

Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners [Metric], ASTM F959M

“Twist Off” Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 830/720 MPa Minimum Tensile Strength, ASTM F1852

ASTM A449 bolts are permitted to be used only in connections requiring bolt diameters greater than 38 mm and shall not be used in slip-critical connections. Manufacturer’s certification shall constitute sufficient evidence of conformity with the standards.

***Note**: Materials with other international designations (e.g. JIS, EN) considered equivalent to ASTM are also approved for use under this code.*

1.3.4 **Anchor Rods and Threaded Rods.** Anchor rods and threaded rod steel shall conform to one of the following standard specifications:

Carbon Structural Steel, ASTM A36/A36M

Alloy Steel and Stainless Steel Bolting Materials for High-Temperature Service, ASTM A193/A193M

Quenched and Tempered Alloy Steel Bolts, Studs and Other Externally Threaded Fasteners, ASTM A354

High-Strength Low-Alloy Columbium-Vanadium Structural Steel, ASTM A572/A572M

High-Strength Low-Alloy Structural Steel with 345 MPa Minimum

Yield Point to 100 mm Thick, ASTM A588/A588M

Anchor Bolts, Steel, 250, 380, 720 MPa - Yield Strength, ASTM F1554

Threads on anchor rods and threaded rods shall conform to the Unified Standard Series of ASME B18.2.6 and shall have Class 2A tolerances.

Steel bolts conforming to other provisions of Section 1.3.3 are permitted as anchor rods. A449 material is acceptable for high-strength anchor rods and threaded rods of any diameter.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

***Note**: Materials with other international designations (e.g. JIS, EN) considered equivalent to ASTM are also approved for use under this code.*

1.3.5 **Filler Metal and Flux for Welding.** Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society:

Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding, AWS A5.1

Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding, AWS A5.5

Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding, AWS A5.17/A5.17M

Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding, AWS A5.18

Specification for Carbon Steel Electrodes for Flux Cored Arc Welding, AWS A5.20

Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding, AWS A5.23/A5.23M

Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding, AWS A5.25/A5.25M

Specification for Carbon and Low-Alloy Steel Electrodes for Electrogas Welding, AWS A5.26/A5.26M

Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding, AWS A5.28

Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding, AWS A5.29

Specification for Welding Shielding Gases, AWS A5.32/A5.32M Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

Filler metals and fluxes that are suitable for the intended application shall be selected.

1.3.6 **Stud Shear Connectors.** Steel stud shear connectors shall conform to the requirements of *Structural Welding Code—Steel*, AWS D1.1.

Manufacturer's certification shall constitute sufficient evidence of conformity with the specifications.

SECTION 1.4
LOADS AND LOAD COMBINATIONS

The nominal loads and factored load combinations shall be as stipulated by SBC 301.

SECTION 1.5
DESIGN BASIS

1.5.1 **Required Strength at Factored Loads.** The required strength of structural members and connections shall be determined by structural analysis for the appropriate factored load combinations as stipulated in Section 1.4.

Design by either elastic or plastic analysis is permitted, except that design by plastic analysis is permitted only for steels with specified minimum yield stresses not exceeding 450 MPa and is subject to provisions of Sections 2.5.2, 3.1.1, 3.2.1, 3.2.2, 5.1.2, 6.1.3, 8.1, and 9.1.

Beams and girders composed of compact sections, as defined in Section 2.5.1, and satisfying the unbraced length requirements of Section 6.1.3 (including composite members) which are continuous over supports or are rigidly framed to columns may be proportioned for nine-tenths of the negative moments produced by the factored gravity loading at points of support, provided that the maximum positive moment is increased by one-tenth of the average negative moments. This reduction is not permitted for hybrid beams, members of A514/A514M steel, or moments produced by loading on cantilevers. If the negative moment is resisted by a column rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial force and flexure, provided that the axial force does not exceed ϕ_c times $0.15A_gF_y$,

Where:

A_g = gross area, mm^2
F_y = specified minimum yield stress, MPa
ϕ_c = resistance factor for compression

1.5.2 **Limit States.** LRFD is a method of proportioning structures so that no applicable limit state is exceeded when the structure is subjected to all appropriate factored load combinations.

Strength limit states are related to safety and concern maximum load carrying capacity. Serviceability limit states are related to performance under normal service conditions. The term "resistance" includes both strength limit states and serviceability limit states.

1.5.3 **Design for Strength.** The required strength shall be determined for each applicable load combination as stipulated in Section 1.4.

The design strength of each structural component or assemblage shall equal or exceed the required strength based on the factored loads. The design strength ϕR_n for each applicable limit state is calculated as the nominal strength R_n multiplied by a resistance factor. Nominal strengths R_n and resistance factors are given in Chapters 4 through 11.

1.5.4 **Design for Serviceability and Other Considerations.** The overall structure and the individual members, connections, and connectors shall be checked for serviceability. Provisions for design for serviceability are given in Chapter 12.

SECTION 1.6
DESIGN DOCUMENTS

The design drawings shall show a complete design with sizes, sections, and relative locations of all members. Floor levels, column centers and offsets shall be dimensioned. Drawings shall be drawn to a scale large enough to show the information clearly.

Design documents shall indicate the type or types of construction as defined in Section 1.2 and include the required strengths (moments and forces) if necessary for preparation of shop drawings.

Where joints are to be assembled with high-strength bolts, the design documents shall indicate the connection type (i.e., snug-tightened, pretensioned, or slip-critical).

Camber of trusses, beams, and girders, if required, shall be specified in the design documents.

The requirements for stiffeners and bracing shall be shown in the design documents.

Welding and inspection symbols used on design and shop drawings shall be the American Welding Society symbols. Welding symbols for special requirements not covered by AWS are permitted to be used provided complete explanations thereof are shown in the design documents.

Weld lengths called for in the design documents and on the shop drawings shall be the net effective lengths.

CHAPTER 2
DESIGN REQUIREMENTS

This chapter contains provisions, which are common to this code as a whole.

SECTION 2.1
GROSS AREA

The gross area A_g of a member at any point is the sum of the products of the thickness and the gross width of each element measured normal to the axis of the member. For angles, the gross width is the sum of the widths of the legs less the thickness.

SECTION 2.2
NET AREA

Critical net area is based on net width and load transfer at a particular chain.

The net area A_n of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as 2 mm greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section 10.3.2, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/4g$

Where:

s = longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm.

g = transverse center-to-center spacing (gage) between fastener gage lines, mm.

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

In determining the net area across plug or slot welds, the filler metal shall not be considered as adding to the net area.

SECTION 2.3
EFFECTIVE AREA OF TENSION MEMBERS

The effective area of tension members shall be determined as follows:

(1) When tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds, the effective area A_e is equal to the net area A_n.

(2) When the tension load is transmitted by fasteners or welds through some but not all of the cross-sectional elements of the member, the effective area A_e shall be computed as follows:

(a) When the tension load is transmitted only by fasteners

$$A_e = A_n U \qquad \textbf{(2.3-1)}$$

Where:
U = reduction coefficient
= $1-(\bar{x}/l) \leq 0.9$
$\bar{x}$ = connection eccentricity, mm
l = length of the connection in the direction of loading, mm

(b) When the tension load is transmitted only by longitudinal welds to other than a plate member or by longitudinal welds in combination with transverse welds

$$A_e = A_g U \quad \textbf{(2.3-2)}$$

Where:
U = $1-(\bar{x}/l) \leq 0.9$
A_g = gross area of member, mm^2

(c) When the tension load is transmitted only by transverse welds

$$A_e = AU \quad \textbf{(2.3-3)}$$

Where:
A = area of directly connected elements, mm^2
U = 1.0

(d) When the tension load is transmitted to a plate only by longitudinal welds along both edges at the end of the plate

$$A_e = A_g U \quad \textbf{(2.3-4)}$$

Where:
For $l \geq 2w$ . . . . $U = 1.00$
For $2w > l \geq 1.5w$ . . .. $U = 0.87$
For $1.5w > l \geq w$ . . . . $U = 0.75$
Where:
l = length of weld, mm
w = plate width (distance between welds), mm

The reduction coefficient U is applied to the net area A_n of bolted members and to the gross area A_g of welded members. As the length of connection l is increased, the shear lag effect is diminished. This concept is expressed empirically by the equation for U.

For any given profile and connected elem*ents* $\bar{x}$ is a fixed geometric property. It is illustrated as the distance from the connection plane, or face of the member, to the centroid of the member section resisting the connection force. See Figure 2.3-1. The length l is dependent upon the number of fasteners or equivalent length of weld required to develop the given tensile force, and this in turn is dependent upon the mechanical properties of the member and the capacity of the fasteners or weld used. The length l is illustrated as the distance, parallel to the line of force, between the first and last fasteners in a line for bolted connections. The number of bolts in a line, for the purpose of the determination of l, is determined by the line with the maximum number of bolts in the connection. For staggered bolts, the out-to-out dimension is used for l. See Figure 2.3-2. If all lines have only one bolt, it is probably conservative to use A_e equal to the net area of the connected element. For welded connections, l is the length of the weld parallel to the line of force. For combinations of longitudinal and transverse welds (see Figure 2.3-3), l is the length of longitudinal weld because the transverse weld has little or no effect on the shear lag problem, i.e., it does little to get the load into the unattached portions of the member.

For bolted or riveted connections the following values of U may be used:

(a) W, M, or S shapes with flange widths not less than two-thirds the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than three fasteners per line in the direction of stress, $U = 0.90$.

(b) W, M, or S shapes not meeting the conditions of subparagraph a, structural tees cut from these shapes, and all other shapes including built-up cross sections, provided the connection has no fewer than three fasteners per line in the direction of stress, $U = 0.85$.

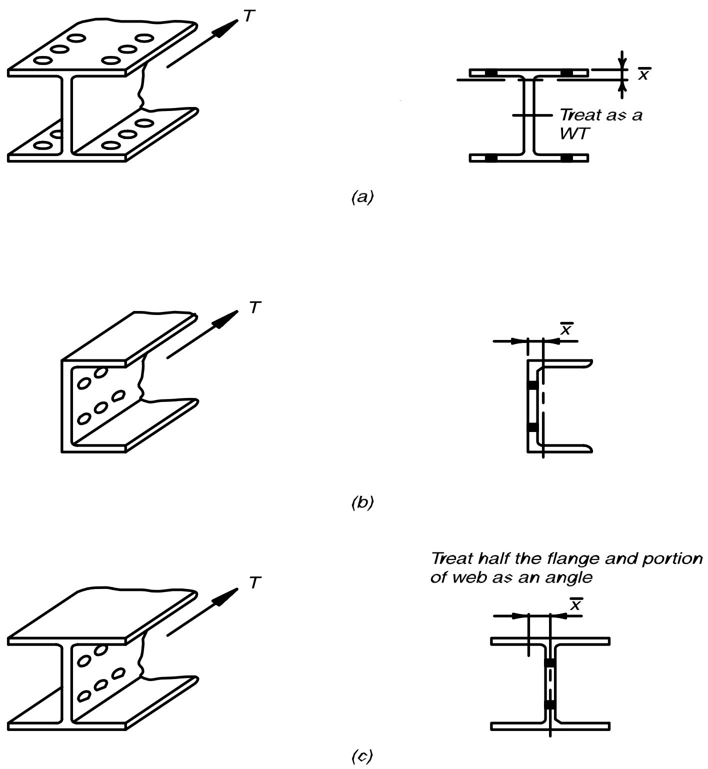


Figure 2.3-1 Determination of $\bar{x}$ for U.

(c) All members having only two fasteners per line in the direction of stress, $U = 0.75$.

When a tension load is transmitted by fillet welds to some but not all elements of a cross section, the weld strength will control.

Larger values of U are permitted to be used when justified by tests or other rational criteria.

For effective area of connecting elements, see Section 10.5.2.

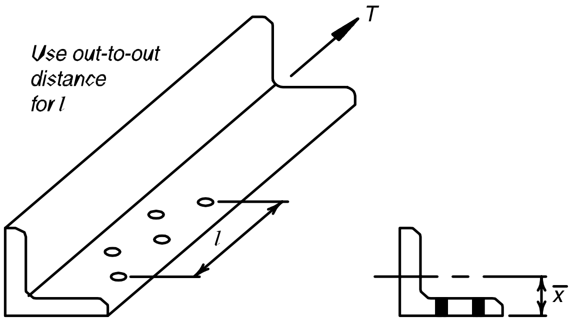


Figure 2.3-2. Staggered holes.

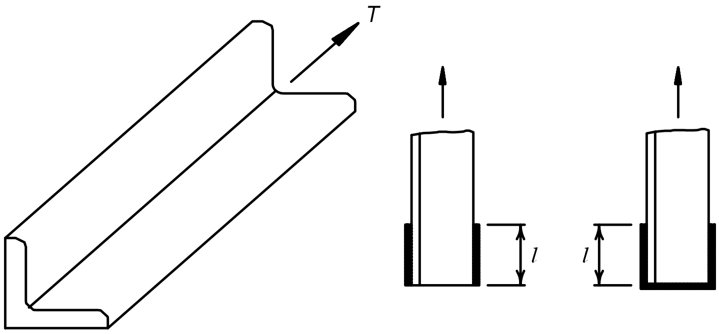


Figure 2.3-3. Longitudinal and transverse welds.

SECTION 2.4
STABILITY

General stability shall be provided for the structure as a whole and for each of its elements.
Consideration shall be given to the significant effects of the loads on the deflected shape of the structure and its individual elements.

SECTION 2.5
LOCAL BUCKLING

2.5.1 **Classification of Steel Sections.** Steel sections are classified as compact, non-compact, or slender-element sections. For a section to qualify as compact, its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios λ_p from Table 2.5-1. If the width-thickness ratio of one or more compression elements exceeds λ_p, but does not exceed λ_r, the section is non-compact. If the width-thickness ratio of any element exceeds λ_r from Table 2.5-1, the section is referred to as a slender-element compression section.

For un-stiffened elements which are supported along only one edge parallel to the direction of the compression force, the width shall be taken as follows:

(a) For flanges of I-shaped members and tees, the width b is half the full-flange width, b_f.

(b) For legs of angles and flanges of channels and zees, the width b is the full nominal dimension.

(c) For plates, the width b is the distance from the free edge to the first row of fasteners or line of welds.

(d) For stems of tees, d is taken as the full nominal depth.

For stiffened elements which are supported along two edges parallel to the direction of the compression force, the width shall be taken as follows:

(a) For webs of rolled or formed sections, h is the clear distance between flanges less the fillet or corner radius at each flange; h_c is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.

(b) For webs of built-up sections, h is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and h_c is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.

(c) For flange or diaphragm plates in built-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.

(d) For flanges of rectangular hollow structural sections, the width b is the clear distance between webs less the inside corner radius on each side. If the corner radius is not known, the width may be taken as the total section width minus three times the thickness. The thickness t shall be taken as the design wall thickness. When the design wall thickness is not known, it is permitted to be taken as 0.93 times the nominal wall thickness.

The limiting width-thickness ratio for: a) the design of webs in combined flexure and axial compression and, b) the design of members containing slender compression elements are as follows:

- For members with unequal flanges and with webs in combined flexural and axial compression, λ_r for the limit state of web local buckling is

$$\lambda_r = 1.49\sqrt{\frac{E}{F_y}}\left[1+2.83\left(\frac{h}{h_c}\right)\left(1-\frac{P_u}{\phi_b P_y}\right)\right] \qquad \textbf{(2.5-1)}$$

$$\frac{3}{4} \le \frac{h}{h_c} \le \frac{3}{2}$$

- For members with unequal flanges with webs subjected to flexure only, λ_r for the limit state of web local buckling is

$$\lambda_r = 1.49\sqrt{\frac{E}{F_y}}\left[1+2.83\left(\frac{h}{h_c}\right)\right] \qquad \textbf{(2.5-2)}$$

$$\frac{3}{4} \le \frac{h}{h_c} \le \frac{3}{2}$$

where λ_r, h, and h_c are as defined in Section 2.5.1.

These substitutions shall be made in Sections 6 and 7 when applied to members with unequal flanges. If the compression flange is larger than the tension flange, λ_r shall be determined using Equation 2.5-1, 2.5-2, or Table 2.5-1.

For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

2.5.2 **Design by Plastic Analysis.** Design by plastic analysis is permitted, as limited in Section 1.5.1, when flanges subject to compression involving hinge rotation and all webs have a width-thickness ratio less than or equal to the limiting λ_p from Table 2.5-1. For circular hollow sections see Footnote d of Table 2.5-1.

2.5.3 **Slender-Element Compression Sections.** For the flexural design of I-shaped sections, channels and rectangular or circular sections with slender flange elements, see section 6.1. For other shapes in flexure or members in axial compression that have slender compression elements, see Section 2.5.3. For plate girders with slender web elements, Section 7.

Axially loaded members containing elements subject to compression which have a width-thickness ratio in excess of the applicable λ_r as stipulated in Section 2.5.1 shall be proportioned according to this section. Flexural members with slender compression elements shall be designed in accordance with Sections 6 and 7. Flexural members with proportions not covered by Section 6.1 shall be designed in accordance with this Section.

The limiting width-thickness ratio for: the design of members containing slender compression elements are given below in Sections 2.5.3.1 to 2.5.3.4.

2.5.3.1 **Un-stiffened Compression Elements.** The design strength of un-stiffened compression elements whose width-thickness ratio exceeds the applicable limit λ_r as stipulated in Section 2.5.1 shall be subject to a reduction factor Q_s. The value of Q_s shall be determined by Equations 2.5-3 through 2.5-10, as applicable. When such elements comprise the compression flange of a flexural member, the design flexural strength, in MPa, shall be computed using $\phi_b F_y Q_s$, where $\phi_b = 0.90$. The design strength of axially loaded compression members shall be modified by the appropriate reduction factor Q, as provided in Section 2.5.3.4.

(a) For single angles:

when $0.45\sqrt{E/F_y} < b/t < 0.91\sqrt{E/F_y}$:

$$Q_s = 1.340 - 0.76(b/t)\sqrt{F_y/E} \qquad \textbf{(2.5-3)}$$

when $b/t \geq 0.91\sqrt{E/F_y}$:

$$Q_s = 0.53E/\left[F_y(b/t)^2\right] \qquad \textbf{(2.5-4)}$$

(b) For flanges, angles, and plates projecting from rolled beams or columns or other compression members:

when $0.56\sqrt{E/F_y} < b/t < 1.03\sqrt{E/F_y}$:

$$Q_s = 1.415 - 0.74(b/t)\sqrt{F_y/E} \qquad \textbf{(2.5-5)}$$

when $b/t \geq 1.03\sqrt{E/F_y}$:

$$Q_s = 0.69E/\left[F_y(b/t)^2\right] \qquad \textbf{(2.5-6)}$$

(c) For flanges, angles and plates projecting from built-up columns or other compression members:

when $0.64\sqrt{E/(F_y/k_c)} < b/t < 1.17\sqrt{E/(F_y/k_c)}$:

$$Q_s = 1.415 - 0.65(b/t)\sqrt{(F_y/k_c E)} \quad \textbf{(2.5-7)}$$

when $b/t \geq 1.17\sqrt{E/(F_y/k_c)}$:

$$Q_s = 0.90 E k_c / \left[F_y (b/t)^2\right] \quad \textbf{(2.5-8)}$$

The coefficient, k_c, shall be computed as follows:

(a) For I-shaped sections:

$$k_c = \frac{4}{\sqrt{h/t_w}},\ 0.35 \leq k_c \leq 0.763$$

where:

h = depth of web, mm

t_w = thickness of web, mm

(b) For other sections:

$k_c = 0.763$

(c) For stems of tees:

when $0.75\sqrt{E/F_y} < d/t < 1.03\sqrt{E/F_y}$:

$$Q_s = 1.908 - 1.22(d/t)\sqrt{F_y/E} \quad \textbf{(2.5-9)}$$

when $d/t \geq 1.03\sqrt{E/F_y}$:

$$Q_s = 0.69 E / \left[F_y (d/t)^2\right] \quad \textbf{(2.5-10)}$$

where:

d = width of un-stiffened compression element as defined in Section 2.5.1, mm

t = thickness of un-stiffened element, mm

2.5.3.2 **Stiffened Compression Elements.** When the width-thickness ratio of uniformly compressed stiffened elements (except perforated cover plates) exceeds the limit λ_r stipulated in Section 2.5.1, a reduced effective width b_e shall be used in computing the design properties of the section containing the element.

(a) For flanges of square and rectangular sections of uniform thickness:

when $\frac{b}{t} \geq 1.40\sqrt{\frac{E}{f}}$:

$$b_e = 1.91t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \quad \textbf{(2.5-11)}$$

otherwise $b_e = b$.

(b) For other uniformly compressed elements:

when $\dfrac{b}{t} \geq 1.49\sqrt{\dfrac{E}{f}}$:

$$b_e = 1.91t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \qquad \textbf{(2.5-12)}$$

otherwise $b_e = b$.

where:

b = actual width of a stiffened compression element, as defined in Section 2.5.1, mm

b_e = reduced effective width, mm

t = element thickness, mm

f = computed elastic compressive stress in the stiffened elements, based on the design properties as specified in Section 2.5.3.3, MPa. If un-stiffened elements are included in the total cross section, f for the stiffened element must be such that the maximum compressive stress in the un-stiffened element does not exceed $\phi_c F_{cr}$ as defined in Section 2.5.3.4 with $Q = Q_s$ and $\phi_c = 0.85$, or $\phi_b F_y Q_s$ with $\phi_b = 0.90$, as applicable.

(c) For axially loaded circular sections with diameter-to-thickness ratio D/t greater than $0.11E/F_y$ but less than $0.45E / F_y$

$$Q = Q_a = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \qquad \textbf{(2.5-13)}$$

where:

D = outside diameter, mm

t = wall thickness, mm

2.5.3.3 **Design Properties.** Properties of sections shall be determined using the full cross section, except as follows:

In computing the moment of inertia and elastic section modulus of flexural members, the effective width of uniformly compressed stiffened elements b_e, as determined in Section 2.5.3.2, shall be used in determining effective cross-sectional properties.

For unstiffened elements of the cross section, Q_s is determined from Section 2.5.3.1. For stiffened elements of the cross section

$$Q_a = \frac{\text{effective area}}{\text{actual area}} \qquad \textbf{(2.5-14)}$$

where the effective area is equal to the summation of the effective areas of the cross section.

2.5.3.4 **Design Strength.** For axially loaded compression members the gross cross-sectional area and the radius of gyration r shall be computed on the basis of the actual cross section. The critical stress F_{cr} shall be determined as follows:

(a) For $\lambda_c \sqrt{Q} \leq 1.5$:

$$F_{cr} = Q(0.658^{Q\lambda_c^2})F_y \qquad \textbf{(2.5-15)}$$

(b) For $\lambda_c \sqrt{Q} > 1.5$:

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right]F_y \qquad \textbf{(2.5-16)}$$

where

$$Q = Q_s Q_a \qquad \textbf{(2.5-17)}$$

Cross sections comprised of only un-stiffened elements, $Q = Q_s$, (Q_a = 1.0),

Cross sections comprised of only stiffened elements, $Q = Q_a$, (Q_s = 1.0),

Cross sections comprised of both stiffened and un-stiffened elements, $Q = Q_s Q_a$.

TABLE 2.5-1
Limiting Width-Thickness Ratios for Compression Elements

Description of Element		Width-Thickness Ratio	Limiting Width-Thickness Ratios	
			λ_p (compact)	λ_r (non-compact)
Unstiffened Elements	Flanges of I-shaped rolled beams and channels in flexure	b/t	$0.38\sqrt{E/F_y}$ [c]	$0.83\sqrt{E/F_L}$ [e]
	Flanges of I-shaped hybrid or welded beams in flexure	b/t	$0.38\sqrt{E/F_{yf}}$	$0.95\sqrt{E/(F_L/k_c)}$ [e], [f]
	Flanges projecting from built-up compression members	b/t	NA	$0.64\sqrt{E/(F_y/k_c)}$ [f]
	Flanges of I-shaped sections in pure compression, plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact; flanges of channels in pure compression	b/t	NA	$0.56\sqrt{E/F_y}$
	Legs of single angle struts; legs of double angle struts with separators; un-stiffened elements, i.e., supported along one edge	b/t	NA	$0.45\sqrt{E/F_y}$
	Stems of tees	d/t	NA	$0.75\sqrt{E/F_y}$

PROPORTIONS OF BEAMS AND GIRDERS

TABLE 2.5-1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

	Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios: λ_p (compact)	λ_r (non-compact)
Stiffened Elements	Flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t		
	for uniform compression		$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$
	for plastic analysis		$0.939\sqrt{E/F_y}$	
	Unsupported width of cover plates perforated with a succession of access holes [b]	b/t	NA	$1.86\sqrt{E/F_y}$
	Webs in flexural compression [a]	h/t_w	$3.76\sqrt{E/F_y}$ [c], [g]	$5.70\sqrt{E/F_y}$ [h]
	Webs in combined flexural and axial compression	h/t_w	for $P_u/\phi_b P_y \le 0.125$ [c], [g] $3.76\sqrt{\frac{E}{F_y}}\left(1-\frac{2.75P_u}{\phi_b P_y}\right)$ for $P_u/\phi_b P_y > 0.125$ [c], [g] $1.12\sqrt{\frac{E}{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right)$ $\ge 1.49\sqrt{\frac{E}{F_y}}$	[h] $5.70\sqrt{\frac{E}{F_y}}\left(1-0.74\frac{P_u}{\phi_b P_y}\right)$

PROPORTIONS OF BEAMS AND GIRDERS

TABLE 2.5-1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

	Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios: λ_p (compact)	λ_r (non-compact)
	All other uniformly compressed stiffened elements, i.e., supported along two edges	b/t h/t_w	NA	$1.49\sqrt{E/F_y}$
	Circular hollow sections In axial compression In flexure	D/t	[d] NA $0.07E/F_y$	 $0.11E/F_y$ $0.31E/F_y$

[a] For hybrid beams, use the yield strength of the flange F_{yf} instead of F_y.
[b] Assumes net area of plate at widest hole.
[c] Assumes an inelastic ductility ratio (ratio of strain at fracture to strain at yield) of 3. When the seismic response modification factor R is taken greater than 3, a greater rotation capacity may be required.
[d] For plastic design use $0.045E/F_y$.
[e] F_L = smaller of $(F_{yf} - F_r)$ or F_{yw}, (MPa)
F_r = compressive residual stress in flange
= 69 MPa for rolled shapes
= 114 MPa for welded shapes
[f] $k_c = \frac{4}{\sqrt{h/t_w}}$ and $0.35 \le k_c \le 0.763$
[g] For members with unequal flanges, use h_p of h when comparing to λ_p.
[h] For members with unequal flanges, see Section 2.5.

SECTION 2.6
BRACING AT SUPPORTS

At points of support for beams, girders and trusses, restraint against rotation about their longitudinal axis shall be provided.

SECTION 2.7
LIMITING SLENDERNESS RATIOS

For members in which the design is based on compression, the slenderness ratio *Kl/r* preferably should not exceed 200.
For members in which the design is based on tension, the slenderness ratio l/r preferably should not exceed 300. The above limitation does not apply to rods in tension. Members in which the design is dictated by tension loading, but which may be subject to some compression under other load conditions, need not satisfy the compression slenderness limit.

SECTION 2.8
SIMPLE SPANS

Beams, girders and trusses designed on the basis of simple spans shall have an effective length equal to the distance between centers of gravity of the members to which they deliver their end reactions.

SECTION 2.9
END RESTRAINT

Beams, girders, and trusses designed on the assumptions of full or partial end restraint, as well as the sections of the members to which they connect, shall have design strengths, as prescribed in Chapters 4 through 11, equal to or exceeding the effect of factored forces and moments except that some inelastic but self-limiting deformation of a part of the connection is permitted.

SECTION 2.10
PROPORTIONS OF BEAMS AND GIRDERS

When rolled or welded shapes, plate girders and cover-plated beams are proportioned on the basis of flexural strength of the gross section:

(a) If $$0.75F_u A_{fn} \geq 0.9Fy\, A_{fg} \qquad \textbf{(2.10-1)}$$

no deduction shall be made for bolt or rivet holes in either flange, where

A_{fg} = gross flange area, mm^2

A_{fn} = net tension flange area calculated in accordance with the provisions of Section 2.1 and 2.2, mm^2

F_u = specified minimum tensile strength, MPa

(b) If $$0.75F_u A_{fn} < 0.9F_y A_{fg} \qquad \textbf{(2.10-2)}$$

the member flexural properties shall be based on an effective tension flange area A_{fe}

$$A_{fe} = \frac{5}{6}\frac{F_u}{F_y}A_{fn} \qquad \textbf{(2.10-3)}$$

and the maximum flexural strength shall be based on the elastic section modulus.

Other design requirements for *proper* proportioning of beams and girders are as follows:

Hybrid girders shall be proportioned by the flexural strength of their gross section, subject to the applicable provisions in Section 7.1, provided they are not required to resist an axial force greater than ϕ_b times $0.15F_{yf}\,A_g$, where F_{yf} is the specified minimum yield stress of the flange material and A_g is the gross area. No limit is placed on the web stresses produced by the applied bending moment for which a hybrid girder is designed, except as provided in Section 11.3. To qualify as hybrid girders, the flanges at any given section shall have the same cross-sectional area and be made of the same grade of steel.

Flanges of welded beams or girders may be varied in thickness or width by splicing a series of plates or by the use of cover plates.

The total cross-sectional area of cover plates of bolted or riveted girders shall not exceed 70 percent of the total flange area.

High-strength bolts, rivets, or welds connecting flange to web, or cover plate to flange, shall be *proportioned to resist the total horizontal shear* resulting from the bending forces on the girder. The longitudinal distribution of these bolts, rivets, or intermittent welds shall be in proportion to the intensity of the shear. However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Section 5.4 or 4.2, respectively. Bolts, rivets, or welds connecting flange to web shall also be proportioned to transmit to the web any loads applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical connection, rivets, or fillet welds. The attachment shall be adequate, at the applicable design strength given in Sections 10.2.2, 10.3.8 or 11.3 to develop the cover plate's portion of the flexural design strength in the beam or girder at the theoretical cutoff point.

For *welded cover plates*, the welds connecting the cover plate termination to the beam or girder shall have continuous welds along both edges of the cover plate in the length a', defined below, and shall be adequate, at the applicable design strength, to develop the cover plate's portion of the design strength in the beam or girder at the distance a' from the end of the cover plate.

(a) When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate

$$a' = w \qquad \textbf{(2.10-4)}$$

where:

w = width of cover plate, mm

(b) When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate

$$a' = 1.5w \qquad \textbf{(2.10-5)}$$

(c) When there is no weld across the end of the plate

$$a' = 2w \qquad \textbf{(2.10-6)}$$

CHAPTER 3
FRAMES AND OTHER STRUCTURES

This chapter contains general requirements for stability of the structure as a whole.

SECTION 3.1
SECOND ORDER EFFECTS

Second order ($P\Delta$) effects shall be considered in the design of frames.

3.1.1 **Design by Plastic Analysis.** In structures designed on the basis of plastic analysis, as limited in Section 1.5.1, the required flexural strength M_u shall be determined from a second-order plastic analysis that satisfies the requirements of Section 3.2.

3.1.2 **Design by Elastic Analysis.** In structures designed on the basis of elastic analysis, M_u for beam-columns, connections, and connected members shall be determined from a second-order elastic analysis or from the following approximate second-order analysis procedure:

$$M_u = B_1 M_{nt} + B_2 M_{lt} \qquad \textbf{(3.1-1)}$$

where

M_{nt} = required flexural strength in member assuming there is no lateral translation of the frame, N.mm

M_{lt} = required flexural strength in member as a result of lateral translation of the frame only, N.mm

$$B_1 = \frac{C_m}{(1 - P_u / P_{e1})} \geq 1 \qquad \textbf{(3.1-2)}$$

$$P_{e1} = \frac{\pi^2 EI}{(KL)^2}, \text{ N}$$

where I is the moment of inertia in the plane of bending and K is the effective length factor in the plane of bending determined in accordance with Section 3.2.1, for the braced frame.

P_u = required axial compressive strength for the member under consideration, N

C_m = a coefficient based on elastic first-order analysis assuming no lateral translation of the frame whose value shall be taken as follows:

(a) For compression members not subject to transverse loading between their supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1 / M_2) \qquad \textbf{(3.1-3)}$$

where M_1 / M_2 is the ratio of the smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1 / M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature.

(b) For compression members subjected to transverse loading between their supports, the value of C_m shall be determined either by rational analysis (see commentary) or by the use of the following values:

For members whose ends are restrained. . . . . . . $C_m = 0.85$

For members whose ends are unrestrained. . . . . . $C_m = 1.00$

$$B_2 = \frac{1}{1 - \Sigma P_u \left(\frac{\Delta_{oh}}{\Sigma HL} \right)} \quad \textbf{(3.1-4)}$$

or

$$B_2 = \frac{1}{1 - \left(\frac{\Sigma P_u}{\Sigma P_{e2}} \right)} \quad \textbf{(3.1-5)}$$

$\sum P_u$ = required axial strength of all columns in a story, N

Δ_{oh} = lateral inter-story deflection, mm

$\sum H$ = sum of all story horizontal forces producing Δ_{oh}, N

L = story height, mm

P_{e_2} = $\frac{\pi^2 EI}{(KL)^2}$, N

where I is the moment of inertia in the plane of bending and K is the effective length factor in the plane of bending determined in accordance with Section 3.2.2, for the unbraced frame.

SECTION 3.2
FRAME STABILITY

3.2.1 **Braced Frames.** In trusses and frames where lateral stability is provided by diagonal bracing, shear walls, or equivalent means, the effective length factor K for compression members shall be taken as unity, unless structural analysis shows that a smaller value may be used.

The vertical bracing system for a braced multistory frame shall be determined by structural analysis to be adequate to prevent buckling of the structure and to maintain the lateral stability of the structure, including the overturning effects of drift under the factored load combinations stipulated in Section 1.4.

The vertical bracing system for a braced multistory frame may be considered to function together with in-plane shear-resisting exterior and interior walls, floor slabs, and roof decks, which are properly secured to the structural frames. The columns, girders, beams, and diagonal members, when used as the vertical bracing system, may be considered to comprise a vertically cantilevered simply connected truss in the analyses for frame buckling and lateral stability. Axial deformation of all members in the vertical bracing system shall be included in the lateral stability analysis.

3.2.1.1 **Design by Plastic Analysis.** In braced frames designed on the basis of plastic analysis, as limited in Section 1.5.1, the axial force in these members caused by factored gravity plus factored horizontal loads shall not exceed 0.85 ϕ_c times A_gF_y.

3.2.2 **Unbraced Frames.** In frames where lateral stability depends upon the bending stiffness of rigidly connected beams and columns, the effective length factor K of compression members shall be determined by structural analysis. The destabilizing effects of gravity loaded columns whose simple connections to the frame do not provide resistance to lateral loads shall be included in the design of the moment-frame columns. Stiffness reduction adjustment due to column inelasticity is permitted.

Analysis of the required strength of unbraced multistory frames shall include the effects of frame instability and column axial deformation under the factored load combinations stipulated in Section 1.4.

3.2.2.1 **Design by Plastic Analysis.** In unbraced frames designed on the basis of plastic analysis, as limited in Section 1.5.1, the axial force in the columns caused by factored gravity plus factored horizontal loads shall not exceed 0.75 ϕ_c times $A_g F_y$.

SECTION 3.3
STABILITY BRACING

3.3.1 **Scope.** These requirements address the minimum brace strength and stiffness necessary to ensure member design strengths based on the unbraced length between braces with an effective length factor K equal to unity. Bracing is assumed to be perpendicular to the member(s) to be braced; for inclined or diagonal bracing, the brace strength (force or moment) and stiffness (force per unit displacement or moment per unit rotation) must be adjusted for the angle of inclination. The evaluation of the stiffness furnished by a brace shall include its member and geometric properties, as well as the effects of connections and anchoring details.

Two general types of bracing systems are considered, relative and nodal. A relative brace controls the movement of the brace point with respect to adjacent braced points. A nodal brace controls the movement at the braced point without direct interaction with adjacent braced points. The strength and stiffness furnished by the stability bracing shall not be less than the required limits. A second order analysis that includes an initial out-of-plumbness of the structure or out-of-straightness of the member to obtain brace strength and stiffness can be used in lieu of the requirements of this section.

3.3.2 **Frames.** In braced frames where lateral stability is provided by diagonal bracing, shear walls, or other equivalent means, the required story or panel bracing shear force is:

$$P_{br} = 0.004\Sigma P_u \qquad \textbf{(3.3-1)}$$

The required story or panel shear stiffness is:

$$\beta_{br} = \frac{2\Sigma P_u}{\phi L} \qquad \textbf{(3.3-2)}$$

where

ϕ = 0.75

ΣP_u = summation of the factored column axial loads in the story or panel supported by the bracing, N

L = story height or panel spacing, mm

These story stability requirements shall be combined with the lateral forces and drift requirements from other sources, such as wind or seismic loading.

3.3.3 **Columns.** An individual column can be braced at intermediate points along its length by relative or nodal bracing systems. It is assumed that nodal braces are equally spaced along the column.

(a) *Relative Bracing*

The required brace strength is:

$$P_{br} = 0.004P_u \quad \textbf{(3.3-3)}$$

The required brace stiffness is:

$$\beta_{br} = \frac{2P_u}{\phi L_b} \quad \textbf{(3.3-4)}$$

where

ϕ = 0.75

P_u = required compressive strength, N

L_b = distance between braces, mm

(b) *Nodal Bracing*

The required brace strength is:

$$P_{br} = 0.01P_u \quad \textbf{(3.3-5)}$$

The required brace stiffness is:

$$\beta_{br} = \frac{8P_u}{\phi L_b} \quad \textbf{(3.3-6)}$$

where

ϕ = 0.75

When the actual spacing of braced points is less than L_q, where L_q is the maximum unbraced length for the required column force with K equal to one, then L_b in Equations 3.3-4 and 3.3-6 is permitted to be taken equal to L_q.

3.3.4 **Beams.** Beam bracing must prevent the relative displacement of the top and bottom flanges, i.e. twist of the section. Lateral stability of beams shall be provided by lateral bracing, torsional bracing, or a combination of the two. In members subjected to double curvature bending, the inflection point shall not be considered a brace point.

3.3.4.1 **Lateral Bracing.** Bracing shall be attached near the compression flange, except for a cantilevered member, where an end brace shall be attached near the top (tension) flange. Lateral bracing shall be attached to both flanges at the brace point near the inflection point for beams subjected to double curvature bending along the length to be braced.

(a) *Relative Bracing*

The required brace strength is:

$$P_{br} = 0.008M_u C_d / h_o \quad \textbf{(3.3-7)}$$

The required brace stiffness is:

$$\beta_{br} = \frac{4M_u C_d}{\phi L_b h_o} \quad \textbf{(3.3-8)}$$

where

ϕ = 0.75

M_u = required flexural strength, N.mm

h_o = distance between flange centroids, mm

C_d = 1.0 for bending in single curvature; 2.0 for double curvature;

C_d = 2.0 only applies to the brace closest to the inflection point.

L_b = distance between braces, mm

(b) *Nodal Bracing*

The required brace strength is:

$$P_{br} = 0.02 M_u C_d / h_o \tag{3.3-9}$$

The required brace stiffness is:

$$\beta_{br} = \frac{10 M_u C_d}{\phi L_b h_o} \tag{3.3-10}$$

where

$\phi = 0.75$

When the actual spacing of braced points is less than L_q, the maximum unbraced length for M_u, then L_b in Equations 3.3-8 and 3.3-10 shall be permitted to be taken equal to L_q.

3.3.4.2 **Torsional Bracing.** Torsional bracing can be nodal or continuous along the beam length. The bracing can be attached at any cross-sectional location and need not be attached near the compression flange. The connection between a torsional brace and the beam must be able to support the required moment given below.

(a) *Nodal Bracing*

The required bracing moment is:

$$M_{br} = \frac{0.024 M_u L}{n C_b L_b} \tag{3.3-11}$$

The required cross-frame or diaphragm bracing stiffness is:

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \dfrac{\beta_T}{\beta_{sec}}\right)} \tag{3.3-12}$$

where

$$\beta_T = \frac{2.4 L M_u^2}{\phi n E I_y C_b^2} \tag{3.3-13}$$

$$\beta_{sec} = \frac{3.3E}{h_o}\left(\frac{1.5 h_o t_w^3}{12} + \frac{t_s b_s^3}{12}\right) \tag{3.3-14}$$

ϕ = 0.75

L = span length, mm

n = number of nodal braced points within the span

E = 200,000 MPa

I_y = out-of-plane moment of inertia, mm^4

C_b = is a modification factor defined in Chapter 6

t_w = beam web thickness, mm

t_s = web stiffener thickness, mm

b_s = stiffener width for one-sided stiffeners (use twice the individual stiffener width for pairs of stiffeners), mm

β_T = brace stiffness excluding web distortion, N-mm/radian

β_{sec} = web distortional stiffness, including the effect of web transverse stiffeners, if any, N-mm/radian

If $\beta_{sec} < \beta_T$, Equation 3.3-12 is negative, which indicates that torsional beam bracing will not be effective due to inadequate web distortional stiffness.

When required, the web stiffener shall extend the full depth of the braced member and shall be attached to the flange if the torsional brace is also attached to the flange. Alternatively, it shall be permissible to stop the stiffener short by a distance equal to $4t_w$ from any beam flange that is not directly attached to the torsional brace. When the actual spacing of braced points is less than L_q, then L_b in Equation 3.3-11 shall be permitted to be taken equal to L_q.

(b) *Continuous Torsional Bracing*

For continuous bracing, use Equations 3.3-11, 3.3-12 and 3.3-13 with L/n taken as 1.0; the bracing moment and stiffness are given per unit span length. The distortional stiffness for an unstiffened web is

$$\beta_{sec} = \frac{3.3Et_w^3}{12h_o} \qquad \textbf{(3.3-15)}$$

CHAPTER 4
TENSION MEMBERS

This chapter applies to prismatic members subject to axial tension caused by static forces acting through the centroidal axis. For members subject to combined axial tension and flexure, see Section 8.1.1. For threaded rods, see Section 10.3. For block shear rupture strength at end connections of tension members, see Section 10.4.3. For the design tensile strength of connecting elements, see Section 10.5.2. For members subject to fatigue, see Section 11.3.

SECTION 4.1
DESIGN TENSILE STRENGTH

The design strength of tension members $\phi_t P_n$, shall be the lower value obtained according to the limit states of yielding in the gross section and fracture in the net section.

(a) For yielding in the gross section:

$$\phi_t = 0.90$$

$$P_n = F_y A_g \qquad \textbf{(4.1-1)}$$

(b) For fracture in the net section:

$$\phi_t = 0.75$$

$$P_n = F_u A_e \qquad \textbf{(4.1-2)}$$

where

A_e = effective net area, mm^2

A_g = gross area of member, mm^2

F_y = specified minimum yield stress, MPa

F_u = specified minimum tensile strength, MPa

When members without holes are fully connected by welds, the effective net section used in Equation 4.1-2 shall be defined as Section 2.3. When holes are present in a member with welded-end connections, or at the welded connection in the case of plug or slot welds, the net section through the holes shall be used in Equation 4.1-2.

SECTION 4.2
BUILT-UP MEMBERS

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section 10.3.5.

The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300.

Either perforated cover plates or tie plates without lacing are permitted to be used on the open sides of built-up tension members. Tie plates shall have a length not less than two-thirds the distance between the lines of welds or fasteners connecting them to the components of the member. The thickness of such tie plates shall not be less than one-fiftieth of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed 150 mm. The spacing of tie plates shall be such that the slenderness ratio of any component in the length between tie plates should preferably not exceed 300.

SECTION 4.3
PIN-CONNECTED MEMBERS AND EYEBARS

4.3.1 Pin-Connected Members

4.3.1.1 Design Strength

The design strength of a pin-connected member, ϕP_n shall be the lowest value of the following limit states:

(a) Tension on the net effective area:

$$\phi = 0.75$$

$$P_n = 2tb_{eff}F_u \quad \textbf{(4.3-1)}$$

(b) Shear on the effective area:

$$\phi = 0.75$$

$$P_n = 0.6A_{sf}F_u \quad \textbf{(4.3-2)}$$

(c) For bearing on the projected area of the pin, see Section 10.8.

(d) For yielding in the gross section, use Equation 4.1-1.

where

A_{sf} = $2t(a+d/2)$, mm^2

a = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the force, mm

b_{eff} = $2t+16$, mm but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force

d = pin diameter, mm

t = thickness of plate, mm

4.3.1.2 Detailing Requirements. The pin hole shall be located midway between the edges of the member in the direction normal to the applied force. When the pin is expected to provide for relative movement between connected parts while under full load, the diameter of the pin hole shall not be more than 1 mm greater than the diameter of the pin.

The width of the plate beyond the pin hole shall not be less than $2b_{eff}+d$ and the minimum extension, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than 1.33 x b_{eff}.

The corners beyond the pinhole are permitted to be cut at 45^o to the axis of the member, provided the net area beyond the pinhole, on a plane perpendicular to the cut, is not less than that required beyond the pinhole parallel to the axis of the member.

4.3.2 Eyebars

4.3.2.1 Design Strength. The design strength of eyebars shall be determined in accordance with 4.1, with A_g taken as the cross-sectional area of the body.
For calculation purposes, the width of the body of the eyebars shall not exceed eight times its thickness.

4.3.2.2 **Detailing Requirements.** Eyebars shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads whose periphery is concentric with the pin hole.

The radius of transition between the circular head and the eyebar body shall not be less than the head diameter.
The pin diameter shall not be less than seven-eighths times the eyebar body width, and the pin hole diameter shall not be more than 1 mm greater than the pin diameter.

For steels having F_y greater than 485 MPa, the hole diameter shall not exceed five times the plate thickness and the width of the eyebar body shall be reduced accordingly.

A thickness of less than 12 mm is permissible only if external nuts are provided to tighten pin plates and filler plates into snug contact. The width from the hole edge to the plate edge perpendicular to the direction of applied load shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

CHAPTER 5
COLUMN AND OTHER COMPRESSION MEMBERS

This chapter applies to compact and non-compact prismatic members subject to axial compression through the centroidal axis. For members subject to combined axial compression and flexure, see Section 8.1.2. For members with slender compression elements, see Section 2.5.3. For tapered members, see Section 6.3.

SECTION 5.1
EFFECTIVE LENGTH AND SLENDERNESS LIMITATIONS

5.1.1 **Effective Length.** The effective length factor K shall be determined in accordance with Section 3.2.

5.1.2 **Design by Plastic Analysis.** Design by plastic analysis, as limited in Section 1.5.1, is permitted if the column slenderness parameter λ_c does not exceed 1.5K.

SECTION 5.2
DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING

5.2.1 **Width-Thickness Ratio of Elements Less than or Equal to λ_r.** The design strength for flexural buckling of compression members whose elements have width-thickness ratios less than or equal to λ_r from Section 2.5.1 is $\phi_c P_n$:

$$\phi_c = 0.85$$

$$P_n = A_g F_{cr} \qquad \textbf{(5.2-1)}$$

(a) For $\lambda_c \le 1.5$

$$F_{cr} = \left(0.658^{\lambda_c^2}\right) F_y \qquad \textbf{(5.2-2)}$$

(b) For $\lambda_c > 1.5$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y \qquad \textbf{(5.2-3)}$$

where

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} \qquad \textbf{(5.2-4)}$$

A_g = gross area of member, mm^2
F_y = specified minimum yield stress, MPa
E = modulus of elasticity, MPa
K = effective length factor
l = laterally unbraced length of member, mm
r = governing radius of gyration about the axis of buckling, mm

5.2.2 **Width-Thickness Ratio of Elements Exceeds λ_r**

For members whose elements do not meet the requirements of Section 2.5.1, see 2.5.3.

SECTION 5.3
DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL-TORSIONAL BUCKLING

5.3.1 **Width-Thickness Ratios of Elements Less than or Equal to λ_r.** The design strength for flexural-torsional buckling of double-angle and tee-shaped compression members whose elements have width-thickness ratios less than λ_r from Section 2.5.1 is $\phi_c P_n$:

where

$$\phi_c = 0.85$$

$$P_n = A_g F_{crft} \quad \textbf{(5.3-1)}$$

$$F_{crft} = \left(\frac{F_{cry} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}}\right] \quad \textbf{(5.3-2)}$$

$F_{crz} = \dfrac{GJ}{A\bar{r}_o^2}$

$\bar{r}_o$ = polar radius of gyration about shear center, mm (see Equation 5.3-10)

$H = 1 - \dfrac{y_o^2}{\bar{r}_o^2}$

y_o = distance between shear center and centroid, mm

F_{cry} = is determined according to Section 5.2 for flexural buckling about the y-axis of symmetry for $\lambda_c = \dfrac{Kl}{r_y \pi}\sqrt{\dfrac{F_y}{E}}$.

For double-angle and tee-shaped members whose elements do not meet the requirements of Section 2.5.1, see 2.5.3 to determine F_{cry} for use in Equation 5.3-2.

5.3.2 **Width-Thickness Ratio of Elements Exceeds λ_r.** This section applies to the strength of doubly symmetric columns with thin plate elements, and singly symmetric and unsymmetric columns for the limit states of flexural-torsional and torsional buckling.

The design strength of compression members determined by the limit states of torsional and flexural-torsional buckling is $\phi_c P_n$,

where

ϕ_c = 0.85

P_n = nominal resistance in compression, N

= $A_g F_{cr}$ **(5.3-3)**

A_g = gross area of cross section, mm^2

The nominal critical stress F_{cr} is determined as follows:

(a) For $\lambda_e \sqrt{Q} \le 1.5$:

$$F_{cr} = Q(0.658^{Q\lambda_e^2}) F_y \quad \textbf{(5.3-4)}$$

(b) For $\lambda_e \sqrt{Q} > 1.5$:

$$F_{cr} = \left[\frac{0.877}{\lambda_e^2}\right] F_y \qquad \textbf{(5.3-5)}$$

where

$$\lambda_e = \sqrt{F_y / F_e} \qquad \textbf{(5.3-6)}$$

Q = 1.0 for elements meeting the width-thickness ratios λ_r of Section 2.5.1

= $Q_s Q_a$ for elements not meeting the width-thickness ratios λ_r of Section 2.5.1 and determined in accordance with the provisions of Section 2.5.3

The critical torsional or flexural-torsional elastic buckling stress F_e is determined as follows:

(a) For doubly symmetric shapes:

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z l)^2} + GJ\right] \frac{1}{I_x + I_y} \qquad \textbf{(5.3-7)}$$

(b) For singly symmetric shapes where y is the axis of symmetry:

$$F_e = \frac{F_{ey} + F_{ez}}{2H}\left(1 - \sqrt{1 - \frac{4 F_{ey} F_{ez} H}{(F_{ey} + F_{ez})^2}}\right) \qquad \textbf{(5.3-8)}$$

(c) For unsymmetric shapes, the critical flexural-torsional elastic buckling stress F_e is the lowest root of the cubic equation

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2 (F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2 (F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \qquad \textbf{(5.3-9)}$$

where

K_z = effective length factor for torsional buckling

G = shear modulus, MPa

C_w = warping constant, mm^6

J = torsional constant, mm^4

I_x, I_y = moment of inertia about the principal axes, mm^4

x_o, y_o = coordinates of shear center with respect to the centroid, mm

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A} \qquad \textbf{(5.3-10)}$$

$$H = 1 - \left(\frac{x_o^2 + y_o^2}{\bar{r}_o^2}\right) \qquad \textbf{(5.3-11)}$$

$$F_{ex} = \frac{\pi^2 E}{(K_x l / r_x)^2} \qquad \textbf{(5.3-12)}$$

$$F_{ey} = \frac{\pi^2 E}{(K_y l / r_y)^2} \qquad \textbf{(5.3-13)}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z l)^2} + GJ\right)\frac{1}{A \bar{r}_o^2} \qquad \textbf{(5.3-14)}$$

A = cross-sectional area of member, mm^2

l = unbraced length, mm

K_x , K_y = effective length factors in x and y directions
r_x , r_y = radii of gyration about the principal axes, mm
$\bar{r}_o$ = polar radius of gyration about the shear center, mm

SECTION 5.4
BUILT-UP MEMBERS

5.4.1 **Design Strength.** The design strength of built-up members composed of two or more shapes shall be determined in accordance with Section 5.2 and Section 5.3 subject to the following modification. If the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, Kl/r is replaced by $(Kl/r)_m$ determined as follows:

(a) For intermediate connectors that are snug-tight bolted:

$$\left(\frac{Kl}{r}\right)_m = \sqrt{\left(\frac{Kl}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \textbf{(5.4-1)}$$

(b) For intermediate connectors that are welded or fully tensioned bolted:

$$\left(\frac{Kl}{r}\right)_m = \sqrt{\left(\frac{Kl}{r}\right)_o^2 + 0.82\frac{\alpha^2}{(1+\alpha^2)}\left(\frac{a}{r_{ib}}\right)^2} \qquad \textbf{(5.4-2)}$$

where

$\left(\frac{Kl}{r}\right)_o$ = column slenderness of built-up member acting as a unit

$\left(\frac{Kl}{r}\right)_m$ = modified column slenderness of built-up member

a = distance between connectors, mm

r_i = minimum radius of gyration of individual component, mm

r_{ib} = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, mm

α = separation ratio = $h/2r_{ib}$

h = distance between centroids of individual components perpendicular to the member axis of buckling, mm

5.4.2 **Detailing Requirements.** At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1½ times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds, bolts, or rivets shall be adequate to provide for the transfer of the required forces. For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section 10.3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $0.75\sqrt{E/F_y}$, nor 305 mm, when intermittent welds are provided along the edges of the components or when fasteners are provided on all gage lines at each section.

When fasteners are staggered, the maximum spacing on each gage line shall not exceed the thickness of the thinner outside plate times $1.12\sqrt{E/F_y}$ nor 460 mm.

Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, a, such that the effective slenderness ratio Ka/r_i of each of the component shapes, between the connectors, does not exceed three-fourths times the governing slenderness ratio of the built-up member. The least radius of gyration r_i shall be used in computing the slenderness ratio of each component part. The end connection shall be welded or fully tensioned bolted with clean mill scale or blast-cleaned faying surfaces with Class A coatings.

Open sides of compression members built up from plates or shapes shall be provided with continuous cover plates perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Section 2.5.1, is assumed to contribute to the design strength provided that:

(1) The width-thickness ratio conforms to the limitations of Section 2.5.1.

(2) The ratio of length (in direction of stress) to width of hole shall not exceed two.

(3) The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.

(4) The periphery of the holes at all points shall have a minimum radius of 38 mm.

As an alternative to perforated cover plates, lacing with tie plates is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In main members providing design strength, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall aggregate not less than one-third the length of the plate. In bolted and riveted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, shall be so spaced that l/r of the flange included between their connections shall not exceed the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to two percent of the compressive design strength of the member. The l/r ratio for lacing bars arranged in single systems shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. For lacing bars in compression, l is permitted to be taken as the unsupported length of the lacing bar between welds or fasteners connecting it to the components of the built-up member for single lacing, and 70 percent of that distance for double lacing. The inclination of lacing bars to the axis of the member shall preferably be not less than 60^o for single lacing and 45^o for double lacing. When the distance between the lines of welds or fasteners in the flanges is more than 380 mm, the lacing shall preferably be double or be made of angles.

For additional spacing requirements, see Section 10.3.

SECTION 5.5
CONNECTIONS FOR PIN-CONNECTED COMPRESSION MEMBERS

Pin connections of pin-connected compression members shall conform to the requirements of Sections 4.3.1 and 4.3.2, except Equations 4.3-1 and 4.3-2 do not apply.

CHAPTER 6
BEAMS AND OTHER FLEXURAL MEMBERS

This chapter applies to compact and noncompact prismatic members subject to flexure and shear. For member subject to combined flexure and axial force, see Section 8.1. For members subject to fatigue, see Section 11.3. For members with slender compression elements, see Section 2.5. For web-tapered members, see Section 6.3. For members with slender web elements (plate girders), see Chapter 7.

SECTION 6.1
DESIGN FOR FLEXURE

The nominal flexural strength M_n is the lowest value obtained according to the limit states of: (a) yielding; (b) lateral-torsional buckling; (c) flange local buckling; and (d) web local buckling. For laterally braced compact beams with $L_b \leq L_p$, only the limit state of yielding is applicable. For unbraced compact beams and noncompact tees and double angles, only the limit states of yielding and lateral-torsional buckling are applicable. The lateral-torsional buckling limit state is not applicable to members subject to bending about the minor axis, or to square or circular shapes.

This section applies to homogeneous and hybrid shapes with at least one axis of symmetry and which are subject to simple bending about one principal axis. For simple bending, the beam is loaded in a plane parallel to a principal axis that passes through the shear center or the beam is restrained against twisting at load points and supports. Only the limit states of yielding and lateral-torsional buckling are considered in this section. The lateral-torsional buckling provisions are limited to doubly symmetric shapes, channels, double angles, and tees. For lateral-torsional buckling of other singly symmetric shapes and for the limit states of flange local buckling and web local buckling of noncompact or slender-element sections, see Section 6.1.2.4 of this Chapter.

6.1.1 **Yielding.** The flexural design strength of beams, determined by the limit state of yielding, is $\phi_b M_n$:

$$\phi_b = 0.90$$

$$M_n = M_p \qquad \textbf{(6.1-1)}$$

where

M_p = plastic moment ($= F_y Z \leq 1.5\, M_y$ for homogeneous sections), N-mm

M_y = moment corresponding to onset of yielding at the extreme fiber from an elastic stress distribution ($= F_y S$ for homogeneous section and $F_{yf} S$ for hybrid sections), N-mm

See Section 2.10 for further limitations on M_n where there are holes in the tension flange.

6.1.2 **Lateral-Torsional Buckling.** This limit state is only applicable to members subject to major axis bending. The flexural design strength, determined by the limit state of lateral-torsional buckling, is $\phi_b M_n$:

$\phi_b =$ 0.90

$M_n =$ nominal flexural strength determined as follows

6.1.2.1 Doubly Symmetric Shapes and Channels with $L_b \leq L_r$. The nominal flexural strength is:

$$M_n = C_b\left[M_p - \left(M_p - M_r\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \quad \textbf{(6.1-2)}$$

where

L_b = distance between points braced against lateral displacement of the compression flange, or between points braced to prevent twist of the cross section, mm

L_p = limiting laterally unbraced length as defined below, mm

L_r = limiting laterally unbraced length as defined below, mm

M_r = limiting buckling moment as defined below, N-mm

In the above equation, C_b is a modification factor for non-uniform moment diagrams where, when both ends of the beam segment are braced:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad \textbf{(6.1-3)}$$

where

$M_{\max}$ = absolute value of maximum moment in the unbraced segment, N-mm

M_A = absolute value of moment at quarter point of the unbraced segment, N-mm

M_B = absolute value of moment at centerline of the unbraced beam segment, N-mm

M_C = absolute value of moment at three-quarter point of the unbraced beam segment, N-mm

C_b = is permitted to be conservatively taken as 1.0 for all cases. Equations 6.1-4 and 6.1-6 are conservatively based on C_b = 1.0. For cantilevers or overhangs where the free end is unbraced, C_b = 1.0.

The limiting unbraced length, L_p, shall be determined as follows.

(a) For I-shaped members including hybrid sections and channels:

$$L_p = 1.76r_y\sqrt{\frac{E}{F_{yf}}} \quad \textbf{(6.1-4)}$$

(b) For solid rectangular bars and box sections:

$$L_p = \frac{0.13r_yE}{M_p}\sqrt{JA} \quad \textbf{(6.1-5)}$$

where

A = cross-sectional area, mm^2

J = torsional constant, mm^4

The limiting laterally unbraced length L_r and the corresponding buckling moment M_r shall be determined as follows.

(a) For doubly symmetric I-shaped members and channels:

$$L_r = \frac{r_y X_1}{F_L}\sqrt{1+\sqrt{1+X_2 F_L^2}} \qquad \textbf{(6.1-6)}$$

$$M_r = F_L S_x \qquad \textbf{(6.1-7)}$$

where

$$X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}} \qquad \textbf{(6.1-8)}$$

$$X_2 = 4\frac{C_w}{I_y}\left(\frac{S_x}{GJ}\right)^2 \qquad \textbf{(6.1-9)}$$

S_x = section modulus about major axis, mm^3
E = modulus of elasticity of steel, 200,000 MPa
G = shear modulus of elasticity of steel, 77,200 MPa
F_L = smaller of (F_{yf} - F_r) or F_{yw}, MPa
F_r = compressive residual stress in flange; 69 MPa for rolled shapes, 114 MPa for welded built-up shapes
F_{yf} = yield stress of flange, MPa
F_{yw} = yield stress of web, MPa
I_y = moment of inertia about y-axis, mm^4
C_w = warping constant, mm^6

(b) For solid rectangular bars and box sections:

$$L_r = \frac{2 r_y E\sqrt{JA}}{M_r} \qquad \textbf{(6.1-10)}$$

$$M_r = F_{yf} S_x \qquad \textbf{(6.1-11)}$$

6.1.2.2 **Doubly Symmetric Shapes and Channels with $L_b > L_r$**

The nominal flexural strength is:

$$M_n = M_{cr} \le M_p \qquad \textbf{(6.1-12)}$$

where

(a) For doubly symmetric I-shaped members and channels:

$$\begin{aligned} M_{cr} &= C_b \frac{\pi}{L_b}\sqrt{EI_y GJ + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w} \\ &= \frac{C_b S_x X_1\sqrt{2}}{L_b / r_y} 1 + \frac{X_1^2 X_2}{2(L_b / r_y)^2} \end{aligned} \qquad \textbf{(6.1-13)}$$

(b) For solid rectangular bars and symmetric box sections:

$$M_{cr} = \frac{57000 C_b \sqrt{JA}}{L_b / r_y} \qquad \textbf{(6.1-14)}$$

6.1.2.3 **Tees and Double Angles**. For tees and double-angle beams loaded in the plane of symmetry:

$$M_n = M_{cr} = \frac{\pi \sqrt{E I_y G J}}{L_b}\left[B + \sqrt{1 + B^2}\right] \qquad \textbf{(6.1-15)}$$

where

$M_n \leq 1.5 M_y$ for stems in tension

$M_n \leq 1.0 M_y$ for stems in compression

$$B = \pm 2.3 (d / L_b) \sqrt{I_y / J} \qquad \textbf{(6.1-16)}$$

The plus sign for B applies when the stem is in tension and the minus sign applies when the stem is in compression. If the tip of the stem is in compression anywhere along the unbraced length, use the negative value of B.

6.1.2.4 Yielding, lateral-torsional buckling, flange local buckling, and web local buckling of singly symmetric shapes other than those addressed in Sections 6.1.1 and 6.1.2 of this Chapter and of noncompact or slender-element sections:

The nominal flexural strength M_n is the lowest value obtained according to the following limit states determined as follows:

(a) For $\lambda \leq \lambda_p$:

$$M_n = M_p \qquad \textbf{(6.1-17)}$$

(b) For $\lambda_p < \lambda \leq \lambda_r$:

For the limit state of lateral-torsional buckling:

$$M_n = C_b \left[M_p - (M_p - M_r) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \right] \leq M_p \qquad \textbf{(6.1-18)}$$

For the limit states of flange and web local buckling:

$$M_n = M_p - (M_p - M_r) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \qquad \textbf{(6.1-19)}$$

(c) For $\lambda > \lambda_r$:

For the limit states of lateral-torsional buckling and flange local buckling:

$$M_n = M_{cr} = S F_{cr} \leq M_p \qquad \textbf{(6.1-20)}$$

The terms used in the above equations are:

M_n = nominal flexural strength, N-mm

M_p = $F_y Z$, plastic moment $\leq 1.5\, F_y S$, N-mm

M_{cr} = buckling moment, N-mm

M_r = limiting buckling moment (equal to M_{cr} when $\lambda = \lambda_r$), N-mm

λ = controlling slenderness parameter

= minor axis slenderness ratio L_b / r_y for lateral-torsional buckling

= flange width-thickness ratio b / t for flange local buckling as defined in Section 2.5.1

= web depth-thickness ratio h / t_w for web local buckling as defined in Section 2.5.1

λ_p = largest value of λ for which $M_n = M_p$

λ_r = largest value of λ for which buckling is inelastic

F_{cr} = critical stress, MPa

C_b = bending coefficient dependent on moment gradient, see Section 6.1.2.1, Equation 6.1-3

S = section modulus, mm^3

L_b = laterally unbraced length, mm

r_y = radius of gyration about minor axis, mm

6.1.3 **Design by Plastic Analysis.** Design by plastic analysis, as limited in Section 1.5.1, is permitted for a compact section member bent about the major axis when the laterally unbraced length L_b of the compression flange adjacent to plastic hinge locations associated with the failure mechanism does not exceed L_{pd}, determined as follows.

(a) For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange (including hybrid members) loaded in the plane of the web:

$$L_{pd} = \left[0.12 + 0.076\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y \qquad \textbf{(6.1-21)}$$

where

F_y = specified minimum yield stress of the compression flange, MPa

M_1 = smaller moment at end of unbraced length of beam, N-mm

M_2 = larger moment at end of unbraced length of beam, N-mm

r_y = radius of gyration about minor axis, mm

(M_1 / M_2) = is positive when moments cause reverse curvature and negative for single curvature

(b) For solid rectangular bars and symmetric box beams:

$$L_{pd} = \left[0.17 + 0.10\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y \geq 0.10\left(\frac{E}{F_y}\right) r_y \qquad \textbf{(6.1-22)}$$

There is no limit on L_b for members with circular or square cross sections or for any beam bent about its minor axis.

In the region of the last hinge to form, and in regions not adjacent to a plastic hinge, the flexural design strength shall be determined in accordance with Section 6.1.2.

SECTION 6.2
DESIGN FOR SHEAR

This section applies to stiffened or un-stiffened webs of singly or doubly symmetric beams, including hybrid beams, and channels subject to shear in the plane of the web. For shear in the weak direction of the shapes above, pipes, and unsymmetric sections, see Section 8.2. For web panels subject to high shear, see Section 11.1.7. For shear strength at connections, see Sections 10.4 and 10.5.

6.2.1 Web Area Determination

The web area A_w shall be taken as the overall depth d times the web thickness t_w.

6.2.2 Design Shear Strength

The design shear strength of stiffened or un-stiffened webs is $\phi_v V_n$,

where

ϕ_v = 0.90

V_n = nominal shear strength defined as follows.

(a) For $h/t_w \le 1.1\sqrt{k_v E/F_{yw}}$:

$$V_n = 0.6F_{yw}A_w \quad \textbf{(6.2-1)}$$

(b) For $1.10\sqrt{k_v E/F_{yw}} < h/t_w \le 1.37\sqrt{k_v E/F_{yw}}$:

$$V_n = 0.6F_{yw}A_w(1.10\sqrt{k_v E/F_{yw}})/(h/t_w) \quad \textbf{(6.2-2)}$$

(c) For $h/t_w > 1.37\sqrt{k_v E/F_{yw}}$:

$$V_n = A_w(0.91Ek_v)/(h/t_w)^2 \quad \textbf{(6.2-3)}$$

where

$k_v = 5 + 5/(a/h^2)$

= 5 when a/h >3 or a/h> $[260/(h/t)]^2$

a = distance between transverse stiffeners, mm

h = for rolled shapes, the clear distance between flanges less the fillet or corner radius, mm

= for built-up welded sections, the clear distance between flanges, mm

= for built-up bolted or riveted sections, the distance between fastener lines, mm

6.2.3 **Transverse Stiffeners.** Transverse stiffeners are not needed where the required shear, V_u, as determined by structural analysis for the factored loads, is less than or equal to $0.6\phi_v A_w F_{yw} C_v$, where $\phi_v = 0.90$ and the shear coefficient C_v, defined in Section 7.3, is determined for $k_v = 5$.

Transverse stiffeners used to develop the web design shear strength as provided in Section 6.2.2 shall have a moment of inertia about an axis in the web center for stiffener pairs, or about the face in contact with the web plate for single stiffeners, which shall not be less than $at_w^3 j$, where

$$j = 2.5/(a/h)^2 - 2 \geq 0.5 \qquad \textbf{(6.2-4)}$$

Intermediate stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which intermediate stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe of the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange. When lateral bracing is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit one percent of the total flange stress, unless the flange is composed only of angles. Bolts connecting stiffeners to the girder web shall be spaced not more than 300 mm on center. If intermittent fillet welds are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 250 mm.

SECTION 6.3
WEB-TAPERED MEMBERS

The design of tapered members meeting the requirements of this section shall be governed by the provisions of Chapters 4 through 8, except as modified by the following provisions:

6.3.1 **General Requirements.** In order to qualify under this specification, a tapered member shall meet the following requirements:

(1) It shall possess at least one axis of symmetry, which shall be perpendicular to the plane of bending if moments are present.

(2) The flanges shall be of equal and constant area.

(3) The depth shall vary linearly as

$$d = d_o\left(1 + \gamma \frac{z}{L}\right) \qquad \textbf{(6.3-1)}$$

where

γ = $(d_L - d_o)/d_o \leq$ the smaller of $0.268(L/d_o)$ or 6.0

d_o = depth at smaller end of member, mm

d_L = depth at larger end of member, mm

z = distance from the smaller end of member, mm

L = unbraced length of member measured between the center of gravity of the bracing members, mm

6.3.2 **Design Tensile Strength.** The design strength of tapered tension members shall be determined in accordance with Section 4.1.

6.3.3 **Design Compressive Strength.** The design strength of tapered compression members shall be determined in accordance with Section 5.3, using an effective slenderness parameter λ_{eff} computed as follows:

$$\lambda_{eff} = \frac{S}{\pi}\sqrt{\frac{QF_y}{E}} \qquad \textbf{(6.3-2)}$$

where

S = KL/r_{oy} for weak axis buckling and $K_\gamma L/r_{ox}$ for strong axis buckling

K = effective length factor for a prismatic member

K_γ = effective length factor for a tapered member as determined by a rational analysis

r_{ox} = strong axis radius of gyration at the smaller end of a tapered member, mm

r_{oy} = weak axis radius of gyration at the smaller end of a tapered member, mm

F_y = specified minimum yield stress, MPa

Q = reduction factor

= 1.0 if all elements meet the limiting width-thickness ratios λ_r of Section 2.5.1

= Q_sQ_a, determined in accordance with Section 2.5.3, if any stiffened and/or unstiffened elements exceed the ratios λ_r of Section 2.5.1

E = modulus of elasticity for steel, MPa

The smallest area of the tapered member shall be used for A_g in Equation 5.2-1.

6.3.4 **Design Flexural Strength.** The design flexural strength of tapered flexural members for the limit state of lateral-torsional buckling is $\phi_b M_n$, where $\phi_b = 0.90$ and the nominal strength is

$$M_n = (5/3)S'_x F_{b\gamma} \qquad \textbf{(6.3-3)}$$

where

S'_x = the section modulus of the critical section of the unbraced beam length under consideration

$$F_{b\gamma} = \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{wr}}}\right]F_y \le 0.60F_y \qquad \textbf{(6.3-4)}$$

Unless $F_{b\gamma} \le F_y/3$, in which case

$$F_{b\gamma} = B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \qquad \textbf{(6.3-5)}$$

In the preceding equations,

$$F_{s\gamma} = \frac{0.41E}{h_s L d_o / A_f} \qquad \textbf{(6.3-6)}$$

$$F_{w\gamma} = \frac{5.9E}{(h_w L / r_{To})^2} \qquad \textbf{(6.3-7)}$$

where

h_s = factor equal $1.0 + 0.230\gamma\sqrt{Ld_o / A_f}$

h_w = factor equal to $1.0 + 0.00385\gamma\sqrt{L / r_{To}}$

r_{To} = radius of gyration of a section at the smaller end, considering only the compression flange plus one-third of the compression web area, taken about an axis in the plane of the web, mm

A_f = area of the compression flange, mm^2

and where B is determined as follows:

(a) When the maximum moment M_2 in three adjacent segments of approximately equal unbraced length is located within the central segment and M_1 is the larger moment at one end of the three-segment portion of a member:

$$B = 1.0 + 0.37\left(1.0 + \frac{M_1}{M_2}\right) + 0.50\gamma\left(1.0\frac{M_1}{M_2}\right) \geq 1.0 \qquad \textbf{(6.3-8)}$$

(b) When the largest computed bending stress f_{b2} occurs at the larger end of two adjacent segments of approximately equal unbraced lengths and f_{b1} is the computed bending stress at the smaller end of the two-segment portion of a member:

$$B = 1.0 + 0.58\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) - 0.70\gamma\left(1.0\frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \qquad \textbf{(6.3-9)}$$

(c) When the largest computed bending stress f_{b2} occurs at the smaller end of two adjacent segments of approximately equal unbraced length and f_{b1} is the computed bending stress at the larger end of the two-segment portion of a member:

$$B = 1.0 + 0.55\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) + 2.20\gamma\left(1.0\frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \qquad \textbf{(6.3-10)}$$

In the foregoing, $\gamma = (d_L - d_o)/d_o$ is calculated for the unbraced length that contains the maximum computed bending stress. M_1/M_2 is considered as negative when producing single curvature. In the rare case where M_1/M_2 is positive, it is recommended that it be taken as zero. f_{b1}/f_{b2} is considered as negative when producing single curvature. If a point of contraflexure occurs in one of two adjacent unbraced segments, f_{b1}/f_{b2} is considered as positive. The ratio $f_{b1}/f_{b2} \neq 0$.

(d) When the computed bending stress at the smaller end of a tapered member or segment thereof is equal to zero:

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}} \qquad \textbf{(6.3-11)}$$

where $\gamma = (d_L - d_o)/d_o$ is calculated for the unbraced length adjacent to the point of zero bending stress.

6.3.5 **Design Shear Strength.** The design shear strength of tapered flexural members shall be determined in accordance with Section 6.2.

6.3.6 **Combined Flexure and Axial Force.** For tapered members with a single web taper subject to compression and bending about the major axis, Equation 8.1-1 applies, with the following modifications. P_n and P_{ex} shall be determined for the properties of the smaller end, using appropriate effective length factors. M_{nx}, M_u, and M_{px} shall be determined for the larger end; $M_{nx} = (5/3)S'_x F_{b\gamma}$, where S'_x is the elastic section modulus of the larger end, and $F_{b\gamma}$ is the design flexural stress of tapered members. C_{mx} is replaced by C'_m determined as follows:

(a) When the member is subjected to end moments which cause single curvature bending and approximately equal computed moments at the ends:

$$C'_m = 1.0 + 0.1\left(\frac{P_u}{\phi_b P_{ex}}\right) + 0.3\left(\frac{P_u}{\phi_b P_{ex}}\right)^2 \qquad \textbf{(6.3-12)}$$

(b) When the computed bending moment at the smaller end of the unbraced length is equal to zero:

$$C'_m = 1.0 + 0.9\left(\frac{P_u}{\phi_b P_{ex}}\right) + 0.6\left(\frac{P_u}{\phi_b P_{ex}}\right)^2 \qquad \textbf{(6.3-13)}$$

When the effective slenderness parameter $\lambda_{eff} \geq 1.5$ and combined stress is checked incrementally along the length, the actual area and the actual section modulus at the section under investigation is permitted to be used.

SECTION 6.4
BEAMS AND GIRDERS WITH WEB OPENINGS

The effect of all web openings on the design strength of steel and composite beams shall be determined. Adequate reinforcement shall be provided when the required strength exceeds the design strength of the member at the opening.

CHAPTER 7
PLATE GIRDERS

I-shaped plate girders shall be distinguished from I-shaped beams on the basis of the web slenderness ratio h / t_w. When this value is greater than λ_r the provisions of Sections 7.1 and 7.2 shall apply for design flexural strength. For $h / t_w \le \lambda_r$, the provisions of Chapter 6 shall apply for design flexural strength. For girders with unequal flanges, see Section 2.5.1.

The design shear strength and transverse stiffener design shall be based on either Section 6.2 (without tension-field action) or Section 7.3 (with tension-field action). For girders with unequal flanges, see Section 2.5.1.

SECTION 7.1
LIMITATIONS

Doubly and singly symmetric single-web non-hybrid and hybrid plate girders loaded in the plane of the web shall be proportioned according to the provisions of this section or Section 6.2, provided that the following limits are satisfied:

(a) For $\frac{a}{h} \le 1.5$

$$\frac{h}{t_w} \le 11.7 \frac{E}{F_{yf}} \qquad \textbf{(7.1-1)}$$

(b) For $\frac{a}{h} > 1.5$:

$$\left(\frac{h}{t_w} \le \frac{0.48E}{\sqrt{F_{yf}\left(F_{yf} + 114\right)}} \right) \qquad \textbf{(7.1-2)}$$

where

a = clear distance between transverse stiffeners, mm

h = clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, mm

t_w = web thickness, mm

F_{yf} = specified minimum yield stress of a flange, MPa

In unstiffened girders h/t_w shall not exceed 260.

SECTION 7.2
DESIGN FLEXURAL STRENGTH

The design flexural strength for plate girders with slender webs shall be $\phi_b M_n$, where $\phi_b = 0.90$ and M_n is the lower value obtained according to the limit states of tension-flange yield and compression-flange buckling. For girders with unequal flanges, see Section 2.5.1 for the determination of λ_r for the limit state of web local buckling.

(a) For tension-flange yield:

$$M_n = S_{xt} R_e F_{yt} \qquad \textbf{(7.2-1)}$$

(b) For compression-flange buckling:

$$M_n = S_{xc}R_{PG}R_eF_{cr} \quad \textbf{(7.2-2)}$$

where

$$R_{PG} = 1 - \frac{a_r}{1{,}200 + 300a_r}\left(\frac{h_c}{t_w} - 5.70\sqrt{\frac{E}{F_{cr}}}\right) \le 1.0 \quad \textbf{(7.2-3)}$$

R_e = hybrid girder factor

= $\dfrac{12 + a_r(3m - m^3)}{12 + 2a_r} \le 1.0$ (for non-hybrid girders, $R_e = 1.0$)

a_r = ratio of web area to compression flange area (≤ 10)

m = ratio of web yield stress to flange yield stress or to F_{cr}

F_{cr} = critical compression flange stress, MPa

F_{yt} = yield stress of tension flange, MPa

S_{xc} = section modulus referred to compression flange, mm^3

S_{xt} = section modulus referred to tension flange, mm^3

h_c = twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside of the face of the compression flange when welds are used, mm

The critical stress F_{cr} to be used is dependent upon the slenderness parameters λ, λ_p, λ_r, and C_{PG} as follows:

(a) For $\lambda \le \lambda_p$

$$F_{cr} = F_{yf} \quad \textbf{(7.2-4)}$$

(b) For $\lambda_p < \lambda \le \lambda_r$:

$$F_{cr} = C_bF_{yf}\left[1 - \frac{1}{2}\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \le F_{yf} \quad \textbf{(7.2-5)}$$

(c) For $\lambda > \lambda_r$:

$$F_{cr} = \frac{C_{PG}}{\lambda^2} \quad \textbf{(7.2-6)}$$

In the foregoing, the slenderness parameter shall be determined for both the limit state of lateral-torsional buckling and the limit state of flange local buckling; the slenderness parameter which results in the lowest value of F_{cr} governs.

(a) For the limit state of lateral-torsional buckling:

$$\lambda = \frac{L_b}{r_T} \quad \textbf{(7.2-7)}$$

$$\lambda_p = 1.76\sqrt{\frac{E}{F_{yf}}} \quad \textbf{(7.2-8)}$$

$$\lambda_r = 4.44\sqrt{\frac{E}{F_{yf}}} \quad \textbf{(7.2-9)}$$

$$C_{PG} = 1970000\ C_b \quad \textbf{(7.2-10)}$$

where

C_b = see Section 6.1.2, Equation 6.1-3

r_T = radius of gyration of compression flange plus one-third of the compression portion of the web, mm

(b) For the limit state of flange local buckling:

$$\lambda = \frac{b_f}{2t_f} \quad \textbf{(7.2-11)}$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_{yf}}} \quad \textbf{(7.2-12)}$$

$$\lambda_r = 1.35\sqrt{\frac{E}{F_{yf}/k_c}} \quad \textbf{(7.2-13)}$$

$$C_{PG} = 180650k_c \quad \textbf{(7.2-14)}$$

$$C_b = 1.0$$

where $k_c = 4/\sqrt{h/t_w}$ and $0.35 \le k_c \le 0.763$

The limit state of flexural web local buckling is not applicable.

SECTION 7.3
DESIGN SHEAR STRENGTH

The design shear strength with tension field action shall be $\phi_n V_n$, kN, where
ϕ_v = 0.90 and V_n is determined as follows:

(a) For $h/t_w \le 1.10\sqrt{k_v E/F_{yw}}$:

$$V_n = 0.6F_{yw}A_w \quad \textbf{(7.3-1)}$$

(b) For $h/t_w > 1.10\sqrt{k_v E/F_{yw}}$:

$$V_n = 0.6F_{yw}A_w\left(C_v + \frac{1-C_v}{1.15\sqrt{1+(a/h)^2}}\right) \quad \textbf{(7.3-2)}$$

Also see Sections 7.4 and 7.5.
Tension field action is not permitted for end-panels in non-hybrid plate girders, all panels in hybrid and web-tapered plate girders, and when a/h exceeds 3.0 or [260 (h/t_w)]. For these cases, the nominal strength is:

$$V_n = 0.6F_{yw}A_wC_v \quad \textbf{(7.3-3)}$$

The web plate buckling coefficient k_v is given as

$$k_v = 5 + \frac{5}{(a/h)^2} \quad \textbf{(7.3-4)}$$

except that k_v shall be taken as 5.0 if a/h exceeds 3.0 or $[260/(h/t_w)]^2$.

The shear coefficient C_v is determined as follows:

(a) For $1.10\sqrt{\frac{k_v E}{F_{yw}}} \le \frac{h}{t_w} \le 1.37\sqrt{\frac{k_v E}{F_{yw}}}$:

$$C_v = \frac{1.10\sqrt{k_v E / F_{yw}}}{h / t_w} \quad \textbf{(7.3-5)}$$

(b) For $\frac{h}{t_w} > 1.37\sqrt{\frac{k_v E}{F_{yw}}}$:

$$C_v = \frac{1.51 k_v E}{(h / t_w)^2 F_{yw}} \quad \textbf{(7.3-6)}$$

SECTION 7.4
TRANSVERSE STIFFENERS

Transverse stiffeners are not required in plate girders where $h / t_w \le 2.45\sqrt{E / F_{yw}}$, or where the required shear V_u, as determined by structural analysis for the factored loads, is less than or equal to $0.60\phi_v F_{yw} A_w C_v$ where C_v is determined for $k_v = 5$ and $\phi_v = 0.90$. Stiffeners may be required in certain portions of a plate girder to develop the required shear or to satisfy the limitations given in Section 7.1. Transverse stiffeners shall satisfy the requirements of Section 6.2.3.

When designing for tension field action, the stiffener area A_{st} shall not be less than

$$\frac{F_{yw}}{F_{yst}}\left[0.15 D h t_w (1 - C_v)\frac{V_u}{\phi V_n} - 18 t_w^2\right] \ge 0 \quad \textbf{(7.4-1)}$$

where

F_{yst} = specified yield stress of the stiffener material, MPa

D = 1 for stiffeners in pairs

= 1.8 for single angle stiffeners

= 2.4 for single plate stiffeners

C_v and V_n = are defined in Section 7.3, and V_u is the required shear at the location of the stiffener.

SECTION 7.5
FLEXURE-SHEAR INTERACTION

For $0.6\phi V_n \le V_u \le \phi V_n$ and $0.75\phi M_n \le M_u \le \phi M_n$, plate girders with webs designed for tension field action shall satisfy the additional flexure-shear interaction criterion:

$$\frac{M_u}{\phi M_n} + 0.625\frac{V_u}{\phi V_n} \le 1.375 \quad \textbf{(7.5-1)}$$

where

M_n = nominal flexural strength of plate girder from Section 7.2 or Section 6.1

ϕ = 0.90

V_n = nominal shear strength from Section 7.3

CHAPTER 8
MEMBERS UNDER COMBINED FORCES AND TORSION

This chapter applies to prismatic members subject to axial force and flexure about one or both axes of symmetry, with or without torsion, and torsion only. For web-tapered members, see Section 6.3.

SECTION 8.1
SYMMETRIC MEMBERS SUBJECT TO BENDING AND AXIAL FORCE

8.1.1 **Doubly and Singly Symmetric Members in Flexure and Tension**. The interaction of flexure and tension in symmetric shapes shall be limited by Equations 8.1-1a and 8.1-1b.

(a) For $\frac{P_u}{\phi P_n} \geq 0.2$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \quad \textbf{(8.1-1a)}$$

(b) For $\frac{P_u}{\phi P_n} < 0.2$

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \quad \textbf{(8.1-1b)}$$

where

P_u = required tensile strength, N

P_n = nominal tensile strength determined in accordance with Section 4.1, (N)

M_u = required flexural strength determined in accordance with Section 3.1, (N-mm)

M_n = nominal flexural strength determined in accordance with Section 6.1, (N-mm)

x = subscript relating symbol to strong axis bending

y = subscript relating symbol to weak axis bending

ϕ = ϕ_t = resistance factor for tension (see Section 4.1)

ϕ_b = resistance factor for flexure = 0.90

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equations 8.1-1a and 8.1-1b.

8.1.2 **Doubly and Singly Symmetric Members in Flexure and Compression.** The interaction of flexure and compression in symmetric shapes shall be limited by Equations 8.1-1a and 8.1-1b.

where

P_u = required compressive strength, N

P_n = nominal compressive strength determined in accordance with Section 5.2, N

$\phi = \phi_c$ = resistance factor for compression = 0.85 (see Section 5.2)

ϕ_b = resistance factor for flexure = 0.90

SECTION 8.2
UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

The design strength, ϕF_n, of the member shall equal or exceed the required strength expressed in terms of the normal stress f_{un} or the shear stress f_{uv}, determined by elastic analysis for the factored loads:

(a) For the limit state of yielding under normal stress:

$$f_{un} \leq \phi F_y \qquad \textbf{(8.2-1)}$$

$$\phi = 0.90$$

$$F_n = F_y$$

(b) For the limit state of yielding under shear stress:

$$f_{uv} \leq 0.6 \phi F_n \qquad \textbf{(8.2-2)}$$

$$\phi = 0.90$$

$$F_n = F_y$$

(c) For the limit state of buckling:

$$f_{un} \text{ or } f_{uv} \leq \phi_c F_n, \text{ as applicable} \qquad \textbf{(8.2-3)}$$

$$\phi_c = 0.85$$

$$F_n = F_{cr}$$

Some constrained local yielding is permitted adjacent to areas which remain elastic.

SECTION 8.3
ALTERNATIVE INTERACTION EQUATIONS FOR MEMBERS UNDER COMBINED STRESS

This section provides alternative interaction equations for braced frames with I-shaped members with $b_f / d \leq 1.0$ and box-shaped members.

For I-shaped members with $b_f / d \leq 1.0$ and box-shaped members, the use of the following interaction equations in lieu of Equations 8.1-1a and 8.1-1b is permitted for braced frames only. Both Equations 8.3-1 and 8.3-2 shall be satisfied.

$$\left(\frac{M_{ux}}{\phi_b M'_{px}}\right)^{\zeta} + \left(\frac{M_{uy}}{\phi_b M'_{py}}\right)^{\zeta} \leq 1.0 \qquad \textbf{(8.3-1)}$$

$$\left(\frac{C_{mx} M_{ux}}{\phi_b M'_{nx}}\right)^{\eta} + \left(\frac{C_{my} M_{uy}}{\phi_b M'_{ny}}\right)^{\eta} \leq 1.0 \qquad \textbf{(8.3-2)}$$

The terms in Equations 8.3-1 and 8.3-2 are determined as follows:

(a) For I-shaped members:

For $b_f / d < 0.5$:

$$\zeta = 1.0$$

For $0.5 \le b_f / d \le 1.0$:

$$\zeta = 1.6 - \frac{P_u / P_y}{2\left[\ln(P_u / P_y)\right]} \qquad \textbf{(8.3-3)}$$

For $b_f / d < 0.3$:

$$\eta = 1.0$$

For $0.3 \le b_f / d \le 1.0$:

$$\eta = 0.4 + \frac{P_u}{P_y} + \frac{b_f}{d} \ge 1.0 \qquad \textbf{(8.3-4)}$$

where

b_f = flange width, (mm)

d = member depth, (mm)

C_m = coefficient applied to the bending term in the interaction equation for prismatic members and dependent on column curvature caused by applied moments, see Section 3.1.

$$M'_{px} = 1.2M_{px}\left[1-(P_u / P_y)\right] \le M_{px} \qquad \textbf{(8.3-5)}$$

$$M'_{py} = 1.2M_{py}\left[1-(P_u / P_y)^2\right] \le M_{py} \qquad \textbf{(8.3-6)}$$

$$M'_{nx} = M_{nx}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ex}}\right) \qquad \textbf{(8.3-7)}$$

$$M'_{ny} = M_{ny}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ey}}\right) \qquad \textbf{(8.3-8)}$$

(b) For box-section members:

$$\zeta = 1.7 - \frac{P_u / P_y}{\ln(P_u / P_y)} \qquad \textbf{(8.3-9)}$$

$$\eta = 1.7 - \frac{P_u / P_y}{\ln(P_u / P_y)} - a\lambda_x\left(\frac{P_u}{P_y}\right)^b > 1.1 \qquad \textbf{(8.3-10)}$$

For $P_u / P_y \le 4.0$, $a = 0.06$, and $b = 1.0$;

For $P_u / P_y > 4.0$, $a = 0.15$, and $b = 2.0$:

$$M'_{px} = 1.2M_{px}\left[1 - P_u / P_y\right] \le M_{px} \qquad \textbf{(8.3-11a)}$$

$$M'_{py} = 1.2M_{py}\left[1 - P_u / P_y\right] \le M_{py} \qquad \textbf{(8.3-11b)}$$

$$M'_{nx} = M_{nx}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ex}}\frac{1.25}{(B/H)^{1/3}}\right) \qquad \textbf{(8.3-12)}$$

$$M'_{ny} = M_{ny}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ey}}\frac{1.25}{(B/H)^{1/2}}\right) \qquad \textbf{(8.3-13)}$$

where

P_n = nominal compressive strength determined in accordance with Section 5.2, N

P_u = required axial strength, N

P_y = compressive yield strength $A_g F_y$, N

ϕ_b = resistance factor for flexure = 0.90

ϕ_c = resistance factor for compression = 0.85

P_e = Euler buckling strength $A_g F_y / \lambda_c^2$, where λ_c is the column slenderness parameter defined by Equation 5.2-4, N

M_u = required flexural strength, N-mm

M_n = nominal flexural strength, determined in accordance with Section 6.1, N-mm

M_p = plastic moment $\leq 1.5 F_y S$, N-mm

λ_x = column slenderness parameter with respect to the strong axis

B = outside width of box section parallel to major principal axis x, mm

H = outside depth of box section perpendicular to major principal axis x, mm

CHAPTER 9
COMPOSITE MEMBERS

This chapter applies to composite columns composed of rolled or built-up structural steel shapes, pipe or HSS, and structural concrete acting together and to steel beams supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending. Simple and continuous composite beams with shear connectors and concrete-encased beams, constructed with or without temporary shores, are included.

SECTION 9.1
DESIGN ASSUMPTIONS AND DEFINITIONS

Force Determination. In determining forces in members and connections of a structure that includes composite beams, consideration shall be given to the effective sections at the time each increment of load is applied.

Elastic Analysis. For an elastic analysis of continuous composite beams without haunched ends, it is permissible to assume that the stiffness of a beam is uniform throughout the beam length. The stiffness is permitted to be computed using the weighted average of the moments of inertia in the positive moment region and the negative moment region.

Plastic Analysis. When plastic analysis is used, as limited in Section 1.5.1, the strength of flexural composite members shall be determined from plastic stress distributions.

Plastic Stress Distribution for Positive Moment. If the slab in the positive moment region is connected to the steel beam with shear connectors, a concrete stress of $0.85f_c^{'}$ is permitted to be assumed uniformly distributed throughout the effective compression zone, where $f_c^{'}$ is the specified compressive strength of the concrete. Concrete tensile strength shall be neglected. A uniformly distributed steel stress of F_y shall be assumed throughout the tension zone and throughout the compression zone in the structural steel section. The net tensile force in the steel section shall be equal to the compressive force in the concrete slab.

Plastic Stress Distribution for Negative Moment. If the slab in the negative moment region is connected to the steel beam with shear connectors, a tensile stress of F_{yr} shall be assumed in all adequately developed longitudinal reinforcing bars within the effective width of the concrete slab. Concrete tensile strength shall be neglected. A uniformly distributed steel stress of F_y shall be assumed throughout the tension zone and throughout the compression zone in the structural steel section. The net compressive force in the steel section shall be equal to the total tensile force in the reinforcing steel.

Elastic Stress Distribution. When a determination of elastic stress distribution is required, strains in steel and concrete shall be assumed directly proportional to the distance from the neutral axis. The stress shall equal strain times modulus of elasticity for steel, E, or modulus of elasticity for concrete, E_c. Concrete tensile strength shall be neglected. Maximum stress in the steel shall not exceed F_y. Maximum compressive stress in the concrete shall not exceed $0.85f_c^{'}$. In composite hybrid beams, the maximum stress in the steel flange shall not exceed

F_{yf} but the strain in the web may exceed the yield strain; the stress shall be taken as F_{yw} at such locations.

Fully Composite Beam. Shear connectors are provided in sufficient numbers to develop the maximum flexural strength of the composite beam. For elastic stress distribution it shall be assumed that no slip occurs.

Partially Composite Beam. The shear strength of shear connectors governs the flexural strength of the partially composite beam. Elastic computations such as those for deflections, fatigue, and vibrations shall include the effect of slip.

Concrete-Encased Beam. A beam totally encased in concrete cast integrally with the slab may be assumed to be interconnected to the concrete by natural bond, without additional anchorage, provided that: (1) concrete cover over the beam sides and soffit is at least 50 mm; (2) the top of the beam is at least 38 mm below the top and 50 mm above the bottom of the slab; and (3) concrete encasement contains adequate mesh or other reinforcing steel to prevent spalling of concrete.

Composite Column. A steel column fabricated from rolled or built-up steel shapes and encased in structural concrete or fabricated from steel pipe or HSS and filled with structural concrete.

Encased Composite Column. A steel column fabricated from rolled or built-up shapes and encased in structural concrete.

Filled Composite Column. Structural steel HSS or pipes that are filled with structural concrete.

SECTION 9.2
COMPRESSION MEMBERS

9.2.1 **Limitations.** To qualify as a composite column, the following limitations shall be met:

(1) The cross-sectional area of the steel shape, pipe, or HSS shall comprise at least four percent of the total composite cross section.

(2) Concrete encasement of a steel core shall be reinforced with longitudinal load-carrying bars, longitudinal bars to restrain concrete, and lateral ties. Longitudinal load-carrying bars shall be continuous at framed levels; longitudinal restraining bars may be interrupted at framed levels. The spacing of ties shall be not greater than two-thirds of the least dimension of the composite cross section. The cross-sectional area of the transverse and longitudinal reinforcement shall be at least 180 mm^2 per m of bar spacing. The encasement shall provide at least 38 mm of clear cover outside of both transverse and longitudinal reinforcement.

(3) Concrete shall have a specified compressive strength $f_c^{'}$ of not less than 21 MPa nor more than 55 MPa for normal weight concrete and not less than 28 MPa for lightweight concrete.

(4) The specified minimum yield stress of structural steel and reinforcing bars used in calculating the strength of a composite column shall not exceed 415 MPa.

(5) The minimum wall thickness of structural steel pipe or HSS filled with concrete shall be equal to $b\sqrt{F_y/3E}$ for each face of width b in rectangular sections and $D\sqrt{F_y/8E}$ for circular sections of outside diameter D.

9.2.2 **Design Strength.** The design strength of axially loaded composite columns is $\phi_c P_n$,

where

ϕ_c = 0.85

P_n = nominal axial compressive strength determined from Equations 5.2-1 through 5.2-4 with the following modifications:

(1) A_g is replaced by A_s, the gross area of steel shape, pipe, or HSS, mm^2

(2) r is replaced by r_m, the radius of gyration of the steel shape, pipe, or HSS except that for steel shapes it shall not be less than 0.3 times the overall thickness of the composite cross section in the plane of buckling, mm

(3) F_y is replaced by F_{my}, the modified yield stress from Equation 9.2-1

$$F_{my} = F_y + c_1 F_{yr}(A_r / A_s) + c_2 f_c'(A_c / A_s) \qquad \textbf{(9.2-1)}$$

(4) E is replaced by E_m, the modified modulus of elasticity from Equation 9.2-2.

$$E_m = E + c_3 E_c (A_c / A_s) \qquad \textbf{(9.2-2)}$$

where

A_c = area of concrete, mm^2

A_r = area of longitudinal reinforcing bars, mm^2

A_s = area of steel, mm^2

E = modulus of elasticity of steel, MPa

E_c = modulus of elasticity of concrete. E_c is permitted to be computed from $E_c = 0.041 w^{1.5}\sqrt{f_c'}$ where w, the unit weight of concrete, is expressed in kg/m^3 and f_c' is expressed in MPa.

F_y = specified minimum yield stress of steel shape, pipe, or HSS, MPa

F_{yr} = specified minimum yield stress of longitudinal reinforcing bars, MPa

f_c' = specified compressive strength of concrete, MPa

c_1, c_2, c_3 = numerical coefficients. For concrete-filled pipe and HSS: $c_1 = 1.0$, $c_2 = 0.85$, and $c_3 = 0.4$; for concrete-encased shapes, $c_1 = 0.7$, $c_2 = 0.6$, and $c_3 = 0.2$

9.2.3 **Columns with Multiple Steel Shapes.** If the composite cross section includes two or more steel shapes, the shapes shall be interconnected with lacing, tie plates, or batten plates to prevent buckling of individual shapes before hardening of concrete.

9.2.4 **Load Transfer.** Loads applied to axially loaded encased composite columns shall be transferred between the steel and concrete in accordance with the following requirements:

(a) When the external force is applied directly to the steel section, shear connectors shall be provided to transfer the force V_u' as follows:

$$V_u' = V_u(1 - A_s F_y / P_n) \qquad \textbf{(9.2-3)}$$

where

V_u = force introduced to column, N

A_s = area of steel section, mm^2

F_y = yield strength of the steel section, MPa

P_n = nominal compressive strength of the composite column without consideration of slenderness effects, N

(b) When the external force is applied directly to the concrete encasement, shear connectors shall be provided to transfer the force V_u' as follows:

$$V_u' = V_u(A_s F_y / P_n) \qquad \textbf{(9.2-4)}$$

Shear connectors transferring the force V_u' shall be distributed along the length of the member. The maximum connector spacing shall be 405 mm and connectors shall be placed on at least two faces of the steel shape in a configuration symmetrical about the steel shape axes.

Where the supporting concrete area in direct bearing is wider than the loaded area on one or more sides and otherwise restrained laterally on the remaining sides, the maximum design strength shall be:

$$\phi_B\, 1.7 f_c'\, A_B \qquad \textbf{(9.2-5)}$$

where

ϕ_B = 0.65

A_B = loaded area, mm^2

SECTION 9.3
FLEXURAL MEMBERS

9.3.1 **Effective Width.** The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline, each of which shall not exceed:

(1) one-eighth of the beam span, center-to-center of supports;

(2) one-half the distance to the centerline of the adjacent beam; or

(3) the distance to the edge of the slab.

9.3.2 **Design Strength of Beams with Shear Connectors.** The positive design flexural strength $\phi_b M_n$ shall be determined as follows:

(a) For $h / t_w \leq 3.76\sqrt{E / F_{yf}}$:

$\phi_b = 0.85$; M_n shall be determined from the plastic stress distribution on the composite section.

(b) For $h/t_w > 3.76\sqrt{E/F_{yf}}$:

$\phi_b = 0.90$; M_n shall be determined from the superposition of elastic stresses, considering the effects of shoring.

The negative design flexural strength $\phi_b M_n$ shall be determined for the steel section alone, in accordance with the requirements of Chapter 6.

Alternatively, the negative design flexural strength $\phi_b M_n$ shall be computed with $\phi_b = 0.85$ and M_n determined from the plastic stress distribution on the composite section, provided that:

(1) Steel beam is an adequately braced compact section, as defined in Section 2.5.

(2) Shear connectors connect the slab to the steel beam in the negative moment region.

(3) Slab reinforcement parallel to the steel beam, within the effective width of the slab, is properly developed.

9.3.3 **Design Strength of Concrete-Encased Beams.** The design flexural strength $\phi_b M_n$ shall be computed with $\phi_b = 0.90$ and M_n determined from the superposition of elastic stresses, considering the effects of shoring.

Alternatively, the design flexural strength $\phi_b M_n$ shall be computed with $\phi_b = 0.90$ and M_n determined from the plastic stress distribution on the steel section alone.

If shear connectors are provided and the concrete meets the requirements of Section 9.2.1(2), the design flexural strength $\phi_b M_n$ shall be computed based upon the plastic stress distribution on the composite section with $\phi_b = 0.85$.

9.3.4 **Strength During Construction.** When temporary shores are not used during construction, the steel section alone shall have adequate strength to support all loads applied prior to the concrete attaining 75 percent of its specified strength f_c'.

The design flexural strength of the steel section shall be determined in accordance with the requirements of Section 6.1.

9.3.5 Formed Steel Deck

9.3.5.1 **General.** The design flexural strength, $\phi_b M_n$, of composite construction consisting of concrete slabs on formed steel deck connected to steel beams shall be determined by the applicable portions of Section 9.3.2, with the following modifications:

(1) This section is applicable to decks with nominal rib height not greater than 75 mm. The average width of concrete rib or haunch w_r shall be not less than 50 mm, but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.

(2) The concrete slab shall be connected to the steel beam with welded stud shear connectors 19 mm or less in diameter (AWS D1.1). Studs shall be welded either through the deck or directly to the steel beam. Stud shear connectors, after installation, shall extend not less 38 mm above the top of the steel deck.

The slab thickness above the steel deck shall be not less than 50 mm.

9.3.5.2 **Deck Ribs Oriented Perpendicular to Steel Beam.** Concrete below the top of the steel deck shall be neglected in determining section properties and in calculating A_c for deck ribs oriented perpendicular to the steel beams.

The spacing of stud shear connectors along the length of a supporting beam shall not exceed 915 mm.

The nominal strength of a stud shear connector shall be the value stipulated in Section 9.5 multiplied by the following reduction factor:

$$\frac{0.85}{\sqrt{N_r}}(w_r / h_r)[(H_s / h_r) - 1.0] \le 1.0 \qquad \textbf{(9.3-1)}$$

where

h_r = nominal rib height, mm

H_s = length of stud connector after welding, mm, not to exceed the value $h_r + 75$ mm in computations, although actual length may be greater

N_r = number of stud connectors in one rib at a beam intersection, not to exceed three in computations, although more than three studs may be installed

w_r = average width of concrete rib or haunch (as defined in Section 9.3.5.1), mm

Where there is only a single stud placed in a rib oriented perpendicular to the steel beam, the reduction factor of Equation 9.3-1 shall not exceed 0.75.

To resist uplift, steel deck shall be anchored to all supporting members at a spacing not to exceed 460 mm. Such anchorage shall be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other devices specified by the designer.

9.3.5.3 **Deck Ribs Oriented Parallel to Steel Beam.** Concrete below the top of the steel deck may be included in determining section properties and shall be included in calculating A_c in Section 9.5.

Steel deck ribs over supporting beams may be split longitudinally and separated to form a concrete haunch.

When the nominal depth of steel deck is 38 mm or greater, the average width w_r of the supported haunch or rib shall be not less than 50 mm for the first stud in the transverse row plus four stud diameters for each additional stud.

The nominal strength of a stud shear connector shall be the value stipulated in Section 9.5, except that when w_r / h_r is less than 1.5, the value from Section 9.5 shall be multiplied by the following reduction factor:

$$0.6(w_r / h_r)[(H_s / h_r) - 1.0] \le 1.0 \qquad \textbf{(9.3-2)}$$

where h_r and H_s are as defined in Section 9.3.5.2 and w_r is the average width of concrete rib or haunch as defined in Section 9.3.5.1.

9.3.6 **Design Shear Strength.** The design shear strength of composite beams shall be determined by the shear strength of the steel web, in accordance with Section 6.2.

SECTION 9.4
COMBINED COMPRESSION AND FLEXURE

The interaction of axial compression and flexure in the plane of symmetry on composite members shall be limited by Section 8.1.2 with the following modifications:

M_n = nominal flexural strength determined from plastic stress distribution on the composite cross section except as provided below, N-mm

P_{e1}, P_{e2} = $A_s F_{my} / \lambda_c^2$ elastic buckling load, N

F_{my} = modified yield stress, MPa, see Section 9.2

ϕ_b = resistance factor for flexure from Section 9.3

ϕ_c = resistance factor for compression = 0.85

λ_c = column slenderness parameter defined by Equation 5.2-4 as modified in Section 9.2.2

When the axial term in Equations 8.1-1a and 8.1-1b is less than 0.3, the nominal flexural strength M_n shall be determined by straight line transition between the nominal flexural strength determined from the plastic distribution on the composite cross sections at $(P_u/\phi_c P_n) = 0.3$ and the flexural strength at $P_u = 0$ as determined in Section 9.3. If shear connectors are used at $P_u = 0$, they shall be provided whenever $P_u/\phi_c P_n$ is less than 0.3.

SECTION 9.5
SHEAR CONNECTORS

This section applies to the design of stud and channel shear connectors. For connectors of other types, see Section 9.6.

9.5.1 **Materials.** Shear connectors shall be headed steel studs not less than four stud diameters in length after installation, or hot rolled steel channels. The stud connectors shall conform to the requirements of Section 1.3.6. The channel connectors shall conform to the requirements of Section 1.3. Shear connectors shall be embedded in concrete slabs made with ASTM C33 aggregate or with rotary kiln produced aggregates conforming to ASTM C330, with concrete unit weight not less than 1440 kg/m^3.

9.5.2 **Horizontal Shear Force.** The entire horizontal shear at the interface between the steel beam and the concrete slab shall be assumed to be transferred by shear connectors, except for concrete-encased beams as defined in Section 9.1. For composite action with concrete subject to flexural compression, the total horizontal shear force between the point of maximum positive moment and the point of zero moment shall be taken as the smallest of the following: (a) $0.85 f_c' A_c$; (b) $A_s F_y$; and (c) ΣQ_n;

where

A_c = area of concrete slab within effective width, mm^2

A_s = area of steel cross section, mm^2

ΣQ_n = sum of nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment, N

For hybrid beams, the yield force shall be computed separately for each component of the cross section; $A_s F_y$ of the entire cross section is the sum of the component yield forces.

In continuous composite beams where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel beam, the total horizontal shear force between the point of maximum negative moment and the point of zero moment shall be taken as the smaller of A_rF_{yr} and ΣQ_n;

where

A_r = area of adequately developed longitudinal reinforcing steel within the effective width of the concrete slab, mm^2

F_{yr} = minimum specified yield stress of the reinforcing steel, MPa

9.5.3 **Strength of Stud Shear Connectors.** The nominal strength of one stud shear connector embedded in a solid concrete slab is

$$Q_n = 0.5A_{sc}\sqrt{f'_cE_c} \leq A_{sc}F_u \quad \textbf{(9.5-1)}$$

where

A_{sc} = cross-sectional area of stud shear connector, mm^2

F_u = specified minimum tensile strength of a stud shear connector, MPa

E_c = modulus of elasticity of concrete, MPa

For a stud shear connector embedded in a slab on a formed steel deck, refer to Section 9.3 for reduction factors given by Equations 9.3-1 and 9.3-2 as applicable. The reduction factors apply only to the $0.5A_{sc}\sqrt{f'_cE_c}$ term in Equation 9.5-1.

9.5.4 **Strength of Channel Shear Connectors.** The nominal strength of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f'_cE_c} \quad \textbf{(9.5-2)}$$

where

t_f = flange thickness of channel shear connector, mm

t_w = web thickness of channel shear connector, mm

L_c = length of channel shear connector, mm

9.5.5 **Required Number of Shear Connectors.** The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the horizontal shear force as determined in Section 9.5.2 divided by the nominal strength of one shear connector as determined from Section 9.5.3 or Section 9.5.4.

9.5.6 **Shear Connector Placement and Spacing.** Shear connectors required on each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment, unless otherwise specified. However, the number of shear connectors placed between any concentrated load and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the concentrated load point.

Shear connectors shall have at least 25 mm of lateral concrete cover, except for connectors installed in the ribs of formed steel decks. The diameter of studs shall not be greater than 2.5 times the thickness of the flange to which they are welded, unless located over the web. The minimum center-to-center spacing of stud connectors shall be six diameters along the longitudinal axis of the supporting composite beam and four diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks oriented perpendicular to the steel beam the minimum center-to-center spacing shall be four diameters in any direction. The maximum center-to-center spacing of shear connectors shall not exceed eight times the total slab thickness. Also see Section 9.3.5.2.

SECTION 9.6
SPECIAL CASES

When composite construction does not conform to the requirements of Section 9.1 through Section 9.5, the strength of shear connectors and details of construction shall be established by a suitable test program.

CHAPTER 10
CONNECTIONS, JOINTS, AND FASTENERS

This chapter applies to connecting elements, connectors, and the affected elements of the connected members subject to static loads. For connections subject to fatigue, see Section 11.3.

SECTION 10.1
GENERAL PROVISIONS

10.1.1 **Design Basis.** Connections consist of affected elements of connected members (e.g., beam webs), connecting elements (e.g., gussets, angles, brackets), and connectors (e.g., welds, bolts, rivets). These components shall be proportioned so that their design strength equals or exceeds the required strength determined by structural analysis for factored loads acting on the structure or a specified proportion of the strength of the connected members, whichever is appropriate.

10.1.2 **Simple Connections.** Connections of beams, girders, or trusses shall be designed as flexible and are permitted to be proportioned for the reaction shears only, except as otherwise indicated in the design documents. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, some inelastic but self-limiting deformation in the connection is permitted.

10.1.3 **Moment Connections.** End connections of restrained beams, girders, and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections.

10.1.4 **Compression Members with Bearing Joints.** When columns bear on bearing plates or are finished to bear at splices, there shall be sufficient connectors to hold all parts securely in place.
When compression members other than columns are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for 50 % of the required strength of the member.
All compression joints shall be proportioned to resist any tension developed by the factored load combinations stipulated in Section 1.4.

10.1.5 **Splices in Heavy Sections.** This paragraph applies to ASTM A6/A6M Group 4 and 5 and equivalent rolled shapes, or shapes built-up by welding plates more than 50 mm thick together to form the cross section, and where the cross section is to be spliced and subject to primary tensile stresses due to tension or flexure. When the individual elements of the cross section are spliced prior to being joined to form the cross section in accordance with AWS D1.1, Article 5.21.6, the applicable provisions of AWS D1.1 apply in lieu of the requirements of this section. When tensile forces in these sections are to be transmitted through splices by complete-joint-penetration groove welds, material notch-toughness requirements as given in Section 1.3.1.3, weld access hole details as given in Section 10.1.6, welding preheat requirements as given in Section 10.2.8, and thermal-cut surface preparation and inspection requirements as given in Section 12.2.2 apply.

At tension splices in ASTM A6/A6M Group 4 and 5 and equivalent shapes and built-up members of material more than 50 mm thick, weld tabs and backing shall be removed and the surfaces ground smooth.

When splicing ASTM A6/A6M Group 4 and 5 and equivalent rolled shapes and their equivalents or shapes built-up by welding plates more than 50 mm thick to form a cross section, and where the section is to be used as a primary compression member, all weld access holes required to facilitate groove welding operations shall satisfy the provisions of Section 10.1.6.

Alternatively, splicing of such members subject to compression, including members which are subject to tension due to wind or seismic loads, shall be accomplished using splice details which do not induce large weld shrinkage strains; for example partial-joint-penetration flange groove welds with fillet-welded surface lap plate splices on the web, bolted lap plate splices, or combination bolted/fillet-welded lap plate splices.

10.1.6 **Beam Copes and Weld Access Holes.** All weld access holes required to facilitate welding operations shall have a length from the toe of the weld preparation not less than 1.5 times the thickness of the material in which the hole is made. The height of the access hole shall be adequate for deposition of sound weld metal in the adjacent plates and provide clearance for weld tabs for the weld in the material in which the hole is made, but not less than the thickness of the material. In hot-rolled shapes and built-up shapes, all beam copes and weld access holes shall be shaped free of notches and sharp re-entrant corners, except that when fillet web-to-flange welds are used in built-up shapes, access holes are permitted to terminate perpendicular to the flange.

For ASTM A6/A6M Group 4 and 5 and equivalent shapes and built-up shapes of material more than 50 mm thick, the thermally cut surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods prior to deposition of splice welds. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle methods.

10.1.7 **Minimum Strength of Connections.** Connections providing design strength shall be designed to support a factored load not less than 44 kN, except for lacing, sag rods, or girts.

10.1.8 **Placement of Welds and Bolts.** Groups of welds or bolts at the ends of any member which transmit axial force into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically-loaded single angle, double angle, and similar members.

10.1.9 **Bolts in Combination with Welds.** In new work, A307 bolts or high-strength bolts proportioned as bearing-type connections shall not be considered as sharing the load in combination with welds. Welds, if used, shall be proportioned for the entire force in the connection. In slip-critical connections, high-strength bolts are permitted to be considered as sharing the load with the welds. These calculations shall be made at factored loads.

In making welded alterations to structures, existing rivets and high-strength bolts tightened to the requirements for slip-critical connections are permitted to be utilized for carrying loads present at the time of alteration and the welding need only provide the additional design strength required.

10.1.10 High-Strength Bolts in Combination with Rivets. In both new work and alterations, in connections designed as slip-critical connections in accordance with the provisions of Section 10.3, high-strength bolts are permitted to be considered as sharing the load with rivets.

10.1.11 Limitations on Bolted and Welded Connections. Fully pretensioned high-strength bolts (see Table 10.3-1) or welds shall be used for the following connections:

Column splices in all tier structures 60 m or more in height.

Column splices in tier structures 30 m to 60 m in height, if the least horizontal dimension is less than 40 percent of the height.

Column splices in tier structures less than 30 m in height, if the least horizontal dimension is less than 25 percent of the height.

Connections of all beams and girders to columns and of any other beams and girders on which the bracing of columns is dependent, in structures over 38 m in height.

In all structures carrying cranes of over 50 kN capacity: roof-truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports.

Connections for supports of running machinery, or of other live loads which produce impact or reversal of stress.

Any other connections stipulated on the design drawings.

In all other cases connections are permitted to be made with A307 bolts or snug-tight high-strength bolts.

For the purpose of this section, the height of a tier structure shall be taken as the vertical distance from the curb level to the highest point of the roof beams in the case of flat roofs, or to the mean height of the gable in the case of roofs having a slope of more than 25 percent. Where the curb level has not been established, or where the structure does not adjoin a street, the mean level of the adjoining land shall be used instead of curb level. It is permissible to exclude penthouses in computing the height of the structure.

SECTION 10.2
WELDS

All provisions of AWS D1.1, apply under this code, except the provisions applicable to Tubular Structures, which are outside the scope of SBC 306, and except that the provisions of the listed SBC 306 sections in lieu of the cited AWS Code provisions as follows:

Section 10.1.5 and 10.1.6 in lieu of AWS D1.1 Section 5.17.
Section 10.2.2 in lieu of AWS D1.1 Section 2.4.1.1.
Table 10.2-5 in lieu of AWS D1.1 Table 2.3
Table 11.3-1 in lieu of AWS D1.1 Section 2.27.1
Section 11.3 in lieu of AWS Section 2, Part C
Section 13.2.2 in lieu of AWS Sections 5.15.1.2, 5.15.4.3 and 5.15.4.4

The length and disposition of welds, including end returns shall be indicated on the design and shop drawings.

10.2.1 Groove Welds

10.2.1.1 Effective Area. The effective area of groove welds shall be considered as the effective length of the welds times the effective throat thickness.
The effective length of a groove weld shall be the width of the part joined.
The effective throat thickness of a complete-joint-penetration groove weld shall be the thickness of the thinner part joined.

The effective throat thickness of a partial-joint-penetration groove weld shall be as shown in Table 10.2-1.

The effective throat thickness of a flare groove weld when flush to the surface of a bar or 90° bend in formed section shall be as shown in Table 10.2-2. Random sections of production welds for each welding procedure, or such test sections as may be required by design documents, shall be used to verify that the effective throat is consistently obtained.

Larger effective throat thicknesses than those in Table 10.2-2 are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication or as required by the designer.

10.2.1.2 Limitations. The minimum effective throat thickness of a partial-joint-penetration groove weld shall be as shown in Table 10.2-3. Weld size is determined by the thicker of the two parts joined, except that the weld size need not exceed the thickness of the thinnest part joined, even when a larger size is required by calculated strength. For this exception, particular care shall be taken to provide sufficient preheat for soundness of the weld.

TABLE 10.2-1
Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Welding Process	Welding Position	Included Angle at Root of Groove	Effective Throat Thickness
Shielded metal arc Submerged arc	All	J or U joint	Depth of chamfer
Gas metal arc		Bevel or V joint $\geq 60^{o}$	
Flux-cored arc		Bevel or V joint $< 60^{o}$ Bevel or V but $\geq 45°$	Depth of chamfer Minus 3 mm

TABLE 10.2-2
Effective Throat Thickness of Flare Groove Welds

Type of Weld	Radius (R) of Bar or Bend	Effective Throat Thickness
Flare bevel groove	All	$5/16R$
Flare V-groove	All	$1/2R$
[a] Use $3/8R$ for Gas Metal Arc Welding (except short circuiting transfer process) when $R \geq 25$ mm		

TABLE 10.2-3 Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds	
Material Thickness of Thicker Part Joined (mm)	**Minimum Effective Throat Thickness[a], (mm)**
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6
Over 19 to 38	8
Over 38 to 57	10
Over 57 to 150	13
Over 150	16
[a] See Table 10.2-1	

10.2.2 Fillet Welds

10.2.2.1 Effective Area. The effective area of fillet welds shall be as defined in AWS D1.1 Section 2.4.3 and 2.11. The effective throat thickness of a fillet weld shall be the shortest distance from the root of the joint to the face of the diagrammatic weld, except that for fillet welds made by the submerged arc process, the effective throat thickness shall be taken equal to the leg size for 10 mm and smaller fillet welds, and equal to the theoretical throat plus 3 mm for fillet welds over 10 mm.

For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

TABLE 10.2-4 Minimum Size of Fillet Welds	
Material Thickness of Thicker Part Joined, mm	**Minimum Size of Fillet Weld[a] mm**
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6
Over 19	8
[a] Leg dimension of fillet welds. Single pass welds must be used. [b] See Section 10.2.2b for maximum size of fillet welds.	

10.2.2.2 Limitations

The *minimum size of fillet welds* shall be not less than the size required to transmit calculated forces nor the size as shown in Table 10.2-4, which is based upon experiences and provides some margin for uncalculated stress encountered during fabrication, handling, transportation, and erection. These provisions do not apply to fillet weld reinforcements of partial- or complete-joint-penetration welds.

The *maximum size of fillet welds* of connected parts shall be:

(a) Along edges of material less than 6 mm thick, not greater than the thickness of the material.

(b) Along edges of material 6 mm or more in thickness, not greater than the thickness of the material minus 2 mm, unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 2 mm provided the weld size is clearly verifiable.

For flange-web welds and similar connections, the actual weld size need not be larger than that required to develop the web capacity, and the requirements of Table

10.2-4 need not apply.

The *minimum effective length of fillet welds* designed on the basis of strength shall be not less than four times the nominal size, or else the size of the weld shall be considered not to exceed ¼ of its effective length. If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. For the effect of longitudinal fillet weld length in end connections upon the effective area of the connected member, see Section 2.3.

For end-loaded fillet welds with a length up to 100 times the leg dimension, it is permitted to take the effective length equal to the actual length. When the length of the end-loaded fillet weld exceeds 100 times the weld size, the effective length shall be determined by multiplying the actual length by the reduction factor, β.

where

$$\beta = 1.2 - 0.002(L/w) \leq 1.0 \qquad \textbf{(10.2-1)}$$

L = actual length of end-loaded weld, mm

w = weld leg size, mm

When the length of the weld exceeds 300 times the leg size, the value of β shall be taken as 0.60.

Intermittent fillet welds may be used to transfer calculated stress across a joint or faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of built-up members. The effective length of any segment of intermittent fillet welding shall be not less than four times the weld size, with a minimum of 38 mm.

In lap joints, the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than 25 mm. Lap joints joining plates or bars subjected to axial stress that utilize transverse fillet welds only shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

Fillet weld terminations are permitted to extend to the ends or sides of parts or be stopped short or boxed except as limited by the following:

(1) For lap joints in which one part extends beyond an edge subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from that edge.

(2) For connections and structural elements with cyclic forces, normal to outstanding legs, of frequency and magnitude that would tend to cause a progressive fatigue failure initiating from a point of maximum stress at the end of the weld, fillet welds shall be returned around the corner for a distance not less than the smaller of two times the weld size or the width of the part.

(3) For connections whose design requires flexibility of the outstanding legs, if end returns are used, their length shall not exceed four times the nominal size of the weld.

(4) Fillet welds joining transverse stiffeners to plate girder webs shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of stiffeners are welded to the flange.

(5) Fillet welds, which occur on opposite sides of a common plane, shall be interrupted at the corner common to both welds.

Fillet welds in holes or slots may be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap, subject to the provisions of Section 10.2. Fillet welds in holes or slots are not to be considered plug or slot welds.

10.2.3 Plug and Slot Welds

10.2.3.1 Effective Area. The effective shearing area of plug and slot welds shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the faying surface

10.2.3.2 Limitations. Plug or slot welds are permitted to be used to transmit shear in lap joints or to prevent buckling of lapped parts and to join component parts of built-up members.

The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus 8 mm, rounded to the next larger even mm, nor greater than the minimum diameter plus 3 mm or 2.25 times the thickness of the weld. The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus 8 mm rounded to the next larger even mm, nor shall it be larger than 2.25 times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot. The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

The thickness of plug or slot welds in material 16 mm or less in thickness shall be equal to the thickness of the material. In material over 16 mm thick, the thickness of the weld shall be at least one-half the thickness of the material but not less than 16 mm.

10.2.4 Design Strength. The design strength of welds shall be the lower value of (a) $\phi F_{BM} A_{BM}$ and (b) $\phi F_w A_w$, when applicable. The values of ϕ, F_{BM}, and F_w and limitations thereon are given in Table 10.2-5,

where

F_{BM} = nominal strength of the base material, MPa

F_w = nominal strength of the weld electrode, MPa

A_{BM} = cross-sectional area of the base material, mm^2

A_w = effective cross-sectional area of the weld, mm^2

ϕ = resistance factor

Alternatively, in lieu of the constant design strength for fillet welds given in Table 10.2-5, fillet welds loaded in-plane are permitted to be designed in accordance with the following procedure.

(a) For a linear weld group loaded in-plane through the center of gravity, the design strength is $\phi F_w A_w$,

where

$$F_w = 0.60\, F_{EXX}(1.0 + 0.50 \sin^{1.5}\theta) \qquad \textbf{(10.2-2)}$$

ϕ = 0.75

F_{EXX} = electrode classification number, i.e., minimum specified strength, MPa

θ = angle of loading measured from the weld longitudinal axis, degrees

A_w = effective area of weld throat, mm^2.

(b) For weld elements within a weld group that are loaded in-plane and analyzed using an instantaneous center of rotation method, the components of the design strength are $\phi\ F_{wx}\ A_w$ and $\phi\ F_{wy}\ A_w$

Where:

$F_{wx} = \Sigma F_{wix}$

$F_{wy} = \Sigma F_{wiy}$

$F_{wi} = 0.60\, F_{EXX}\,(1.0 + 0.50 \sin^{1.5}\theta)\, f(p)$

$f(p) = [p\,(1.9 - 0.9p)]^{0.3}$

$\phi = 0.75$

where

F_{wi} = nominal stress in any ith weld element, MPa

F_{wix} = x component of stress F_{wi}

F_{wiy} = y component of stress F_{wi}

p = Δ_i / Δ_m, ratio of element i deformation to its deformation at maximum stress

Δ_m = $0.209\,(\theta + 2)^{-0.32}\, w$, deformation of weld element at maximum stress, mm

Δ_i = deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, mm.

= $r_i\ \Delta_u / r_{crit}$

$\Delta_{\underline{u}}$ = $1.087(\theta + 6)^{-0.65} w \leq 0.17w$, deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, mm

w = leg size of the fillet weld, mm

r_{crit} = distance from instantaneous center of rotation to weld element with minimum $\Delta_{\underline{u}} / r_i$ ratio, mm

10.2.5 **Combination of Welds.** If two or more of the general types of welds (groove, fillet, plug, slot) are combined in a single joint, the design strength of each shall be separately computed with reference to the axis of the group in order to determine the design strength of the combination.

TABLE 10.2-5 Design Strength of Welds				
Types of Weld and Stress [a]	**Material**	**Resistance Factor ~**	**Nominal Strength F_{BM} or Fw**	**Filler Metal Requirements [b, c]**
Complete-Joint-Penetration Groove Weld				
Tension normal to Effective area	Base	0.90	F_y	Matching filler metal shall be used. For CVN requirements see footnote [d].
Compression normal to effective area	Base	0.90	F_y	Filler metal with a strength level equal to or less than matching filler metal is permitted to be used.
Tension or compression parallel to axis of weld				
Shear on effective Area	Base Weld	0.90 0.80	$0.60F_y$ $0.60F_{EXX}$	
Partial-Joint-Penetration Groove Weld				
Compression normal to effective area	Base	0.90	F_y	Filler metal with a strength level equal to or less than matching filler metal is permitted to be used.
Tension or compression parallel to axis of weld [e]				
Shear parallel to axis of weld	Base Weld	[f] 0.75	[f] $0.60F_{EXX}$	
Tension normal to Effective area	Base Weld	0.90 0.80	F_y $0.60F_{EXX}$	
Fillet Welds				
Shear on effective Area	Base Weld	[f] 0.75	[f] $0.60F_{EXX}$ [g]	Filler metal with a strength level equal to or less than
Tension or compression parallel to axis of weld [e]	Base	0.90	F_y	matching filler metal is permitted to be used.
Plug or Slot Welds				
Shear parallel to faying surfaces (on Effective area)	Base Weld	[f] 0.75	[f] $0.60F_{EXX}$	Filler metal with a strength level equal to or less than matching filler metal is permitted to be used.

[a] For definition of effective area, see Section 10.2.
[b] For matching filler metal, see Table 3.1, AWS D1.1.
[c] Filler metal one strength level stronger than matching filler metal is permitted.
[d] For T and corner joints with the backing bar left in place during service, filler metal with a classification requiring a minimum Charpy V-notch (CVN) toughness of 27 J @ 4°C shall be used. If filler metal without the required toughness is used and the backing bar is left in place, the joint shall be sized using the resistance factor and nominal strength for a partial-joint-penetration weld.
[e] Fillet welds and partial-joint-penetration groove welds joining component elements of built-up members, such as flange-to-web connections, are not required to be designed with Regard to the tensile or compressive stress in these elements parallel to the axis of the welds.
[f] The design of connected material is governed by Sections 10.4 and 10.5.
[g] For alternative design strength, see below.

10.2.6 **Weld Metal Requirements.** The choice of electrode for use with complete-joint-penetration groove welds subject to tension normal to the effective area shall comply with the requirements for matching weld metals given in AWS D1.1.

Weld metal with a specified Charpy V-notch (CVN) toughness of 27 J at 4°C shall be used in the following joints:

(a) Complete-joint-penetration groove welded T and corner joints with steel backing left in place, subject to tension normal to the effective area, unless the joints are designed as noted in Table 10.2-5 (see footnote d).

(b) Complete-joint-penetration groove welded splices subject to tension normal to the effective area in Group 4 and Group 5 shapes and shapes built up by welding plates more than 50 mm thick.

The manufacturer's Certificate of Conformance shall be sufficient evidence of compliance.

10.2.7 **Mixed Weld Metal.** When notch-toughness is specified, the process consumables for all weld metal, tack welds, root pass, and subsequent passes deposited in a joint shall be compatible to assure notch-tough composite weld metal.

10.2.8 **Preheat for Heavy Shapes.** For ASTM A6/A6M Group 4 and 5 and equivalent shapes and welded built-up members made of plates more than 50 mm thick, a preheat equal to or greater than 350°F (175°C) shall be used when making groove-weld splices.

SECTION 10.3
BOLTS AND THREADED PARTS

10.3.1 **High-Strength Bolts.** Use of high-strength bolts shall conform to the provisions of the *Load and Resistance Factor Design specification for Structural Joints Using ASTM A325 or A490 Bolts,* as approved by the Research Council on Structural Connections, except as otherwise provided in this code.

If required to be tightened to more than 50 percent of their specified minimum tensile strength, A449 bolts in tension and bearing-type shear connections shall have an ASTM F436 hardened washer installed under the bolt head, and the nuts shall meet the requirements of ASTM A563. When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale. All A325 or A325M and A490 or A490M bolts shall be tightened to a bolt tension not less than that given in Table 10.3-1, except as noted below. Tightening shall be done by any of the following methods: turn-of-nut method, a direct tension indicator, calibrated wrench, or alternative design bolt.

TABLE 10.3-1
Minimum Bolt Pretension, kN*

Bolt Size, mm	A325M Bolts	A490M Bolts
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334
M30	326	408
M36	475	595

*Equal to 0.70 of minimum tensile strength of bolts, rounded off to nearest kN, as specified in ASTM specifications for A325M and A490M bolts with UNC threads.

Bolts need only be tightened to the snug-tight condition when in: (a) bearing-type connections where slip is permitted, or (b) tension or combined shear and tension applications, for ASTM A325 or A325M bolts only, where loosening or fatigue due to vibration or load fluctuations are not design considerations. The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact. The nominal strength value given in Table 10.3-2 and Table 10.3-5 shall be used for bolts tightened to the snug-tight condition. Bolts tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings.

When A490 or A490M bolts over 25 mm in diameter are used in slotted or oversize holes in external plies, a single hardened washer conforming to ASTM F436, except with 8 mm minimum thickness, shall be used in lieu of the standard washer. In slip-critical connections in which the direction of loading is toward an edge of a connected part, adequate design bearing strength shall be provided based upon the applicable requirements of Section 10.3.10.

TABLE 10.3-2
Design Strength of Fasteners

Description of Fasteners	Tensile Strength		Shear Strength in Bearing-type Connections	
	Resistance Factor ϕ	Nominal Strength, MPa	Resistance Factor ϕ	Nominal Strength, MPa
A307 bolts	0.75	310 [a]	0.75	165 [b,e]
A325 or A325M bolts, when threads are not excluded from shear planes		620 [d]		330 [e]
A325 or A325M bolts, when threads are excluded from shear planes		620 [d]		414 [e]
A490 or A490M bolts, when threads are not excluded from shear planes		780 [d]		414 [e]
A490 or A490M bolts, when threads are excluded from shear planes		780 [d]		520 [e]
Threaded parts meeting the requirements of Section A3, when threads are not excluded from shear planes		$0.75F_u$ [a,c]		$0.40F_u$
Threaded parts meeting the requirements of Section A3, when threads are excluded from shear planes		$0.75F_u$ [a,c]		$0.50F_u$ [a,c]
A502, Gr. 1, hot-driven Rivets		310 [a]		172 [e]
A502, Gr. 2 & 3, hot-driven Rivets		414 [a]		228 [e]

[a] Static loading only.

[b] Threads permitted in shear planes.

[c] The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, A_D shall be larger than the nominal body area of the rod before upsetting times F_y.

[d] For A325 or A325M and A490 or A490M bolts subject to tensile fatigue loading, see Section 11.3.

[e] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 1270 mm, tabulated values shall be reduced by 20 percent.

10.3.2 Size and Use of Holes. The *maximum sizes* of holes for rivets and bolts are given in Table 10.3-3, except that larger holes, required for tolerance on location of anchor rods in concrete foundations, are allowed in column base details.

Standard holes shall be provided in member-to-member connections, unless over-sized, short-slotted, or long-slotted holes in bolted connections are approved by the designer. Finger shims up to 6 mm are permitted in slip-critical connections designed on the basis of standard holes without reducing the nominal shear strength of the fastener to that specified for slotted holes.

Oversized holes are allowed in any or all plies of slip-critical connections, but they shall not be used in bearing-type connections. Hardened washers shall be installed over oversized holes in an outer ply.

TABLE 10.3-3
Nominal Hole Dimensions, mm

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-slot (Width x Length)	Long-slot (Width x Length)
M16	18	20	18 × 22	18 × 40
M20	22	24	22 × 26	22 × 50
M22	24	28	24 × 30	24 × 55
M24	27 [a]	30	27 × 32	27 × 60
M27	30	35	30 × 37	30 × 67
M30	33	38	33 × 40	33 × 75
$\geq$M36	$d + 3$	$d + 8$	$(d + 3) \times (d + 10)$	$(d + 3) \times 2.5d$

[a] clearance provided allows the use of a 25 mm, bolt, if desirable.

Short-slotted holes are allowed in any or all plies of slip-critical or bearing-type connections. The slots are permitted to be used without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

Long-slotted holes are allowed in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes are permitted to be used without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 8 mm thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

10.3.3 Minimum Spacing. The distance between centers of standard, oversized, or slotted holes, shall not be less than 2.67 times the nominal diameter of the fastener; a distance of $3d$ is preferred. Refer to Section 10.3.10 for bearing strength requirements.

10.3.4 **Minimum Edge Distance.** The distance from the center of a standard hole to an edge of a connected part shall not be less than either the applicable value from Table 10.3-4, or as required in Section 10.3.10. The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole to an edge of a connected part plus the applicable increment C_2 from Table 10.3-6. Refer to Section 10.3.10 for bearing strength requirements.

TABLE 10.3-4
Minimum Edge Distance,[a] mm , From Center of Standard Hole[b] to Edge of Connected Part

Nominal Rivet or Bolt Diameter (mm)	At Sheared Edges	At Rolled Edges of Plates, Gas Cut Edges[c]
16	28	22
20	34	26
22	38[d]	28
24	42[d]	30
27	48	34
30	52	38
36	64	46
Over 36	1.75d	1.25d

[a] Lesser edge distances are permitted to be used provided Equations from Section 10.3.10, as appropriate, are satisfied.
[b] For oversized or slotted holes, see Table 10.3-6.
[c] All edge distances in this column are permitted to be reduced 3 mm when the hole is at a point where stress does not exceed 25 percent of the maximum design strength in the element.
[d] These are permitted to be 32 mm at the ends of beam connection angles and shear end plates.

10.3.5 **Maximum Spacing and Edge Distance.** The maximum distance from the center of any bolt or rivet to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 150 mm. The longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates shall be as follows:

(a) or painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 305 mm.
(b) For unpainted members of weathering steel subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or 180 mm.

TABLE 10.3-5
Nominal Tension Stress (F_t), MPa
Fasteners in Bearing-type Connections

Description of Fasteners	Threads Included in the Shear Plane	Threads Excluded from the Shear Plane
A307 bolts	$407 - 2.5f_v \leq 310$	
A325M bolts	$807 - 2.5f_v \leq 621$	$807 - 2.0f_v \leq 621$
A490M bolts	$1010 - 2.5f_v \leq 779$	$1010 - 2.0f_v \leq 779$
Threaded parts A449 bolts Over 38 mm diameter	$0.98F_u - 2.5f_v \leq 0.75F_u$	$0.98F_u - 2.0f_v \leq 0.75F_u$
A502 Gr. 1 rivets	$407 - 2.4f_v \leq 310$	
A502 Gr. 2 rivets	$538 - 2.4f_v \leq 414$	

TABLE 10.3-5A Nominal Tension Stress (F_t), MPa Fasteners in Bearing-type Connections		
Description of Fasteners	**Threads Included in the Shear Plane**	**Threads Excluded from the Shear Plane**
A307 bolts	$\sqrt{310^2 - 6.25 f_v^2}$	
A325M bolts	$\sqrt{621^2 - 6.25 f_v^2}$	$\sqrt{621^2 - 4.00 f_v^2}$
A490M bolts	$\sqrt{779^2 - 6.31 f_v^2}$	$\sqrt{779^2 - 4.04 f_v^2}$
Threaded parts A449 bolts over 38 mm	$\sqrt{(0.75F_u)^2 - 6.25 f_v^2}$	$\sqrt{(0.75F_u)^2 - 4.00 f_v^2}$
A502 Gr. 1 rivets	$\sqrt{310^2 - 5.76 f_v^2}$	
A502 Gr. 2 rivets	$\sqrt{414^2 - 5.86 f_v^2}$	

10.3.6 **Design Tension or Shear Strength.** The design tension or shear strength of a high-strength bolt or threaded part is

$$\phi F_n A_b,$$

where

ϕ = resistance factor tabulated in Table 10.3-2

F_n = nominal tensile strength F_t, or shear strength, F_v, tabulated in Table 10.3-2, MPa

A_b = nominal unthreaded body area of bolt or threaded part (for upset rods, see Footnote c, Table 10.3-2), mm^2

The applied load shall be the sum of the factored loads and any tension resulting from prying action produced by deformation of the connected parts.

10.3.7 **Combined Tension and Shear in Bearing-Type Connections.** The design strength of a bolt or rivet subject to combined tension and shear is

$$\phi F_v A_b,$$

where

ϕ = 0.75

F_t = nominal tension stress computed from the equations in Table 10.3-5 as a function of f_v, the required shear stress produced by the factored loads. Alternately, the use of the equations in Table 10.3-5A is permitted. The design shear strength ϕF_v tabulated in Table 10.3-2 shall equal or exceed the shear stress, f_v.

10.3.8 **High-Strength Bolts in Slip-Critical Connections.** The design for shear of high-strength bolts in slip-critical connections shall be in accordance with Section 10.3.8 and checked for shear in accordance with Sections 10.3.6 and 10.3.7 and bearing in accordance with Sections 10.3.1 and 10.3.10.

10.3.8.1 **Slip-Critical Connections Designed at Factored Loads.** The design slip resistance per bolt, ϕr_{str}, shall equal or exceed the required force per bolt due to factored loads,

$$r_{str} = 1.13\ \mu_{\mu} T_b N_s \qquad \textbf{(10.3-1)}$$

where:

T_b = minimum fastener tension given in Table 10.3-1, kN

N_s = number of slip planes

μ = mean slip coefficient for Class A, B, or C surfaces, as applicable, or as established by tests

(a) For Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel),

$$\mu = 0.33$$

(b) For Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel),

$$\mu = 0.50$$

(c) For Class C surfaces (hot-dip galvanized and roughened surfaces),

$$\mu = 0.35$$

ϕ = resistance factor

(a) For standard holes, $\phi = 1.0$

(b) For oversized and short-slotted holes, $\phi = 0.85$

(c) For long-slotted holes transverse to the direction of load, $\phi = 0.70$

(d) For long-slotted holes parallel to the direction of load, $\phi = 0.60$

Finger shims up to 6 mm are permitted to be introduced into slip-critical connections designed on the basis of standard holes without reducing the design shear stress of the fastener to that specified for slotted holes.

TABLE 10.3-6
Values of Edge Distance Increment C_2, mm

Nominal Diameter of Fastener (mm)	Oversized Holes	Slotted Holes: Long Axis Perpendicular to Edge – Short Slots	Slotted Holes: Long Axis Perpendicular to Edge – Long Slots [a]	Slotted Holes: Long Axis Parallel to Edge
< 22	2	3	0.75d	0
24	3	3		
≥ 27	3	5		

[a] When length of slot is less than maximum allowable (see Table 10.3-5), C_2 are permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

10.3.9 **Combined Tension and Shear in Slip-Critical Connections.** The design of slip-critical connections subject to tensile forces shall be in accordance with either Sections 10.3.9.1 and 10.3.8.1 or Sections 10.3.9.2 and 10.3.8.2.

10.3.9.1 **Slip-Critical Connections Designed at Factored Loads.** When a slip-critical connection is subjected to an applied tension T_u that reduces the net clamping force, the slip resistance ϕr_{str} according to Section 10.3.8.1, shall be multiplied by the following factor:

$$1 - \{ T_u / (1.13\ T_b N_b) \}$$

where

T_b = minimum bolt pretension from Table 10.3-1, kN

N_b = number of bolts carrying factored-load tension T_u

10.3.10 **Bearing Strength at Bolt Holes.** Bearing strength shall be checked for both bearing-type and slip-critical connections. The use of oversized holes and short- and long-slotted holes parallel to the line of force is restricted to slip-critical connections per Section 10.3.2. The design bearing strength at bolt holes is ϕR_n,

where

$\phi = 0.75$

and R_n is determined as follows:

(a) For a bolt in a connection with standard, oversized, and short-slotted holes in-dependent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force:

when deformation at the bolt hole at service load is a design consideration:

$$R_n = 1.2L\ tF_u \leq 2.4\ dtF_u \quad \textbf{(10.3-2a)}$$

when deformation at the bolt hole at service load is not a design consideration:

$$R_n = 1.5L_c tF_u \leq 3.0\ dtF_u \quad \textbf{(10.3-2b)}$$

(b) For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force:

$$R_n = 1.0L_c tF_u \leq 2.0\ dtF_u \quad \textbf{(10.3-2c)}$$

In the foregoing,

R_n = nominal bearing strength of the connected material, kips (N)

F_u = specified minimum tensile strength of the connected material, MPa

L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, mm

d = nominal bolt diameter, mm

t = thickness of connected material, mm

For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts.

10.3.11 **Long Grips.** A307 bolts providing design strength, and for which the grip exceeds five diameters, shall have their number increased one percent for each additional 2 mm in the grip.

SECTION 10.4
DESIGN RUPTURE STRENGTH

10.4.1 **Shear Rupture Strength.** The design strength for the limit state of rupture along a shear failure path in the affected elements of connected members shall be taken as ϕR_n where

ϕ = 0.75

R_n = $F_u A_{nv}$

A_{nv} = net area subject to shear, mm^2

10.4.2 Tension Rupture Strength. The design strength for the limit state of rupture along a tension path in the affected elements of connected members shall be taken as ϕR_n where

ϕ = 0.75

R_n = $F_u A_{nt}$

A_{nt} = net area subject to tension, mm^2

10.4.3 Block Shear Rupture Strength. Block shear is a limit state in which the resistance is determined by the sum of the shear strength on a failure path(s) and the tensile strength on a perpendicular segment. It shall be checked at beam end connections where the top flange is coped and in similar situations, such as tension members and gusset plates. When ultimate rupture strength on the net section is used to determine the resistance on one segment, yielding on the gross section shall be used on the perpendicular segment. The block shear rupture design strength, ϕR_n, shall be determined as follows:

(a) When $F_u A_{nt} \geq 0.6 F_u A_{nv}$:

$$\phi R_n = \phi\ [0.6 F_y A_{gv} + F_u A_{nt}] \leq \phi\ [0.6 F_y A_{nv} + F_u A_{nt}] \qquad \textbf{(10.4-1)}$$

(b) When $F_u A_{nt} < 0.6 F_u A_{nv}$:

$$\phi R_n = \phi\ [0.6 F_u A_{nv} + F_y A_{gt}] \leq \phi\ [0.6 F_y A_{nv} + F_u A_{nt}] \qquad \textbf{(10.4-2)}$$

where

ϕ = 0.75

A_{gv} = gross area subject to shear, mm^2

A_{gt} = gross area subject to tension, mm^2

A_{nv} = net area subject to shear, mm^2

A_{nt} = net area subject to tension, mm^2

SECTION 10.5
CONNECTING ELEMENTS

This section applies to the design of connecting elements, such as plates, gussets, angles, brackets, and the panel zones of beam-to-column connections.

10.5.1 Eccentric Connections. Intersecting axially stressed members shall have their gravity axis intersect at one point, if practicable; if not, provision shall be made for bending and shearing stresses due to the eccentricity. Also see Section 10.1.8.

10.5.2 Design Strength of Connecting Elements in Tension. The design strength, ϕR_n, of welded, bolted, and riveted connecting elements statically loaded in tension (e.g., splice and gusset plates) shall be the lower value obtained according to limit states of yielding, rupture of the connecting element, and block shear rupture.

(a) For tension yielding of the connecting element: ϕ = 0.90

$$R_n = A_g F_y \qquad \textbf{(10.5-1)}$$

(b) For tension rupture of the connecting element: $\phi = 0.75$

$$R_n = A_n F_u \qquad \textbf{(10.5-2)}$$

where A_n is the net area, not to exceed $0.85A_g$.

(c) For block shear rupture of connecting elements, see Section 10.4.3.

10.5.3 **Other Connecting Elements.** For all other connecting elements, the design strength, ϕR_n, shall be determined for the applicable limit state to ensure that the design strength is equal to or greater than the required strength, where R_n is the nominal strength appropriate to the geometry and type of loading on the connecting element. For shear yielding of the connecting element:

$$\phi = 0.90$$

$$R_n = 0.60\, A_g F_y \qquad \textbf{(10.5-3)}$$

If the connecting element is in compression an appropriate limit state analysis shall be made.

SECTION 10.6 FILLERS

In welded construction, any filler 6 mm or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than 6 mm thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plus the thickness of the filler plate.

When a bolt that carries load passes through fillers that are equal to or less than 6 mm thick, the design shear strength shall be used without reduction. When a bolt that carries load passes through fillers that are greater than 6 mm thick, one of the following requirements shall apply:

(1) For fillers that are equal to or less than 19 mm thick, the design shear strength of the bolts shall be multiplied by the factor $[1 - 0.0154(t - 6)]$, where t is the total thickness of the fillers up to 19 mm.

(2) The fillers shall be extended beyond the joint and the filler extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross-section of the connected element and the fillers;

(3) The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or

(4) The joint shall be designed as a slip-critical joint.

SECTION 10.7 SPLICES

Groove-welded splices in plate girders and beams shall develop the full strength of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice.

SECTION 10.8
BEARING STRENGTH

The strength of surfaces in bearing is ϕR_n where

ϕ = 0.75

R_n is defined below for the various types of bearing

(a) For milled surfaces, pins in reamed, drilled, or bored holes, and ends of fitted bearing stiffeners,

$$R_n = 1.8F_y A_{pb} \quad \textbf{(10.8-1)}$$

where

F_y = specified minimum yield stress, MPa

A_{pb} = projected bearing area, mm^2

(b) For expansion rollers and rockers,

If $d \leq 635$ mm,

$$R_n = 1.2\,(F_y - 90)ld / 20 \quad \textbf{(10.8-2)}$$

If $d > 635$ mm,

$$R_n = 6.0(F_y - 90)l\sqrt{d} / 20 \quad \textbf{(10.8-3)}$$

where

d = diameter, mm

l = length of bearing, mm

SECTION 10.9
COLUMN BASES AND BEARING ON CONCRETE

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, design bearing loads on concrete may be taken as $\phi_c P_p$

(a) On the full area of a concrete support

$$P_p = 0.85 f'_c A_1 \quad \textbf{(10.9-1)}$$

(b) On less than the full area of a concrete support

$$P_p = 0.85 f'_c A_1 \sqrt{(A_2 / A_1)} \quad \textbf{(10.9-2)}$$

where

ϕ_c = 0.60

A_1 = area of steel concentrically bearing on a concrete support, mm^2

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm^2

$\sqrt{(A_2 / A_1)} \leq 2$

SECTION 10.10
ANCHOR RODS AND EMBEDMENTS

Steel anchor rods and embedments shall be proportioned to develop the factored load combinations stipulated in Section 1.4. If the load factors and combinations stipulated in Section 1.4 are used to design concrete structural elements, the provisions of SBC-304 shall be used with appropriate ϕ factors as given in SBC-304.

CHAPTER 11
CONCENTRATED FORCES, PONDING, AND FATIGUE

This chapter covers member strength design considerations pertaining to concentrated forces, ponding, and fatigue.

SECTION 11.1
FLANGES AND WEBS WITH CONCENTRATED FORCES

11.1.1 **Design Basis.** Sections 11.1.2 through 11.1.7 apply to single and double concentrated forces as indicated in each Section. A single concentrated force is tensile or compressive. Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member.

Transverse stiffeners are required at locations of concentrated tensile forces in accordance with Section 11.1.2 for the limit state of flange local bending, and at unframed ends of beams and girders in accordance with Section 11.1.8. Transverse stiffeners or doubler plates are required at locations of concentrated forces in accordance with Sections 11.1.3 through 11.1.6 for the limit states of web local yielding, crippling, sidesway buckling, and compression buckling. Doubler plates or diagonal stiffeners are required in accordance with Section 11.1.7 for the limit state of web panel-zone shear.

Transverse stiffeners and diagonal stiffeners required by Sections 11.1.2 through 11.1.8 shall also meet the requirements of Section 11.1.9. Doubler plates required by Sections 11.1.3 through 11.1.6 shall also meet the requirements of Section 11.1.10.

11.1.2 **Flange Local Bending.** This Section applies to both tensile single-concentrated forces and the tensile component of double-concentrated forces.

A pair of transverse stiffeners extending at least one-half the depth of the web shall be provided adjacent to a concentrated tensile force centrally applied across the flange when the required strength of the flange exceeds ϕR_n,

where

ϕ = 0.90

$$R_n = 6.25\, t_f^2\, F_{yf} \qquad \textbf{(11.1-1)}$$

F_{yf} = specified minimum yield stress of the flange, MPa

t_f = thickness of the loaded flange, mm

If the length of loading across the member flange is less than $0.15b$, where b is the member flange width, Equation 11.1-1 need not be checked.

When the concentrated force to be resisted is applied at a distance from the member end that is less than $10t_f$, R_n shall be reduced by 50 percent.

When transverse stiffeners are required, they shall be welded to the loaded flange to develop the welded portion of the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section 11.1.9.

11.1.3 **Web Local Yielding.** This Section applies to single-concentrated forces and both components of double-concentrated forces.

Either a pair of transverse stiffeners or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to a concentrated tensile or compressive force when the required strength of the web at the toe of the fillet exceeds ϕR_n,

where

ϕ = 1.0

and R_n is determined as follows:

(a) When the concentrated force to be resisted is applied at a distance from the member end that is greater than the depth of the member d,

$$R_n = (5k + N)F_{yw}\ t_w \quad \textbf{(11.1-2)}$$

(b) When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member d,

$$R_n = (2.5k + N)F_{yw}\ t_w \quad \textbf{(11.1-3)}$$

In Equations 11.1-2 and 11.1-3, the following definitions apply:

F_{yw} = specified minimum yield stress of the web, MPa

N = length of bearing (not less than k for end beam reactions), mm

k = distance from outer face of the flange to the web toe of the fillet, mm

t_w = web thickness, mm

When required, for a tensile force normal to the flange, transverse stiffeners shall be welded to the loaded flange to develop the connected portion of the stiffener. When required for a compressive force normal to the flange, transverse stiffeners shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section 11.1.9.

Alternatively, when doubler plates are required, see Section 11.1.10.

11.1.4 **Web Crippling.** This Section applies to both compressive single-concentrated forces and the compressive component of double-concentrated forces.

Either a transverse stiffener, a pair of transverse stiffeners, or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to a concentrated compressive force when the required strength of the web exceeds ϕR_n,

where

ϕ = 0.75

and R_n is determined as follows:

(a) When the concentrated compressive force to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$,

$$R_n = 0.80 t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} \qquad \textbf{(11.1-4)}$$

(b) When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than $d/2$,

For $N/d \le 0.2$,

$$R_n = 0.40 t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} \qquad \textbf{(11.1-5a)}$$

For $N/d > 0.2$,

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} \qquad \textbf{(11.1-5b)}$$

In Equations 11.1-4 and 11.1-5, the following definitions apply:

d = overall depth of the member, mm

t_f = flange thickness, mm

When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section 11.1.9.

Alternatively, when doubler plates are required, see Section 11.1.10.

11.1.5 **Web Sidesway Buckling.** This Section applies only to compressive single-concentrated forces applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force.

The design strength of the web is ϕR_n,

where

ϕ = 0.85

and R_n is determined as follows:

(a) If the compression flange is restrained against rotation:

For $(h/t_w)/(l/b_f) \le 2.3$,

$$R_n = \frac{C_r t_w^3 t_f}{h^2} \left[1 + 0.4 \left(\frac{h/t_w}{l/b_f} \right)^3 \right] \qquad \textbf{(11.1-6)}$$

for $(h / t_w) / (l / b_f) > 2.3$, the limit state of sidesway web buckling does not apply. When the required strength of the web exceeds ϕR_n, local lateral bracing shall be provided at the tension flange or either a pair of transverse stiffeners or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to the concentrated compressive force.

When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the full-applied force. The weld connecting transverse stiffeners to the web shall be sized to transmit the force in the stiffener to the web. Also, see Section 11.1.9.

Alternatively, when doubler plates are required, they shall be sized to develop the full-applied force. Also, see Section 11.1.10.

(b) If the compression flange is *not* restrained against rotation:
For $(h / t_w) / (l / b_f) \leq 1.7$,

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h / t_w}{l / b_f}\right)^3\right] \tag{11.1-7}$$

for $(h / t_w) / (l / b_f) > 1.7$, the limit state of sidesway web buckling does not apply. When the required strength of the web exceeds ϕR_n local lateral bracing shall be provided at both flanges at the point of application of the concentrated forces.

In Equations 11.1-6 and 11.1-7, the following definitions apply:

l = largest laterally unbraced length along either flange at the point of load, mm

b_f = flange width, mm

t_f = flange thickness, mm

t_w = web thickness, mm

h = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of fasteners or the clear distance between flanges when welds are used for built-up shapes, mm

C_r = $6.62 \text{x} 10^6$ MPa when $M_u < M_y$ at the location of the force
= $3.31 \text{ x } 10^6$ MPa when $M_u \geq M_y$ at the location of the force

11.1.6 **Web Compression Buckling.** This Section applies to a pair of compressive single-concentrated forces or the compressive components in a pair of double-concentrated forces, applied at both flanges of a member at the same location.

Either a single transverse stiffener, or pair of transverse stiffeners, or a doubler plate, extending the full depth of the web, shall be provided adjacent to concentrated compressive forces at both flanges when the required strength of the web exceed ϕR_n,

where

ϕ = 0.90

and

$$R_n = \frac{24t_w^3\sqrt{EF_{yw}}}{h} \tag{11.1-8}$$

When the pair of concentrated compressive forces to be resisted is applied at a distance from the member end that is less than $d/2$, R_n shall be reduced by 50 percent. When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section 11.1.9.

Alternatively, when doubler plates are required, see Section 11.1.10.

11.1.7 **Web Panel-Zone Shear.** Either doubler plates or diagonal stiffeners shall be provided within the boundaries of the rigid connection of members whose webs lie in a common plane when the required strength exceeds ϕR_v,

where

ϕ = 0.90

and R_v is determined as follows:

(a) When the effect of panel-zone deformation on frame stability is *not* considered in the analysis,

For $P_u \le 0.4P_y$

$$R_v = 0.60F_y d_c t_w \tag{11.1-9}$$

For $P_u > 0.4P_y$

$$R_v = 0.60F_y d_c t_w \left(1.4 - \frac{P_u}{P_y}\right) \tag{11.1-10}$$

(b) When frame stability, including plastic panel-zone deformation, is considered in the analysis:

For $P_u \le 0.75P_y$

$$R_v = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right) \tag{11.1-11}$$

For $P_u > 0.75P_y$

$$R_v = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2P_u}{P_y}\right) \tag{11.1-12}$$

In Equations 11.1-9 through 11.1-12, the following definitions apply:

t_w = column web thickness, in. (mm)

b_{cf} = width of column flange, in. (mm)

t_{cf} = thickness of the column flange, in. (mm)

d_b = beam depth, in. (mm)

d_c = column depth, in. (mm)

F_y = yield strength of the column web, ksi (MPa)

P_y = $F_y A$, axial yield strength of the column, kips (N)

A = column cross-sectional area, in.^{2} (mm^2)

When doubler plates are required, they shall meet the criteria of Section 6.2 and shall be welded to develop the proportion of the total shear force which is to be carried.

Alternatively, when diagonal stiffeners are required, the weld connecting diagonal stiffeners to the web shall be sized to transmit the stiffener force caused by unbalanced moments to the web. Also, see Section 11.1.9.

11.1.8 Unframed Ends of Beams and Girders. At unframed ends of beams and girders not otherwise restrained against rotation about their longitudinal axes, a pair of transverse stiffeners, extending the full depth of the web, shall be provided. Also, see Section 11.1.9.

11.1.9 Additional Stiffener Requirements for Concentrated Forces. Transverse and diagonal stiffeners shall also comply with the following criteria:

(1) The width of each stiffener plus one-half the thickness of the column web shall not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.

(2) The thickness of a stiffener shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated load, and not less than its width times $1.79\sqrt{F_y/E}$.

Full depth transverse stiffeners for compressive forces applied to a beam or plate girder flange shall be designed as axially compressed members (columns) in accordance with the requirements of Section 5.2 with an effective length of $0.75h$, a cross section composed of two stiffeners and a strip of the web having a width of $25t_w$ at interior stiffeners and $12t_w$ at the ends of members.

The weld connecting bearing stiffeners to the web shall be sized to transmit the excess web shear force to the stiffener. For fitted bearing stiffeners, see Section 10.8.

11.1.10 Additional Doubler Plate Requirements for Concentrated Forces. Doubler plates required by Sections 11.1.3 through 11.1.6 shall also comply with the following criteria:

(1) The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.

(2) The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.

SECTION 11.2
PONDING

The roof system shall be investigated by structural analysis to assure adequate strength and stability under ponding conditions, unless the roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater.

The roof system shall be considered stable and no further investigation is needed if:

$$C_p + 0.9C_s \leq 0.25 \quad \textbf{(11.2-1)}$$

$$I_d \geq 3\,940\, S^4 \quad \textbf{(11.2-2)}$$

where

$C_p = \dfrac{504 L_s L_p^4}{I_p}$

$C_s = \dfrac{504 S L_s^4}{I_s}$

L_p = column spacing in direction of girder (length of primary members), m

L_S = column spacing perpendicular to direction of girder (length of secondary members), m

S = spacing of secondary members, m

I_p = moment of inertia of primary members, mm^4

I_S = moment of inertia of secondary members, mm^4

I_d = moment of inertia of the steel deck supported on secondary members, mm^4 per m

For trusses and steel joists, the moment of inertia I_S shall be decreased 15 percent when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

SECTION 11.3
DESIGN FOR CYCLIC LOADING (FATIGUE)

Few members or connections in conventional buildings need to be designed for fatigue, since most load changes in such structures occur only a small number of times or produce only minor stress fluctuations. The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. However, crane runways and supporting structures for machinery and equipment are often subject to fatigue loading conditions.

This section applies to members and connections subject to high cycle loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure (fatigue).

11.3.1 **General.** The provisions of this section apply to stresses calculated on the basis of unfactored loads. The maximum permitted stress due to unfactored loads is $0.66F_y$.

Stress range is defined as the magnitude of the change in stress due to the application or removal of the unfactored live load. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation.

In the case of complete-joint-penetration butt welds, the maximum design stress range calculated by Equation 11.3-1 applies only to welds with internal soundness meeting the acceptance requirements of Section 6.12.2 or 6.13.2 of AWS D1.1. No evaluation of fatigue resistance is required if the live load stress range is less than the threshold stress range, F_{TH}. See Table 11.3-1.

No evaluation of fatigue resistance is required if the number of cycles of application of live load is less than 2 x 10^4.

The cyclic load resistance determined by this provision is applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions.

The cyclic load resistance determined by this provision is applicable only to structures subject to temperatures not exceeding 150°C.

The Engineer of Record shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the connections.

11.3.2 Calculation of Maximum Stresses and Stress Ranges. Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of prying action, if any.

In the case of axial stress combined with bending, the maximum stresses, of each kind, shall be those determined for concurrent arrangements of the applied load. For members having symmetric cross sections, the fasteners and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the stress range.

For axially loaded angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range.

11.3.3 Design Stress Range: The range of stress at service loads shall not exceed the stress range computed as follows.

(a) For stress categories A, B, B´, C, D, E and E´ the design stress range, F_{SR}, shall be determined by Equation 11.3-1.

$$F_{SR} = \left(\frac{C_f \times 327}{N} \right)^{0.333} \geq F_{TH} \qquad \textbf{(11.3-1)}$$

where

F_{SR} = Design stress range, MPa

C_f = Constant from Table 11.3-1 for the category

N = Number of stress range fluctuations in design life

= Number of stress range fluctuations per day × 365 × years of design life

F_{TH} = Threshold fatigue stress range, maximum stress range for indefinite design life from Table 11.3-1, MPa

(b) For stress Category F, the design stress range, F_{SR}, shall be determined by Equation 11.3-2.

$$F_{SR} = \left(\frac{C_f \times 11 \times 10^4}{N} \right)^{0.167} \geq F_{TH} \qquad \textbf{(11.3-2)}$$

(c) For tension-loaded plate elements connected at their end by cruciform, T- or corner details with complete-joint-penetration groove welds or partial-joint-penetration groove welds, fillet welds, or combinations of the preceding, transverse to the direction of stress, the design stress range on the cross section of the tension-loaded plate element at the toe of the weld shall be determined as follows:

Based upon crack initiation from the toe of the weld on the tension loaded plate element the design stress range, F_{SR}, shall be determined by Equation 11.3-1, for Category C which is equal to

$$F_{SR} = \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \geq 68.9$$

Based upon crack initiation from the root of the weld the design stress range, F_{SR}, on the tension loaded plate element using transverse partial-joint-penetration groove welds, with or without reinforcing or contouring fillet welds, the design stress range on the cross section at the toe of the weld shall be determined by Equation 11.3-3, Category C$'$ as follows:

$$F_{SR} = 1.72 R_{PJP} \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \qquad \textbf{(11.3-3)}$$

where

R_{PJP} = reduction factor for reinforced or non-reinforced transverse partial-joint-penetration (PJP) joints. Use Category C if R_{PJP} = 1.0.

$$= \left(\frac{0.65 - 0.59\left(\frac{2a}{t_p}\right) + 0.72\left(\frac{w}{t_p}\right)}{t_p^{0.167}} \right) \leq 1.0$$

$2a$ = the length of the non-welded root face in the direction of the thickness of the tension-loaded plate, mm

w = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, mm

t_p = thickness of tension loaded plate, mm

Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range, F_{SR}, on the cross section at the toe of the welds shall be determined by Equation 11.3-4, Category C as follows:

$$F_{SR} = 1.72 R_{FIL} \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \qquad \textbf{(11.3-4)}$$

where

R_{FIL} = reduction factor for joints using a pair of transverse fillet welds only. Use Category C if R_{FIL} = 1.0.

$$R_{FIL} = \left(\frac{0.06 + 0.72(w / t_p)}{t_p^{0.167}} \right) \leq 1.0$$

11.3.4 **Bolts and Threaded Parts.** The range of stress at service loads shall not exceed the stress range computed as follows.

(a) For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the design stress range computed using Equation 11.3-1 where C_f and F_{TH} are taken from Section 2 of Table 11.3-1.

(b) For high-strength bolts, common bolts, and threaded anchor rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial load and moment plus load due to prying action shall not exceed the design stress range computed using Equation 11.3-1. The factor C_f shall be taken as $3.9\text{x}10^8$ (as for Category E). The threshold stress, F_{TH} shall be taken as 48 MPa (as for category D). The net tensile area is given by Equation 11.3-5.

$$A_t = \frac{\pi}{4} (d_b - 0.9382\text{P})^2 \qquad \textbf{(11.3-5)}$$

where

P = pitch, mm per thread

d_b = the nominal diameter (body or shank diameter),mm

For joints in which the material within the grip is not limited to steel or joints which are not tensioned to the requirements of Table 10.3-1, all axial load and moment applied to the joint plus effects of prying action (if any) shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the grip is limited to steel and which are tensioned to the requirements of Table 10.3-1, an analysis of the relative stiffness of the connected parts and bolts shall be permitted to be used to determine the tensile stress range in the pretensioned bolts due to the total service live load and moment plus effects of prying action. Alternatively, the stress range in the bolts shall be assumed to be equal to the stress on the net tensile area due to 20 percent of the absolute value of the service load axial load and moment from dead, live and other loads.

11.3.5 **Special Fabrication and Erection Requirements.** Longitudinal backing bars are permitted to remain in place, and if used, shall be continuous. If splicing is necessary for long joints, the bar shall be joined with complete penetration butt joints and the reinforcement ground prior to assembly in the joint.

In transverse joints subject to tension, backing bars, if used, shall be removed and the joint back gouged and welded.

In transverse complete-joint-penetration T- and corner joints, a reinforcing fillet weld, not less than 6 mm in size shall be added at re-entrant corners.

The surface roughness of flame cut edges subject to significant cyclic tensile stress ranges shall not exceed 25 μm, where ASME B46.1 is the reference standard.

Re-entrant corners at cuts, copes and weld access holes shall form a radius of not less than 10 mm by pre-drilling or sub-punching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal surface.

For transverse butt joints in regions of high tensile stress, run-off tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Run-off tabs shall be removed and the end of the weld finished flush with the edge of the member.
See Section 10.2.2.2 *Fillet Weld Terminations* for requirements for end returns on certain fillet welds subject to cyclic service loading.

Table 11.3-1
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal, except non-coated weathering steel, with rolled or cleaned surface. Flame-cut edges with surface roughness value of ($25\mu m$) or less, but without re-entrant corners.	A	250×10^8	165	Away from all welds or structural connections
1.2 Non-coated weathering steel base metal with rolled or cleaned surface. Flame-cut edges with surface roughness value of $1000\mu in$ ($25\mu m$) or less, but without re-entrant corners.	B	120×10^8	110	Away from all welds or structural connections
1.3 Member with drilled or reamed holes. Member with re-entrant corners at copes, cuts, block-outs or other geometrical discontinuities made to requirements of Section 11.3.5, except weld access holes.	B	120×10^8	110	At any external edge or at hole perimeter
1.4 Rolled cross sections with weld access holes made to requirements of Sections 10.1.6 and 11.3.5. Members with drilled or reamed holes containing bolts for attachment of light bracing where there is a small longitudinal component of brace force.	C	44×10^8	69	At re-entrant corner of weld access hole or at any small hole (may contain bolt for minor connections)
SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS				
2.1 Gross area of base metal in lap joints connected by high-strength bolts in joints satisfying all requirements for slip-critical connections.	B	120×10^8	110	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections.	B	120×10^8	110	In net section originating at side of hole
2.3 Base metal at the net section of other mechanically fastened joints except eye bars and pin plates.	D	22×10^8	48	In net section originating at side of hole
2.4 Base metal at net section of eyebar head or pin plate.	E	11×10^8	31	In net section originating at side of hole

Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

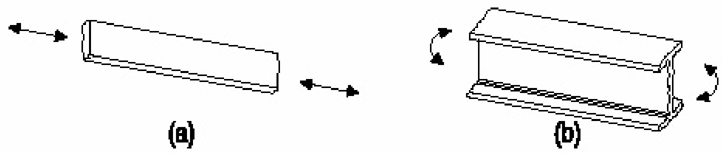
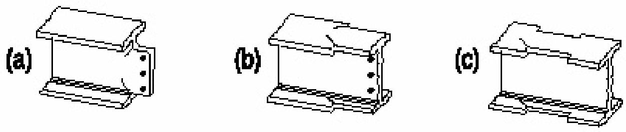
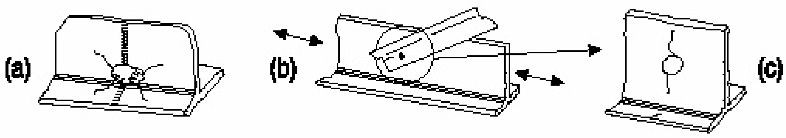
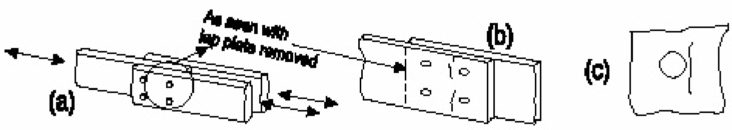
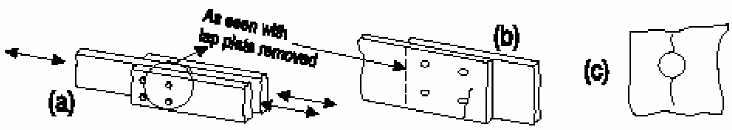
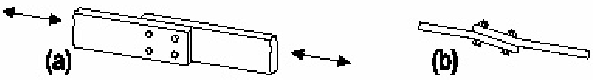
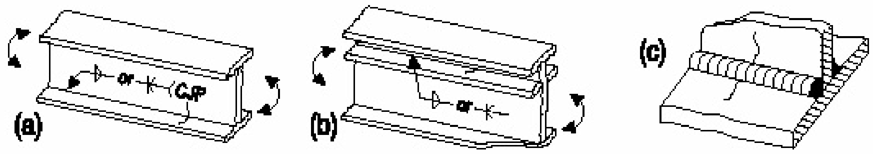
SECTION 1 - PLAIN MATERIAL AWAY FROM ANY WELDING
1.1 and 1.2 
1.3 
1.4 
SECTION 2 - CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS
2.1 
2.2 
2.3 
2.4 

Table 11.3-1(Cont'd) Fatigue Design Parameters				
Description	**Stress Category**	**Constant C_f**	**Threshold F_{TH} MPa**	**Potential Crack Initiation Point**
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete-joint-penetration groove welds, back gouged and welded from second side, or by continuous fillet welds.	B	120×10^8	110	From surface or internal discontinuities in weld away from end of weld
3.2 Base metal and weld metal in members without attachments built-up of plates or shapes, connected by continuous longitudinal complete penetration groove welds with backing bars not removed, or by continuous partial-joint-penetration groove welds.	B′	61×10^8	83	From surface or internal discontinuities in weld, including weld attaching backing bars
3.3 Base metal and weld metal termination of longitudinal welds at weld access holes in connected built-up members.	D	22×10^8	48	From the weld termination into the web or flange
3.4 Base metal at ends of longitudinal intermittent fillet weld segments.	E	11×10^8	31	In connected material at start and stop locations of any weld deposit
3.5 Base metal at ends of partial length welded cover-plates narrower than the flange having square or tapered ends, with or without welds across the ends of cover-plates wider than the flange with welds across the ends. Flange thickness $\leq$ 20 mm Flange thickness > 20 mm	 E E′	 11×10^8 3.9×10^8	 31 18	In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange with wide cover-plates
3.6 Base metal at ends of partial length welded cover-plates wider than the flange without welds across the ends.	E′	3.9×10^8	18	In edge of flange at end of cover-plate weld
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections. Welds shall be on each side of the axis of the member to balance weld stresses. $t \leq$ 13 mm t > 13 mm	 E E′	 11×10^8 3.9×10^8	 31 18	Initiating from end of any weld termination extending into the base metal

Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

SECTION 3 - WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS

3.1



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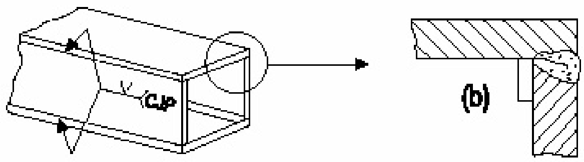

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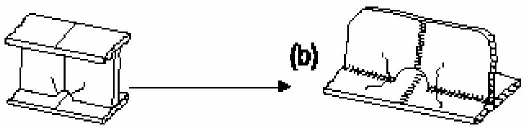

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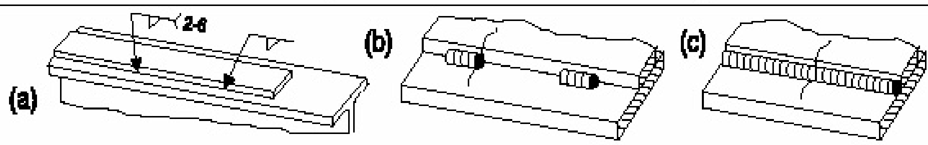

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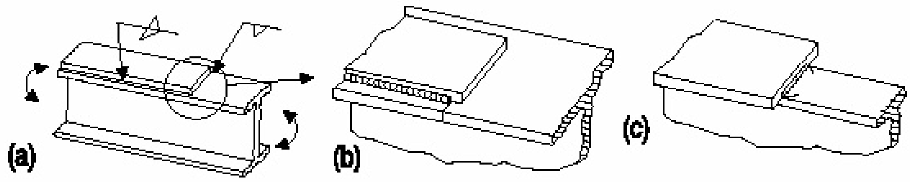
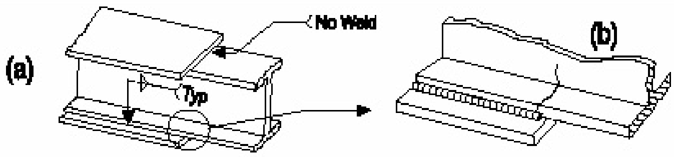


3.6



SECTION 4 - LONGITUDINAL FILLET WELDED END CONNECTIONS

4.1

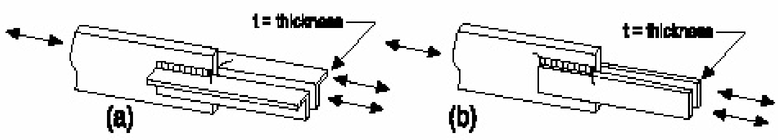


Table 11.3-1(Cont'd)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Base metal and weld metal in or adjacent to complete joint penetration groove welded splices in rolled or welded cross sections with welds ground essentially parallel to the direction of stress.	B	120 x 10^a	110	From internal discontinuities in filler metal or along the fusion boundary
5.2 Base metal and weld metal in or adjacent to complete joint penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in thickness or width made on a slope no greater than 8 to 20%. F_y < 620 MPa F_y ≥ 620 MPa	 B B'	 120 x 10^8 61 x 10^8	 110 83	From internal discontinuities in filler metal or along the fusion boundary or at start of transition when F_y ≥ 620 MPa
5.3 Base metal with F_y equal to or greater than 90 ksi (620 MPa) and weld metal in or adjacent to complete joint penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius of not less than 2 ft. (600 mm) with the point of tangency at the end of the groove weld.	B	120 x 10^a	110	From internal discontinuities in filler metal or discontinuities along the fusion boundary
5.4 Base metal and weld metal in or adjacent to the toe of complete joint penetration T or corner joints or splices, with or without transitions in thickness having slopes no greater than 8 to 20%, when weld reinforcement is not removed.	C	44 x 10^8	69	From surface discontinuity at toe of weld extending into base metal or along fusion boundary
5.5 Base metal and weld metal at transverse end connections of tension-loaded plate elements using partial joint penetration butt or T or corner joints, with reinforcing or contouring fillets, F_{SR} shall be the smaller of the toe crack or root crack stress range. Crack initiating from weld toe: Crack initiating from weld root:	 C C′	 44 x 10^8 (Equation 11.3-3)	 69 None provided	Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld

Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

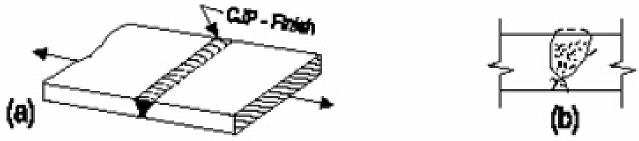
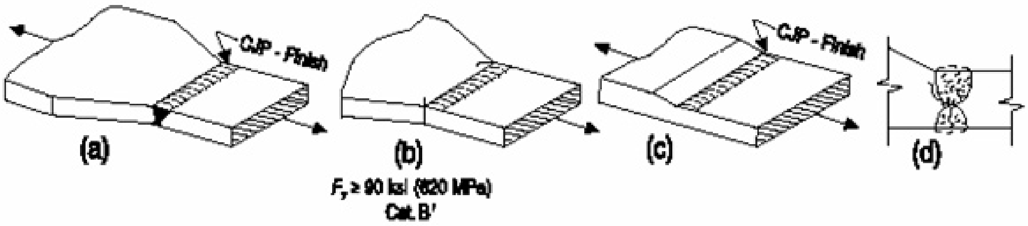
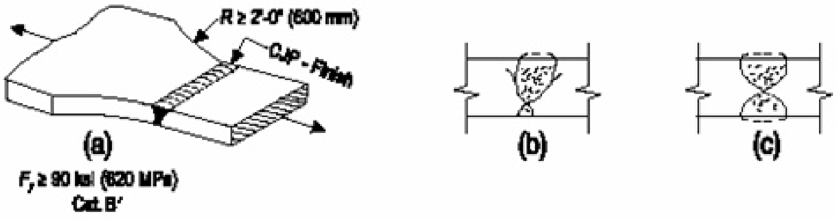
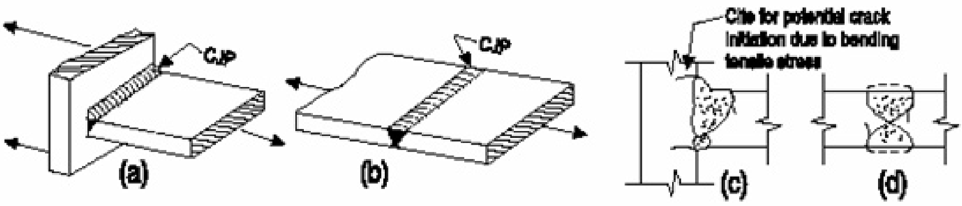
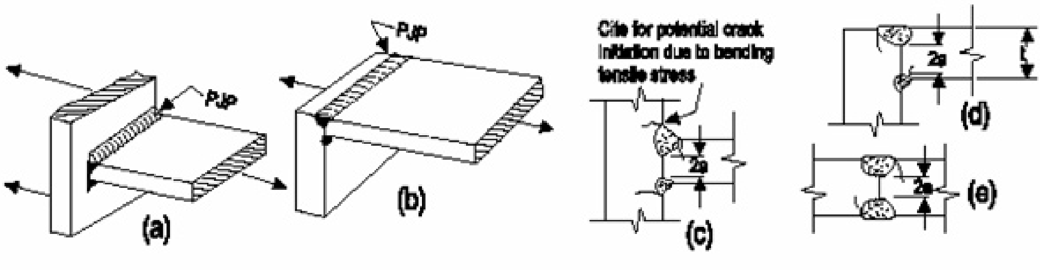
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS
5.1 
5.2 
5.3 
5.4 
5.5 

Table 11.3-1(Cont'd)
Fatigue Design Parameters

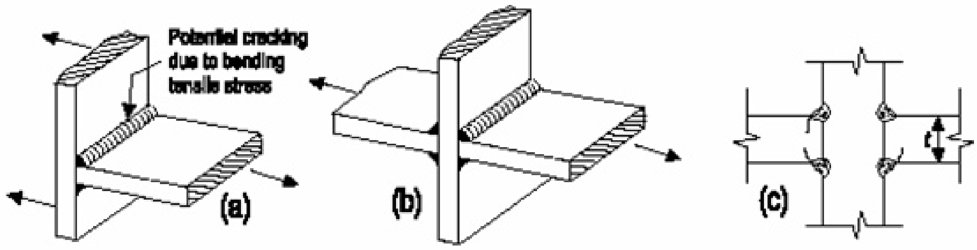
Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.6 Base metal and filler metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate. F_{SR} shall be the smaller of the toe crack or root crack stress range. Crack initiating from weld toe: Crack initiating from weld root:	 C C''	 44 x 10^8 (Equation 11.3-4)	 69 None provided	Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld
5.7 Base metal of tension loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.	C	44 x 10^8	69	From geometrical discontinuity at toe of fillet extending into base metal
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.1 Base metal at details attached by complete joint penetration groove welds subject to longitudinal loading only when the detail embodies a transition radius R with the weld termination ground smooth.				Near point of tangency of radius at edge of member
$R \geq$ 600 mm	B	120 x 10^8	110	
600 mm > $R \geq$ 150 mm	C	44 x 10^8	69	
150 mm > $R \geq$ 50 mm	D	22 x 10^8	48	
50 mm > R	E	11 x 10^8	31	

Table 11.3-1(Cont'd)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)

5.6



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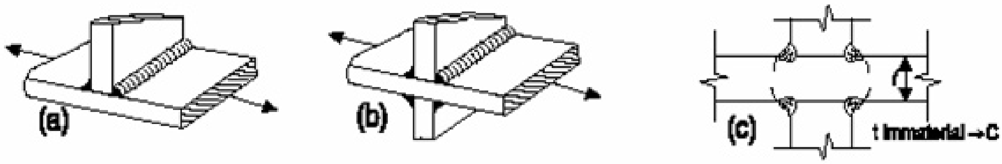


SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS

6.1

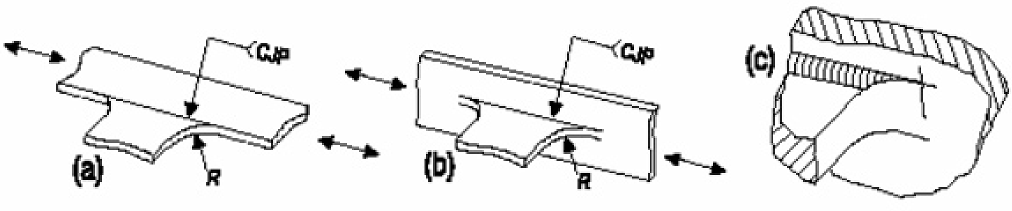


Table 11.3-1(Cont'd)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.2 Base metal at details of equal thickness attached by complete joint penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius *R* with the weld termination ground smooth.				
When weld reinforcement is removed:				
$R \geq 600$ mm	B	120×10^8	110	Near points of tangency of radius or in the weld or at fusion boundary or member or attachment
$600\text{ mm} > R \geq 150$ mm	C	44×10^8	69	
$150\text{ mm} > R \geq 50$ mm	D	22×10^8	48	
$50\text{ mm} > R$	E	11×10^8	31	
When weld reinforcement is not removed:				
$R \geq 600$ mm	C	44×10^8	69	At toe of the weld either along edge of member or the attachment
$600\text{ mm} > R \geq 150$ mm	C	44×10^8	69	
$150\text{ mm} > R \geq 50$ mm	D	22×10^8	48	
$50\text{ mm} > R$	E	11×10^8	31	
6.3 Base metal at details of unequal thickness attached by complete joint penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth.				
When weld reinforcement is removed:				
$R \geq 50$ mm	D	22×10^8	48	At toe of weld along edge of thinner material
$R \geq 50$ mm	E	11×10^8	31	In weld termination in small radius
When reinforcement is not removed:				
$R \geq 600$ mm	E	11×10^8	31	At toe of weld along edge of thinner material

Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

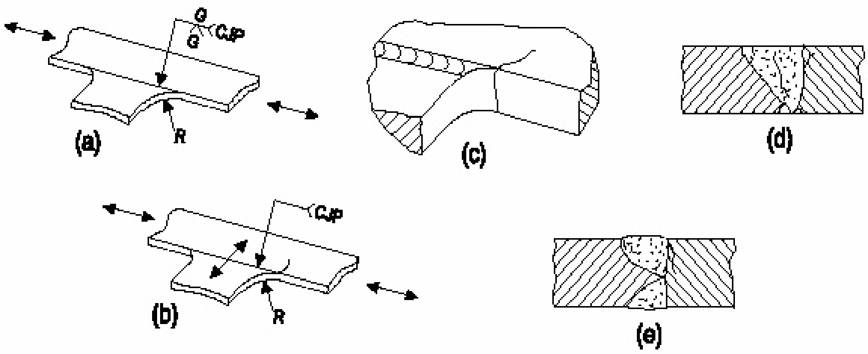
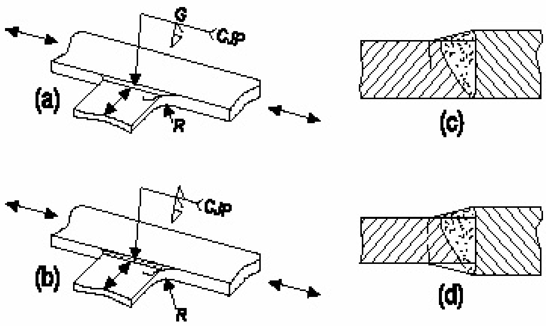
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)
6.2 
6.3 

Table 11.3-1(Cont'd)
Fatigue Design Parameters

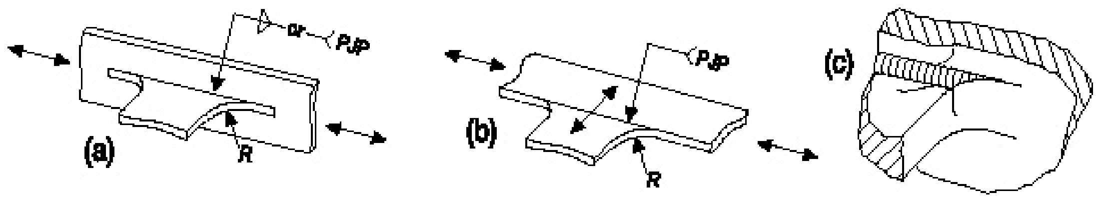
Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.4 Base metal subject to longitudinal stress at transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius *R*, with weld termination ground smooth.				In weld termination or from the toe of the weld extending into member
$R > 50$ mm	D	22×10^8	48	
$R \le 50$ mm	E	11×10^8	31	
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]				
7.1 Base metal subject to longitudinal loading at details attached by complete penetration groove welds parallel to direction of stress where the detail embodies a transition radius, *R*, less than 50 mm, and with detail length in direction of stress, a, and attachment height normal to surface of member, *b*:				In the member at the end of the weld
a < 50 mm	C	44×10^8	69	
50 mm $\le$ a $\le$ 12*b* or 100 mm	D	22×10^8	48	
a > 12*b* or 100 mm when *b* is $\le$ 25 mm	E	11×10^8	31	
a > 12*b* or 100 mm when *b* is > 25 mm	E	3.9×10^8	18	
7.2 Base metal subject to longitudinal stress at details attached by fillet or partial joint penetration groove welds, with or without transverse load on detail, when the detail embodies a transition radius, *R*, with weld termination ground smooth:				In weld termination extending into member
$R > 50$ mm	D	22×10^8	48	
$R \le 50$ mm	E	11×10^8	31	

[1] "Attachment" as used herein, is defined as any steel detail welded to a member which, by its mere presence and independent of its loading, causes a discontinuity in the stress flow in the member and thus reduces the fatigue resistance.

Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

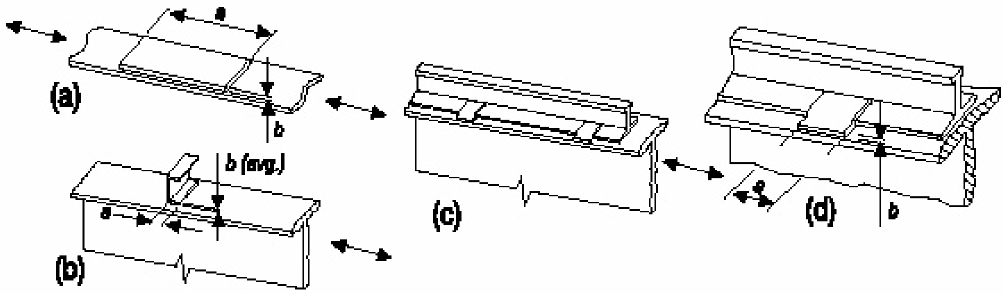
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.4



SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]

7.1



7.2

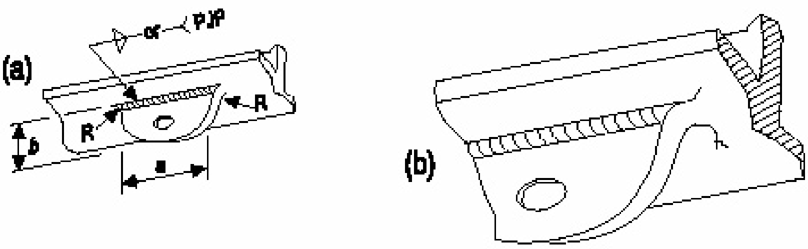


Table 11.3-1(Cont'd)
Fatigue Design Parameters

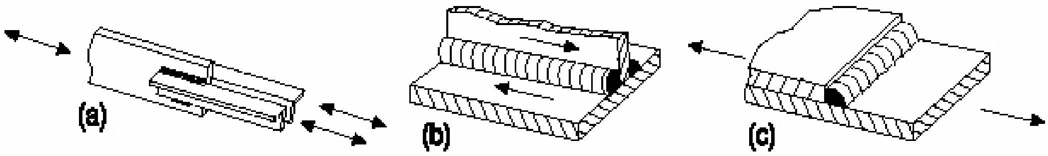
Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 8 – MISCELLANEOUS				
8.1 Base metal at stud-type shear connectors attached by fillet or electric stud welding	C	44 x 10^{a}	69	At toe of weld in base metal
8.2 Shear on throat of continuous or intermittent longitudinal or transverse fillet welds.	F	150 x 10^{8} (Equation 11.3-2)	55	In throat of weld
8.3 Base metal at plug or slot welds.	E	11 x 10^{8}	31	At end of weld in base metal
8.4 Shear on plug or slot welds.	F	150 x 10^{8} (Equation 11.3-2)	55	At faying surface
8.5 Not fully-tightened high-strength bolts, common bolts, threaded anchor rods and hanger rods with cut, ground or rolled threads. Stress range on tensile stress area due to live load plus prying action when applicable.	E'	3.9 x 10^{8}	48	At the root of the threads extending into the tensile stress area

Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

SECTION 8 - MISCELLANEOUS

8.1

8.2



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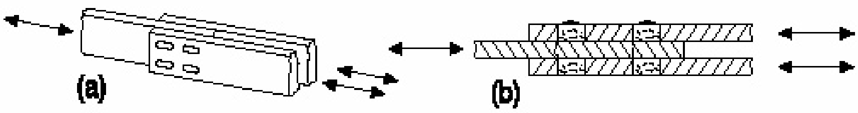

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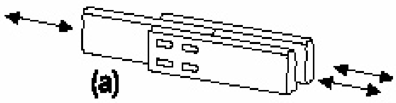

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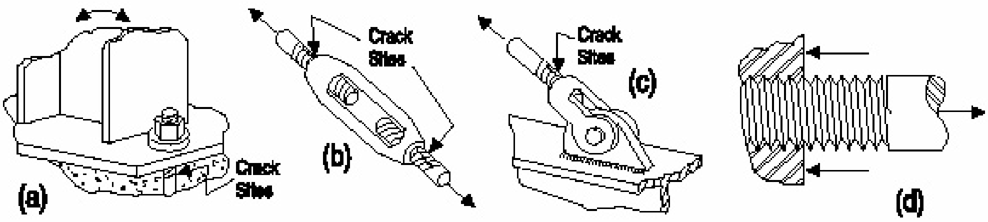

CHAPTER 12
SERVICEABILITY DESIGN CONSIDERATIONS

This chapter is intended to provide design guidance for serviceability considerations.

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage.

Limiting values of structural behavior to ensure serviceability (e.g., maximum deflections, accelerations, etc.) shall be chosen with due regard to the intended function of the structure. Where necessary, serviceability shall be checked using realistic loads for the appropriate serviceability limit state.

It is difficult to specify limiting values of structural performance based on serviceability considerations because these depend to a great extent on the type of structure, its intended use, and subjective physiological reaction. For example, acceptable structural motion in a hospital clearly would be much less than in an ordinary industrial building. It should be noted that humans perceive levels of structural motion that are far less than motions that would cause any structural damage. Serviceability limits must be determined through careful consideration by the designer and client.

SECTION 12.1
CAMBER

If any special camber requirements are necessary to bring a loaded member into proper relation with the work of other trades, as for the attachment of runs of sash, the requirements shall be set forth in the design documents. If camber involves the erection of any member under a preload, this shall be noted in the design documents.

Beams and trusses detailed without specified camber shall be fabricated so that after erection any camber due to rolling or shop assembly shall be upward.

SECTION 12.2
EXPANSION AND CONTRACTION

Adequate provision shall be made for expansion and contraction appropriate to the service conditions of the structure.

SECTION 12.3
DEFLECTIONS, VIBRATION, AND DRIFT

12.3.1 **Deflections.** Deformations in structural members and structural systems due to service loads shall not impair the serviceability of the structure.

12.3.2 **Floor Vibration.** Vibration shall be considered in designing beams and girders supporting large areas free of partitions or other sources of damping where excessive vibration due to pedestrian traffic or other sources within the building is not acceptable.

12.3.3 **Drift.** Lateral deflection or drift of structures due to code-specified wind or seismic loads shall not cause collision with adjacent structures nor exceed the limiting values of such drifts which may be specified or appropriate.

SECTION 12.4
CONNECTION SLIP

For the design of slip-critical connections see Sections 10.3.8 and 10.3.9.

SECTION 12.5
CORROSION

When appropriate, structural components shall be designed to tolerate corrosion or shall be protected against corrosion that may impair the strength or serviceability of the structure.

CHAPTER 13
FABRICATION, ERECTION, AND QUALITY CONTROL

This chapter provides requirements for shop drawings, fabrication, shop painting, erection, and quality control.

SECTION 13.1
SHOP DRAWINGS

Shop drawings giving complete information necessary for the fabrication of the component parts of the structure, including the location, type, and size of all welds, bolts, and rivets, shall be prepared in advance of the actual fabrication. These drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted connections. Shop drawings shall be made in conformity with good practice and with due regard to speed and economy in fabrication and erection.

SECTION 13.2
FABRICATION

13.2.1 **Cambering, Curving, and Straightening.** Local application of heat or mechanical means is permitted to be used to introduce or correct camber, curvature, and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 593°C for ASTM A514/A514M and ASTM A852/A852M steel nor 649°C for other steels.

13.2.2 **Thermal Cutting.** Thermally cut edges shall meet the requirements of AWS 5.15.1.2, 5.15.4.3 and 5.15.4.4 with the exception that thermally cut free edges which will be subject to calculated static tensile stress shall be free of round bottom gouges greater than 5 mm deep and sharp V-shaped notches. Gouges greater than 5 mm deep and notches shall be removed by grinding or repaired by welding.

Re-entrant corners, except re-entrant corners of beam copes and weld access holes, shall meet the requirements of AWS 5.16. If another specified contour is required it must be shown on the contract documents.

Beam copes and weld access holes shall meet the geometrical requirements of Section 10.1.6. For beam copes and weld access holes in ASTM A6/A6M Group 4 and 5 shapes and welded built-up shapes with material thickness greater than 50 mm, a preheat temperature of not less than 66°C shall be applied prior to thermal cutting.

13.2.3 **Planning of Edges.** Planning or finishing of sheared or thermally cut edges of plates or shapes is not required unless specifically called for in the design documents or included in a stipulated edge preparation for welding.

13.2.4 **Welded Construction.** The technique of welding, the workmanship, appearance, and quality of welds and the methods used in correcting nonconforming work shall be in accordance with AWS D1.1 except as modified in Section 10.2.

13.2.5 **Bolted Construction.** All parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a drift pin in bolt holes during assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

If the thickness of the material is not greater than the nominal diameter of the bolt plus 3 mm, the holes are permitted to be punched. If the thickness of the material is greater than the nominal diameter of the bolt plus 3 mm, the holes shall be either drilled or sub-punched and reamed. The die for all sub-punched holes, and the drill for all sub-drilled holes, shall be at least 2 mm smaller than the nominal diameter of the bolt. Holes in ASTM A514/A514M steel plates over 13 mm thick shall be drilled.

Fully-inserted finger shims, with a total thickness of not more than 6 mm within a joint, are permitted in joints without changing the design strength (based upon hole type) for the design of connections. The orientation of such shims is independent of the direction of application of the load.

The use of high-strength bolts shall conform to the requirements of the RCSC *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

13.2.6 **Compression Joints.** Compression joints which depend on contact bearing as part of the splice strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing, or other suitable means except where the surface is mill finished faces of plates or sections which are free from surface unevenness and mill scales.

13.2.7 **Dimensional Tolerances.** Dimensional tolerances shall be in accordance with the AISC *Code of Standard Practice*.

13.2.8 **Finish of Column Bases.** Column bases and base plates shall be finished in accordance with the following requirements:

(1) Steel bearing plates 50 mm or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 50 mm but not over 100 mm in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for all bearing surfaces (except as noted in subparagraphs 2 and 3 of this section), to obtain a satisfactory contact bearing. Steel bearing plates over 100 mm in thickness shall be milled for all bearing surfaces (except as noted in subparagraphs 2 and 3 of this section).

(2) Bottom surfaces of bearing plates and column bases which are grouted to ensure full bearing contact on foundations need not be milled.

(3) Top surfaces of bearing plates need not be milled when complete-joint-penetration groove welds are provided between the column and the bearing plate.

SECTION 13.3
SHOP PAINTING

13.3.1 **General Requirements.** Shop painting and surface preparation shall be in accordance with the provisions of the AISC *Code of Standard Practice*.
Shop paint is not required unless specified by the contract documents.

13.3.2 **Inaccessible Surfaces.** Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

13.3.3 **Contact Surfaces.** Paint is permitted unconditionally in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, paragraph 3(b).

13.3.4 **Finished Surfaces.** Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

13.3.5 **Surfaces Adjacent to Field Welds.** Unless otherwise specified in the design documents, surfaces within 50 mm of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

SECTION 13.4
ERECTION

13.4.1 **Alignment of Column Bases.** Column bases shall be set level and to correct elevation with full bearing on concrete or masonry.

13.4.2 **Bracing.** The frame of steel skeleton buildings shall be carried up true and plumb within the limits defined in the AISC *Code of Standard Practice*. Temporary bracing shall be provided, in accordance with the requirements of the *Code of Standard Practice*, wherever necessary to support all loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as required for safety.

13.4.3 **Alignment.** No permanent bolting or welding shall be performed until the adjacent affected portions of the structure have been properly aligned.

13.4.4 **Fit of Column Compression Joints and Base Plates.** Lack of contact bearing not exceeding a gap of 2 mm, regardless of the type of splice used (partial-joint-penetration groove welded, or bolted), is permitted. If the gap exceeds 2 mm, but is less than 6 mm, and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with non-tapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material.

13.4.5 **Field Welding.** Shop paint on surfaces adjacent to joints to be field welded shall be wire brushed if necessary to assure weld quality.
Field welding of attachments to installed embedments in contact with concrete shall be done in such a manner as to avoid excessive thermal expansion of the embedment which could result in spalling or cracking of the concrete or excessive stress in the embedment anchors.

13.4.6 **Field Painting.** Responsibility for touch-up painting, cleaning, and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the contract documents.

13.4.7 **Field Connections.** As erection progresses, the structure shall be securely bolted or welded to support all dead, wind, and erection loads.

SECTION 13.5
QUALITY CONTROL

The fabricator shall provide quality control procedures to the extent that the fabricator deems necessary to assure that all work is performed in accordance with this SBC 306. In addition to the fabricator's quality control procedures, material and workmanship at all times may be subject to inspection by qualified inspectors representing the purchaser. If such inspection by representatives of the purchaser will be required, it shall be so stated in the contract documents.

13.5.1 **Cooperation.** As far as possible, all inspection by representatives of the purchaser shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The purchaser's inspector shall schedule this work for minimum interruption to the work of the fabricator.

13.5.2 **Rejections.** Material or workmanship not in reasonable conformance with the provisions of SBC 306 may be rejected at any time during the progress of the work. The fabricator shall receive copies of all reports furnished to the purchaser by the inspection agency.

13.5.3 **Inspection of Welding.** The inspection of welding shall be performed in accordance with the provisions of AWS D1.1 except as modified in Section 10.2.
When visual inspection is required to be performed by AWS certified welding inspectors, it shall be so specified in the design documents.
When nondestructive testing is required, the process, extent, and standards of acceptance shall be clearly defined in the contract documents.

13.5.4 **Inspection of Slip-Critical High-Strength Bolted Connections.** The inspection of slip-critical high-strength bolted connections shall be in accordance with the provisions of the RCSC *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

13.5.5 **Identification of Steel.** The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material application and identification, visible at least through the "fit-up" operation, of the main structural elements of a shipping piece.

The identification method shall be capable of verifying proper material application as it relates to:

(1) Material specification designation

(2) Heat number, if required

(3) Material test reports for special requirements

CHAPTER 14
EVALUATION OF EXISTING STRUCTURES

This chapter applies to the evaluation of the strength and stiffness under static vertical (gravity) loads of existing structures by structural analysis, by load tests, or by a combination of structural analysis and load tests when specified by the Engineer of Record or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section 1.3.1. This chapter does not address load testing for the effects of seismic loads or moving loads (vibrations).

SECTION 14.1
GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the design strength of a load resisting member or system. The evaluation shall be performed by structural analysis (Section 14.3), by load tests (Section 14.4), or by a combination of structural analysis and load tests, as specified in the contract documents. Where load tests are used, the Engineer of Record shall first analyze the structure, prepare a testing plan, and develop a written procedure to prevent excessive permanent deformation or catastrophic collapse during testing.

SECTION 14.2
MATERIAL PROPERTIES

14.2.1 **Determination of Required Tests.** The Engineer of Record shall determine the specific tests that are required from Section 14.2.2 through 14.2.6 and specify the locations where they are required. Where available, the use of applicable project records shall be permitted to reduce or eliminate the need for testing.

14.2.2 **Tensile Properties.** Tensile properties of members shall be considered in evaluation by structural analysis (Section 14.3) or load tests (Section 14.4). Such properties shall include the yield stress, tensile strength, and percent elongation. Where available, certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6 or A568, as applicable, shall be permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure.

14.2.3 **Chemical Composition.** Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification (WPS). Where available, results from certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures or Saudi equivalents where applicable shall be permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties, or from samples taken from the same locations.

14.2.4 **Base Metal Notch Toughness.** Where welded tension splices in heavy shapes and plates as defined in Section 1.3.1.3 are critical to the performance of the structure, the Charpy V-notch toughness shall be determined in accordance with the provisions of Section 1.3.1.3. If the notch toughness so determined does not meet the provisions of Section 1.3.1.3, the Engineer of Record shall determine if remedial actions are required.

14.2.5 **Weld Metal.** Where structural performance is dependent on existing welded connections, representative samples of weld metal shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of Saudi equivalent to AWS D1.1 are not met, the Engineer of Record shall determine if remedial actions are required.

14.2.6 **Bolts and Rivets.** Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine tensile strength in accordance with ASTM F606 and the bolt classified accordingly. Alternatively, the assumption that the bolts are A307 shall be permitted. Rivets shall be assumed to be A502, Grade 1, unless a higher grade is established through documentation or testing.

SECTION 14.3
EVALUATION BY STRUCTURAL ANALYSIS

14.3.1 **Dimensional Data.** All dimensions used in the evaluation, such as spans, column heights, member spacings, bracing locations, cross section dimensions, thicknesses, and connection details, shall be determined from a field survey. Alternatively, when available, it shall be permitted to determine such dimensions from applicable project design or shop drawings with field verification of critical values.

14.3.2 **Strength Evaluation.** Forces (load effects) in members and connections shall be determined by structural analysis applicable to the type of structure evaluated. The load effects shall be determined for the loads and factored load combinations stipulated in Section 1.4.

The design strength of members and connections shall be determined from applicable provisions of Chapters 2 through 11 of SBC 306.

14.3.3 **Serviceability Evaluation.** Where required, the deformations at service loads shall be calculated and reported.

SECTION 14.4
EVALUATION BY LOAD TESTS

14.4.1 **Determination of Live Load Rating by Testing.** To determine the live load rating of an existing floor or roof structure by testing, test load shall be applied incrementally in accordance with the Engineer of Record's plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

The tested design strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested design strength equal to $1.2D + 1.6L$, where D is the nominal dead load and L is the nominal live load rating for the structure. The nominal live load rating of the floor structure shall not exceed that which can be calculated using applicable provisions of SBC 306. For roof structures, L_r, or R as defined in the Symbols, shall be substituted for L. More severe load combinations shall be used where required by applicable building codes.

Periodic unloading shall be considered once the service load level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test load for one hour, that the deformation of the structure does not increase by more than 10 percent above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay, representative of the most critical conditions, shall be selected.

14.4.2 **Serviceability Evaluation.** When load tests are prescribed, the structure shall be loaded incrementally to the service load level. Deformations shall be monitored for a period of one hour. The structure shall then be unloaded and the deformation recorded.

SECTION 14.5
EVALUATION REPORT

After the evaluation of an existing structure has been completed, the Engineer of Record shall prepare a report documenting the evaluation. The report shall indicate whether the evaluation was performed by structural analysis, by load testing or by a combination of structural analysis and load testing. Furthermore, when testing is performed, the report shall include the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design drawing, mill test reports, and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the design strength of the structure, including all members and connections, is adequate to withstand the load effects.

APPENDIX A:

GLOSSARY

Alignment chart for columns. A nomograph for determining the effective length factor *K* for some types of columns.

Amplification factor. A multiplier of the value of moment or deflection in the unbraced length of an axially loaded member to reflect the secondary values generated by the eccentricity of the applied axial load within the member.

Aspect ratio. In any rectangular configuration, the ratio of the lengths of the sides.

Batten plate. A plate element used to join two parallel components of a built-up column, girder, or strut rigidly connected to the parallel components and designed to transmit shear between them.

Beam. A structural member whose primary function is to carry loads transverse to its longitudinal axis.

Beam-column. A structural member whose primary function is to carry loads both transverse and parallel to its longitudinal axis.

Bent. A plane framework of beam or truss members which support loads and the columns which support these members.

Biaxial bending. Simultaneous bending of a member about two perpendicular axes.

Bifurcation. The phenomenon whereby a perfectly straight member under compression may either assume a deflected position or may remain undeflected, or a beam under flexure may either deflect and twist out of plane or remain in its in-plane deflected position.

Braced frame. A frame in which the resistance to lateral load or frame instability is primarily provided by a diagonal, a *K* brace, or other auxiliary system of bracing.

Brittle fracture. Abrupt cleavage with little or no prior ductile deformation.

Buckling load. The load at which a perfectly straight member under compression assumes a deflected position.

Built-up member. A member made of structural metal elements that are welded, bolted, or riveted together.

Charpy V-notch impact test. A standard dynamic test in which a notched specimen is struck and broken by a single blow in a specially designed testing machine. The measured test values may be the energy absorbed, the percentage shear fracture, the lateral expansion opposite the notch, or a combination thereof.

Cladding. The exterior covering of the structural components of a building.

Cold-formed members. Structural members formed from steel without the application of heat.

Column. A structural member whose primary function is to carry loads parallel to its longitudinal axis.

Column curve. A curve expressing the relationship between axial column strength and slenderness ratio.

Combined mechanism. A mechanism determined by plastic analysis procedure which combines elementary beam, panel, and joint mechanisms.

Compact section. Compact sections are capable of developing a fully plastic stress distribution and possess rotation capacity of approximately three before the onset of local buckling.

Composite beam. A steel beam structurally connected to a concrete slab so that the beam and slab respond to loads as a unit.

Concrete-encased beam. A beam totally encased in concrete cast integrally with the slab.

Connection. Combination of joints used to transmit forces between two or more members. Categorized by the type and amount of force transferred (moment, shear, end reaction). See also *Splices.*

Critical load. The load at which bifurcation occurs as determined by a theoretical stability analysis.

Curvature. Rotation per unit length due to bending.

Design documents. Documents prepared by the designer (design drawings, design details, and job specifications).

Design strength. Resistance (force, moment, stress, as appropriate) provided by element or connection; the product of the nominal strength and the resistance factor.

Diagonal bracing. Inclined structural members carrying primarily axial load enabling a structural frame to act as a truss to resist horizontal loads.

Diaphragm. Floor slab, metal wall, or roof panel possessing a large in-plane shear stiffness and strength adequate to transmit horizontal forces to resisting systems.

Diaphragm action. The in-plane action of a floor system (also roofs and walls) such that all columns framing into the floor from above and below are maintained in the same position relative to each other.

Double concentrated forces. Two equal and opposite forces which form a couple on the same side of the loaded member.

Double curvature. A bending condition in which end moments on a member cause the member to assume an S shape.

Drift. Lateral deflection of a building.

Drift index. The ratio of lateral deflection to the height of the building.

Ductility factor. The ratio of the total deformation at maximum load to the elastic-limit deformation.

Effective length. The equivalent length *KL* used in compression formulas and determined by a bifurcation analysis.

Effective length factor K. The ratio between the effective length and the unbraced length of the member measured between the centers of gravity of the bracing members.

Effective moment of inertia. The moment of inertia of the cross section of a member that remains elastic when partial plastification of the cross section takes place, usually under the combination of residual stress and applied stress. Also, the moment of inertia based on effective widths of elements that buckle locally. Also, the moment of inertia used in the design of partially composite members.

Effective stiffness. The stiffness of a member computed using the effective moment of inertia of its cross section.

Effective width. The reduced width of a plate or slab which, with an assumed uniform stress distribution, produces the same effect on the behavior of a structural member as the actual plate width with its non uniform stress distribution.

Elastic analysis. Determination of load effects (force, moment, stress, as appropriate) on members and connections based on the assumption that material deformation disappears on removal of the force that produced it.

Elastic-perfectly plastic. A material which has an idealized stress-strain curve that varies linearly from the point of zero strain and zero stress up to the yield point of the material, and then increases in strain at the value of the yield stress without any further increases in stress.

Embedment. A steel component cast in a concrete structure which is used to transmit externally applied loads to the concrete structure by means of bearing, shear, bond, friction, or any combination thereof. The embedment may be fabricated of structural-steel plates, shapes, bars, bolts, pipe, studs, concrete reinforcing bars, shear connectors, or any combination thereof.

Encased steel structure. A steel-framed structure in which all of the individual frame members are completely encased in cast-in-place concrete.

Euler formula. The mathematical relationship expressing the value of the Euler load in terms of the modulus of elasticity, the moment of inertia of the cross section, and the length of a column.

Euler load. The critical load of a perfectly straight, centrally loaded pin-ended column.

Eyebar. A particular type of pin-connected tension member of uniform thickness with forged or flame-cut head of greater width than the body proportioned to provide approximately equal strength in the head and body.

Factored load. The product of the nominal load and a load factor *Fastener.* Generic term for welds, bolts, rivets, or other connecting device *Fatigue.* A fracture phenomenon resulting from a fluctuating stress cycle.

First-order analysis. Analysis based on first-order deformations in which equilibrium conditions are formulated on the undeformed structure.

Flame-cut plate. A plate in which the longitudinal edges have been prepared by oxygen cutting from a larger plate.

Flat width. For a rectangular HSS, the nominal width minus twice the outside corner radius. In absence of knowledge of the corner radius, the flat width may be taken as the total section width minus three times the thickness.

Flexible connection. A connection permitting a portion, but not all, of the simple beam rotation of a member end.

Floor system. The system of structural components separating the stories of a building.

Force. Resultant of distribution of stress over a prescribed area. A reaction that develops in a member as a result of load (formerly called total stress or stress). Generic term signifying axial loads, bending moment, torques, and shears.

Fracture toughness. Measure of the ability to absorb energy without fracture. Generally determined by impact loading of specimens containing a notch having a prescribed geometry.

Frame buckling. A condition under which bifurcation may occur in a frame.

Frame instability. A condition under which a frame deforms with increasing lateral deflection under a system of increasing applied monotonic loads until a maximum value of the load called the stability limit is reached, after which the frame will continue to deflect without further increase in load.

Fully composite beam. A composite beam with sufficient shear connectors to develop the full flexural strength of the composite section.

High-cycle fatigue. Failure resulting from more than 20,000 applications of cyclic stress.

HSS. Hollow structural sections that are prismatic square, rectangular or round products of a pipe or tubing mill and meet the geometric tolerance, tensile strength and chemical composition requirements of a standard specification.

Hybrid beam. A fabricated steel beam composed of flanges with a greater yield strength than that of the web. Whenever the maximum flange stress is less than or equal to the web yield stress the girder is considered homogeneous.

Hysteresis loop. A plot of force versus displacement of a structure or member subjected to reversed, repeated load into the inelastic range, in which the path followed during release and removal of load is different from the path for the addition of load over the same range of displacement.

Inclusions. Nonmetallic material entrapped in otherwise sound metal.

Incomplete fusion. Lack of union by melting of filler and base metal over entire prescribed area.

Inelastic action. Material deformation that does not disappear on removal of the force that produced it.

Instability. A condition reached in the loading of an element or structure in which continued deformation results in a decrease of load-resisting capacity.

Joint. Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and method of force transfer.

K bracing. A system of struts used in a braced frame in which the pattern of the struts resembles the letter K, either normal or on its side.

Lamellar tearing. Separation in highly restrained base metal caused by through-thickness strains induced by shrinkage of adjacent filler metal.

Lateral bracing member. A member utilized individually or as a component of a lateral bracing system to prevent buckling of members or elements and/or to resist lateral loads.

Lateral (or lateral-torsional) buckling. Buckling of a member involving lateral deflection and twist.

Leaning column. Gravity-loaded column where connections to the frame (simple connections) do not provide resistance to lateral loads.

Limit state. A condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (*serviceability limit state)* or to be unsafe (*strength limit state).*

Limit states. Limits of structural usefulness, such as brittle fracture, plastic collapse, excessive deformation, durability, fatigue, instability, and serviceability.

Load factor. A factor that accounts for unavoidable deviations of the actual load from the nominal value and for uncertainties in the analysis that transforms the load into a load effect.

Loads. Forces or other actions that arise on structural systems from the weight of all permanent construction, occupants and their possessions, environmental effects, differential settlement, and restrained dimensional changes. *Permanent* loads are those loads in which variations in time are rare or of small magnitude. All other loads are *variable* loads. See *Nominal loads.*

LRFD (Load and Resistance Factor Design). A method of proportioning structural components (members, connectors, connecting elements, and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations.

Local buckling. The buckling of a compression element which may precipitate the failure of the whole member.

Low-cycle fatigue. Fracture resulting from a relatively high-stress range resulting in a relatively small number of cycles to failure.

Lower bound load. A load computed on the basis of an assumed equilibrium moment diagram in which the moments are not greater than M_p that is less than or at best equal to the true ultimate load.

Mechanism. An articulated system able to deform without an increase in load, used in the special sense that the linkage may include real hinges or plastic hinges, or both.

Mechanism method. A method of plastic analysis in which equilibrium between external forces and internal plastic hinges is calculated on the basis of an assumed mechanism. The failure load so determined is an upper bound.

Nodal Brace. A brace that prevents the lateral movement or twist at the particular brace location along the length of the beam or column without any direct attachment to other braces at adjacent brace points. (See relative brace).

Nominal loads. The magnitudes of the loads specified by the applicable code.

Nominal strength. The capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Noncompact section. Noncompact sections can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at strain levels required for a fully plastic stress distribution.

P-Delta effect. Secondary effect of column axial loads and lateral deflection on the moments in members.

Panel zone. The zone in a beam-to-column connection that transmits moment by a shear panel.

Partially composite beam. A composite beam for which the shear strength of shear connectors governs the flexural strength.

Plane frame. A structural system assumed for the purpose of analysis and design to be two-dimensional.

Plastic analysis. Determination of load effects (force, moment, stress, as appropriate) on members and connections based on the assumption of rigid-plastic behavior, i.e., that equilibrium is satisfied throughout the structure and yield is not exceeded anywhere. Second order effects may need to be considered.

Plastic design section. The cross section of a member which can maintain a full plastic moment through large rotations so that a mechanism can develop; the section suitable for design by plastic analysis.

Plastic hinge. A yielded zone which forms in a structural member when the plastic moment is attained. The beam is assumed to rotate as if hinged, except that it is restrained by the plastic moment M_p.

Plastic-limit load. The maximum load that is attained when a sufficient number of yield zones have formed to permit the structure to deform plastically without further increase in load. It is the largest load a structure will support, when perfect plasticity is assumed and when such factors as instability, second-order effects, strain hardening, and fracture are neglected.

Plastic mechanism. See Mechanism.

Plastic modulus. The section modulus of resistance to bending of a completely yielded cross section. It is the combined static moment about the plastic neutral axis of the cross-sectional areas above and below that axis.

Plastic moment. The resisting moment of a fully-yielded cross section.

Plastic strain. The difference between total strain and elastic strain *Plastic zone.* The yielded region of a member.

Plastification. The process of successive yielding of fibers in the cross section of a member as bending moment is increased.

Plate girder. A built-up structural beam.

Post-buckling strength. The load that can be carried by an element, member, or frame after buckling.

Prying Action. Lever action that exists in connections in which the line of application of the applied load is eccentric to the axis of the bolt, causing deformation of the fitting and an amplification of the axial force in the bolt.

Redistribution of moment. A process which results in the successive formation of plastic hinges so that less highly stressed portions of a structure may carry increased moments.

Relative Brace. A brace that controls the relative movement of two adjacent brace points along the length of a beam or column or the relative lateral displacement of two stories in a frame. (See nodal brace).

Required strength. Load effect (force, moment, stress, as appropriate) acting on element or connection determined either by structural analysis from the factored loads (using appropriate critical load combinations) or explicitly specified.

Residual stress. The stresses that remain in an unloaded member after it has been formed into a finished product. (Examples of such stresses include, but are not limited to, those induced by cold bending, cooling after rolling, or welding).

Resistance. The capacity of a structure or component to resist the effects of loads. It is determined by computations using specified material strengths, dimensions and formulas derived from accepted principles of structural mechanics, or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions. Resistance is a generic term that includes both strength and serviceability limit states.

Resistance factor. A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure.

Rigid frame. A structure in which connections maintain the angular relationship between beam and column members under load.

Root of the flange. Location on the web of the corner radius termination point to the toe of the flange-to-web weld. Measured as the k distance from the far side of the flange.

Rotation capacity. The incremental angular rotation that a given shape can accept prior to local failure defined as R = (O_u / O_p) where O_u is the overall rotation attained at the factored load state and Op is the idealized rotation corresponding to elastic theory applied to the case of $M = M_p$.

St. Venant torsion. That portion of the torsion in a member that induces only shear stresses in the member.

Second-order analysis. Analysis based on second-order deformations, in which equilibrium conditions are formulated on the deformed structure.

Service load. Load expected to be supported by the structure under normal usage; often taken as the nominal load.

Serviceability limit state. Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, or the comfort of its occupants or function of machinery under normal usage.

Shape factor. The ratio of the plastic moment to the yield moment, or the ratio of the plastic modulus to the section modulus for a cross section.

Shear friction. Friction between the embedment and the concrete that transmits shear loads. The relative displacement in the plane of the shear load is considered to be resisted by shear-friction anchors located perpendicular to the plane of the shear load.

Shear lugs. Plates, welded studs, bolts, and other steel shapes that are embedded in the concrete and located transverse to the direction of the shear force and that transmit shear loads, introduced into the concrete by local bearing at the shear lug-concrete interface.

Shear wall. A wall that resists, in its own plane, shear forces resulting from applied wind, earthquake, or other transverse loads or provides frame stability. Also called a structural wall.

Sidesway. The lateral movement of a structure under the action of lateral loads, unsymmetrical vertical loads, or unsymmetrical properties of the structure.

Sidesway buckling. The buckling mode of a multistory frame precipitated by the relative lateral displacements of joints, leading to failure by sidesway of the frame.

Simple plastic theory. See Plastic design.

Single curvature. A deformed shape of a member having one smooth continuous arc, as opposed to double curvature which contains a reversal.

Slender-element section. The cross section of a member which will experience local buckling in the elastic range.

Slenderness ratio. The ratio of the effective length of a column to the radius of gyration of the column, both with respect to the same axis of bending.

Slip-critical joint. A bolted joint in which the slip resistance of the connection is required.

Spaceframe. A three-dimensional structural framework (as contrasted to a plane frame).

Splice. The connection between two structural elements joined at their ends to form a single, longer element.

Stability-limit load. Maximum (theoretical) load a structure can support when second-order instability effects are included.

Stepped column. A column with changes from one cross section to another occurring at abrupt points within the length of the column.

Stiffener. A member, usually an angle or plate, attached to a plate or web of a beam or girder to distribute load, to transfer shear, or to prevent buckling of the member to which it is attached.

Stiffness. The resistance to deformation of a member or structure measured by the ratio of the applied force to the corresponding displacement.

Story drift. The difference in horizontal deflection at the top and bottom of a story.

Strain hardening. Phenomenon wherein ductile steel, after undergoing considerable deformation at or just above yield point, exhibits the capacity to resist substantially higher loading than that which caused initial yielding.

Strain-hardening strain. For structural steels that have a flat (plastic) region in the stress-strain relationship, the value of the strain at the onset of strain hardening.

Strength design. A method of proportioning structural members using load factors and resistance factors such that no applicable limit state is exceeded (also called load and resistance factor design).

Strength limit state. Limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached.

Stress. Force per unit area.

Stress concentration. Localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) due to abrupt changes in geometry or localized loading.

Strong axis. The major principal axis of a cross section.

Structural system. An assemblage of load-carrying components which are joined together to provide regular interaction or interdependence.

Stub column. A short compression-test specimen, long enough for use in measuring the stress-strain relationship for the complete cross section, but short enough to avoid buckling as a column in the elastic and plastic ranges.

Sub assemblage. A truncated portion of a structural frame.

Supported frame. A frame which depends upon adjacent braced or unbraced frames for resistance to lateral load or frame instability. (This transfer of load is frequently provided by the floor or roof system through diaphragm action or by horizontal cross bracing in the roof).

Tangent modulus. At any given stress level, the slope of the stress-strain curve of a material in the inelastic range as determined by the compression test of a small specimen under controlled conditions.

Temporary structure. A general term for anything that is built or constructed (usually to carry construction loads) that will eventually be removed before or after completion of construction and does not become part of the permanent structural system.

Tensile strength. The maximum tensile stress that a material is capable of sustaining.

Tension field action. The behavior of a plate girder panel under shear force in which diagonal tensile stresses develop in the web and compressive forces develop in the transverse stiffeners in a manner analogous to a Pratt truss.

Toe of the fillet. Termination point of fillet weld or of rolled section fillet.

Torque-tension relationship. Term applied to the wrench torque required to produce specified pre-tension in high-strength bolts.

Turn-of-nut method. Procedure whereby the specified pre-tension in high-strength bolts is controlled by rotation of the wrench a predetermined amount after the nut has been tightened to a snug fit.

Unbraced frame. A frame in which the resistance to lateral load is provided by the bending resistance of frame members and their connections.

Unbraced length. The distance between braced points of a member, measured between the centers of gravity of the bracing members.

Undercut. A notch resulting from the melting and removal of base metal at the edge of a weld.

Universal-mill plate. A plate in which the longitudinal edges have been formed by a rolling process during manufacture.

Often abbreviated as UM plate.

Upper bound load. A load computed on the basis of an assumed mechanism which will always be at best equal to or greater than the true ultimate load.

Vertical bracing system. A system of shear walls, braced frames, or both, extending through one or more floors of a building.

Von Mises yield criterion. A theory which states that inelastic action at any point in a body under any combination of stresses begins only when the strain energy of distortion per unit volume absorbed at the point is equal to the strain energy of distortion absorbed per unit volume at any point in a simple tensile bar stressed to the elastic limit under a state of uniaxial stress. It is often called the maximum strain-energy-of-distortion theory. Accordingly, shear yield occurs at 0.58 times the yield strength.

Warping torsion. That portion of the total resistance to torsion that is provided by resistance to warping of the cross section.

Weak axis. The minor principal axis of a cross section.

Weathering steel. A type of high-strength, low-alloy steel which can be used in normal environments (not marine) and outdoor exposures without protective paint covering. This steel develops tight adherent rust at a decreasing rate with respect to time.

Web buckling. The buckling of a web plate.

Web crippling. The local failure of a web plate in the immediate vicinity of a concentrated load or reaction.

Working load. Also called service load. The actual load assumed to be acting on the structure.

Yield moment. In a member subjected to bending, the moment at which an outer fiber first attains the yield stress.

Yield plateau. The portion of the stress-strain curve for uniaxial tension or compression in which the stress remains essentially constant during a period of substantially increased strain.

Yield point. The first stress in a material at which an increase in strain occurs without an increase in stress.

Yield strength. The stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain. Deviation expressed in terms of strain.

Yield stress. Yield point, yield strength, or yield stress level as defined.

Yield-stress level. The average stress during yielding in the plastic range, the stress determined in a tension test when the strain reaches 0.005 mm per mm.

APPENDIX B:

SYMBOLS

The section number in the right hand column refers to the section where the symbol is first used.

Symbol	**Definition**	**Section**
A	Area of directly connected elements	2.3
A_B	Loaded area of concrete, mm^2	9.2.4
A_b	Nominal unthreaded body area of bolt or threaded part, mm^2	10.3.6
A_c	Area of concrete, mm^2	9.2.2
A_c	Area of concrete slab within effective width, mm^2	9.5.2
A_D	Area of an upset rod based on the major thread diameter, mm^2	10.3.6
A_e	Effective area, mm^2	2.3
A_f	Area of the compression flange, mm^2	6.3.4
A_{fe}	Effective tension flange area, mm^2	2.10
A_{fg}	Gross area of flange, mm^2	2.10
A_{fn}	Net area of flange, mm^2	2.10
A_g	Gross area, mm^2	1.5
A_{gt}	Gross area subject to tension, mm^2	10.4.3
A_{gv}	Gross area subject to shear, mm^2	10.4.3
A_n	Net area, mm^2	2.3
A_{nt}	Net area subject to tension, mm^2	10.4.2
A_{nv}	Net area subject to shear, mm^2	10.4.1
A_{pb}	Projected bearing area, mm^2	10.8
A_r	Area of reinforcing bars, mm^2	9.2.2
A_s	Area of steel cross section, mm^2	9.2.2
A_{sc}	Cross-sectional area of stud shear connector, mm^2	9.5.3
A_{sf}	Shear area on the failure path, mm^2	4.3
A_{st}	Area of a transverse stiffener, mm^2	7.4
A_t	Net tensile area, mm^2	11.3.5
A_w	Web area, mm^2	6.2.1
A_1	Area of steel concentrically bearing on a concrete support, m^2	10.9
A_2	Total cross-sectional area of a concrete support, mm^2	10.9
B	Factor for bending stress in tees and double angles	6.1.2
B	Factor for bending stress in web-tapered members, mm, defined by Equations 6.3-8 through 6.3-11	6.3
B_1, B_2	Factors used in determining M_u for combined bending and axial forces when first-order analysis is employed	3.1
C_{PG}	Plate-girder coefficient	7.2
C_b	Bending coefficient dependent on moment gradient	6.1.2
C_f	Constant based on stress category, given in Table 11.3.1	11.3.3
C_m	Coefficient applied to bending term in interaction formula for prismatic members and dependent on column curvature caused by applied moments	3.1
C'_m	Coefficient applied to bending term in interaction formula for tapered members and dependent on axial stress at the small end of the member	6.3.6

C_p	Ponding flexibility coefficient for primary member in a flat roof	11.2
C_s	Ponding flexibility coefficient for secondary member in a flat roof	11.2
C_v	Ratio of "critical" web stress, according to linear buckling theory, to the shear yield stress of web material	7.3
C_w	Warping constant, mm^6	6.1.2
D	Outside diameter of circular hollow section, mm	2.5.3
D	Factor used in Equation 7.4-1, dependent on the type of transverse stiffeners used in a plate girder	7.4
E	Modulus of elasticity of steel, E = 200,000 MPa	5.2
E_c	Modulus of elasticity of concrete, MPa	9.2.2
E_m	Modified modulus of elasticity, MPa	9.2.2
F_{BM}	Nominal strength of the base material to be welded, MPa	10.2.4
F_{EXX}	Classification number of weld metal (minimum specified strength), MPa	10.2.4
F_L	Smaller of $(F_{yf}-F_r)$ or F_{yw}, MPa	6.1.2
F_{SR}	Design stress range, MPa	11.3.3
F_{TH}	Threshold fatigue stress range, maximum stress range for indefinite design life, MPa	11.3.3
$F_{b\gamma}$	Flexural stress for tapered members defined by Equations 6.3-4 and 6.3-5	6.3.4
F_{cr}	Critical stress, MPa	5.2
F_{crft}, F_{cry}, F_{crz}	Flexural-torsional buckling stresses for double-angle and tee-shaped compression members, MPa	5.3
F_e	Elastic buckling stress, MPa	5.3
F_{ex}	Elastic flexural buckling stress about the major axis, MPa	5.3
F_{ey}	Elastic flexural buckling stress about the minor axis, MPa	5.3
F_{ez}	Elastic torsional buckling stress, MPa	5.3
F_{my}	Modified yield stress for composite columns, MPa	9.2.2
R_n	Nominal shear rupture strength, MPa	10.4
F_r	Compressive residual stress in flange 69 MPa for rolled shapes; 114 MPa for welded built-up shapes	2.5.1
$F_{s\gamma}$	Stress for tapered members defined by Equation 6.3-6, MPa	6.3.4
F_u	Specified minimum tensile strength of the type of steel being used, MPa	2.10
F_w	Nominal strength of the weld electrode material, MPa	10.2.4
$F_{w\gamma}$	Stress for tapered members defined by Equation 6.3-7, MPa	6.3.4
F_y	Specified minimum yield stress of the type of steel being used, MPa. As used in this Specification, "yield stress" denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point)	1.5
F_{yf}	Specified minimum yield stress of the flange, MPa	2.5.1
F_{yr}	Specified minimum yield stress of reinforcing bars, MPa	9.2.2
F_{yst}	Specified minimum yield stress of the stiffener material, MPa	7.4
F_{yw}	Specified minimum yield stress of the web, MPa	2.5.1
G	Shear modulus of elasticity of steel, G = 77,200 MPa	6.1.2
H	Horizontal force, N	3.1
H	Flexural constant	5.3
H_s	Length of stud connector after welding, mm	9.3.5

I	Moment of inertia, mm^4	6.1.2
I_d	Moment of inertia of the steel deck supported on secondary members, mm^4	11.2
I_p	Moment of inertia of primary members, mm^4	11.2
I_s	Moment of inertia of secondary members, mm^4	11.2
J	Torsional constant for a section, mm^4	6.1.2
K	Effective length factor for prismatic member	2.7
K_z	Effective length factor for torsional buckling	5.3
K_γ	Effective length factor for a tapered member	6.3.3
L	Story height or panel spacing, mm	3.1
L	Live load due to occupancy and moveable equipment	14.4
L_b	Laterally unbraced length; length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm	6.1.2
L_c	Length of channel shear connector, mm	9.5.4
L_c	Edge distance, mm.	10.3.10
L_p	Limiting laterally unbraced length for full plastic bending capacity, uniform moment case (C_b = 1.0), mm	6.1.2
L_p	Column spacing in direction of girder, m	11.2
L_{pd}	Limiting laterally unbraced length for plastic analysis, m	6.1.2
L_q	Maximum unbraced length for the required column force with K equal to one, mm	3.3
L_r	Limiting laterally unbraced length for inelastic lateral-torsional buckling, mm	6.1.2
L_r	Roof live load	14.4
L_s	Column spacing perpendicular to direction of girder, m	11.2
M_A	Absolute value of moment at quarter point of the unbraced beam segment, N-mm	6.1.2
M_B	Absolute value of moment at centerline of the unbraced beam segment, N-mm	6.1.2
M_C	Absolute value of moment at three-quarter point of the unbraced beam segment, N-mm	6.1.2
M_{cr}	Elastic buckling moment, N-mm	6.1.2
M_{lt}	Required flexural strength in member due to lateral frame translation only, N-mm.	3.1
M_{max}	Absolute value of maximum moment in the unbraced beam segment, N-mm	6.1.2
M_n	Nominal flexural strength, N-mm	6.1.1
M'_{nx}, M'_{ny}	Flexural strength defined in Equations 8.3-12 and 8.3-13 for use in alternate interaction equations for combined bending and axial force, N-mm	8.3
M_{nt}	Required flexural strength in member assuming there is no lateral translation of the frame, N-mm	3.1.2
M_p	Plastic bending moment, N-mm	6.1.1
M_p	Moment defined in Equations 8.3-11(a) and 8.3-11(b), for use in alternate interaction equations for combined bending and axial force, N-mm	8.3
M_r	Limiting buckling moment, M_{cr}, when $\lambda = \lambda_r$ and C_b = 1.0, N-mm	6.1.2
M_u	Required flexural strength, kip-in. N-mm	3.1

M_y	Moment corresponding to onset of yielding at the extreme fiber from an elastic stress distribution (= F_y S for homogeneous sections), N-mm	6.1.1
M_1	Smaller moment at end of unbraced length of beam or beam-column, N-mm	6.1.3
M_2	Larger moment at end of unbraced length of beam or beam-column, N-mm	6.1.3
N	Length of bearing, mm	11.1.3
N	Number of stress range fluctuations in design life	11.3.3
N_r	Number of stud connectors in one rib at a beam intersection	9.3.5
P_{br}	Required story or panel bracing shear force	3.3
P_{e1}, P_{e2}	Elastic Euler buckling load for braced and unbraced frame, respectively, N	3.1
P_n	Nominal axial strength (tension or compression), N	4.1
P_p	Bearing load on concrete, N	10.9
P_u	Required axial strength (tension or compression),	2.5.1
P_y	Yield strength,	2.5.1
Q	Full reduction factor for slender compression elements	5.3
Q_a	Reduction factor for slender stiffened compression elements	2.5
Q_n	Nominal strength of one stud shear connector,	9.5
Q_s	Reduction factor for slender unstiffened compression elements	2.5.3
R	Nominal load due to initial rainwater or ice exclusive of the ponding contribution	14.4
R_{PG}	Plate girder bending strength reduction factor	7.2
R_e	Hybrid girder factor	7.2
R_n	Nominal strength	1.5.3
R_v	Web shear strength, N	11.1.7
S	Elastic section modulus, mm^3	6.1.2
S	Spacing of secondary members, m	11.2
S'_x	Elastic section modulus of larger end of tapered member about its major axis, mm^3	6.3
T	Tension force due to service loads, N	10.3.9
T_b	Specified pretension load in high-strength bolt,	10.3.9
T_u	Required tensile strength due to factored loads,	10.3.9
U	Reduction coefficient, used in calculating effective net area	2.3
V_n	Nominal shear strength, N	6.2.2
V_u	Required shear strength,	7.4
W	Wind load	C1.4
X_1	Beam buckling factor defined by Equation 6.1-8	6.1.2
X_2	Beam buckling factor defined by Equation 6.1-9	6.1.2
Z	Plastic section modulus, mm^3	6.1.1
a	Clear distance between transverse stiffeners, mm	6.2.2
a	Distance between connectors in a built-up member, mm	5.4
a	Shortest distance from edge of pin hole to edge of member measured parallel to direction of force, mm	4.3
a_r	Ratio of web area to compression flange area	7.2
a'	Weld length, mm	2.10
b	Compression element width, mm	2.5.1
b_e	Reduced effective width for slender compression elements, mm	2.5.3
b_{eff}	Effective edge distance, mm	4.3
b_f	Flange width, mm	2.5.1
b_s	Stiffener width for one-sided stiffeners, mm	3.3.4

c_1, c_2, c_3	Numerical coefficients	9.2.2
d	Nominal fastener diameter, mm	10.3.3
d	Overall depth of member, mm	2.5.1
d	Pin diameter, mm	4.3
d	Roller diameter, mm	10.8
d_L	Depth at larger end of unbraced tapered segment, mm	6.3
d_b	Beam depth, mm.	11.1.7
d_b	Nominal diameter (body or shank diameter), mm	11.3.3
d_c	Column depth, mm.	11.1.7
d_o	Depth at smaller end of unbraced tapered segment, mm	6.3
e	Base of natural logarithm = 2.71828.	C5.2
f	Computed compressive stress in the stiffened element, MPa	2.5.3
f_{b1}	Smallest computed bending stress at one end of a tapered segment, MPa	6.3
f_{b2}	Largest computed bending stress at one end of a tapered segment, MPa	6.3
$f_c^{'}$	Specified compressive strength of concrete, MPa.	10.2.2
f_o	Stress due to 1.2D + 1.2R, MPa	11.3
f_{un}	Required normal stress, MPa	8.2
f_{uv}	Required shear stress, MPa	8.2
f_v	Required shear stress due to factored loads in bolts or rivets, MPa	10.3.7
g	Transverse center-to-center spacing (gage) between fastener gage lines, mm	2.2
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, mm	2.5.1
h	Distance between centroids of individual components perpendicular to the member axis of buckling, mm.	5.4.1
h_c	Twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, mm	2.5.1
h_o	Distance between flange centroids, mm	3.3.4.1
h_r	Nominal rib height, mm	9.3.5.2
h_s	Factor used in Equation 6.3-6 for web-tapered members	6.3.4
h_w	Factor used in Equation 6.3-7 for web-tapered members	6.3.4
j	Factor defined by Equation 6.2-4 for minimum moment of inertia for a transverse stiffener	6.2.3
k	Distance from outer face of flange to web toe of fillet, mm	11.1.3
k_v	Web plate buckling coefficient	6.2.2
l	Laterally unbraced length of member at the point of load, mm	2.7
l	Length of bearing, mm	10.8
l	Length of connection in the direction of loading, mm	2.3
l	Length of weld, mm	2.3
l	Length of connection in the direction of loading, mm.	2.3
m	Ratio of web to flange yield stress or critical stress in hybrid beams	7.2
n	Number of nodal braced points within the span	3.3.4.2
r	Governing radius of gyration, mm	2.7

r_{To}	For the smaller end of a tapered member, the radius of gyration, considering only the compression flange plus one-third of the compression web area, taken about an axis in the plane of the web, mm	6.3.4
r_i	Minimum radius of gyration of individual component in a built-up member, mm	5.4.1
r_{ib}	Radius of gyration of individual component relative to centroidal axis parallel to member axis of buckling, mm.	5.4.1
r_m	Radius of gyration of the steel shape, pipe, or tubing in composite columns. For steel shapes it may not be less than 0.3 times the overall thickness of the composite section, mm	9.2.2
r_o	Polar radius of gyration about the shear center, mm	5.3.1
r_{ox}, r_{oy}	Radius of gyration about x and y axes at the smaller end of a tapered member, respectively, mm.	6.3.3
r_x, r_y	Radius of gyration about x and y axes, respectively, mm	5.3.2
r_{yc}	Radius of gyration about y axis referred to compression flange, or if reverse curvature bending, referred to smaller flange, mm	6.1
s	Longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm	2.2
t	Thickness of element, mm.	2.5.1
t	HSS design wall thickness, mm	2.5.1
t_f	Flange thickness of channel shear connector, mm	9.5.4
t_s	Web stiffener thickness, mm	3.3.4.2
t_w	Web thickness of channel shear connector, mm	9.5.4
t_w	Web thickness, mm	2.5.3.1
w	Leg size of the fillet weld, mm	10.2.2
w	Plate width; distance between welds, mm	2.3
w	Unit weight of concrete, kg/m^3	9.2.2
w_r	Average width of concrete rib or haunch, mm	9.3.5.2
x_o, y_o	Coordinates of the shear center with respect to the centroid, m.	5.3
x	Connection eccentricity, mm.	2.3
z	Distance from the smaller end of tapered member used in Equation 6.3-1 for the variation in depth, mm.	6.3
α	Separation ratio for built-up compression members = h / 2$r_{ib.}$	5.4.1
β	Reduction factor given by Equation 10.2-1	10.2.1
β_T	Brace stiffness requirement when there is no web Distortion	3.3.4.2
β_{Tb}	Required nodal torsional bracing stiffness	3.3.4.2
β_{br}	Required story or panel shear stiffness	3.3
$\beta_{\sec}$	Web distortional stiffness, including the effect of web transverse stiffeners, if any	3.3.4.2
Δ_{oh}	Translation deflection of the story under consideration, m	3.1.2
γ	Depth tapering ratio	6.3
γ	Subscript for tapered members	6.3
ζ	Exponent for alternate beam-column interaction equation	8.3
η	Exponent for alternate beam-column interaction equation	8.3
λ_c	Column slenderness parameter	5.2
λ_e	Equivalent slenderness parameter	5.3.2
λ_{eff}	Effective slenderness ratio defined by Equation 6.3-2	6.3
λ_p	Limiting slenderness parameter for compact element	2.5.1

λ_r	Limiting slenderness parameter for noncompact element	2.5.1
ϕ	Resistance factor	1.5.3
ϕ_b	Resistance factor for flexure	6.1.1
ϕ_c	Resistance factor for compression	1.5.1
ϕ_c	Resistance factor for axially loaded composite columns	9.2.2
ϕ_t	Resistance factor for tension	4.1
ϕ_y	Resistance factor for shear	6.2.2

Gratitude

In appreciation and gratitude

to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence

& Aviation and Inspector General

For their continuous support and gracious consideration,

the Saudi Building Code National Committee (SBCNC)

is honored to present the first issue of

the Saudi Building Code (SBC).

PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11th June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Requirements for Concrete Structures (SBC 304) were developed based on ICC code in addition to American Concrete Institute (ACI) materials. ACI grants permission to the SBCNC to include ACI materials in the SBC, and ACI is not responsible for any modifications or changes that SBCNC has made to accommodate local conditions.

The development process of SBC 304 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made on ACI, such as Durability Requirements, the simplified methods for the design of two-way slab system of Appendix C, expanding some topics such as Hot Weather, taking into considerations the properties of local material such as the Saudi steel and the engineering level for those involved in the building sector.

As a follow-up to the *Saudi Building Code*, SBCNC offers a companion document, the *Saudi Building Code Steel Structural Requirements Commentary* (SBC 306C). The basic appeal of the Commentary is thus: it provides in a small package thorough coverage of many issues likely to be dealt with when using the *Saudi Building Code Steel Structural Requirements* (SBC 306) and then supplements that coverage with technical background. Reference lists, information sources and bibliographies are also included.

Strenuous effort has been made to keep the vast quantity of material accessible and its method of presentation useful. With a comprehensive yet concise summary of each section, the Commentary provides a convenient reference for regulations applicable to the construction of buildings and structures. In the chapters that follow, discussions focus on the full meaning and implications of the *Steel Structural Requirements* (SBC 306) text. Guidelines suggest the most effective method of application, and the consequences of not adhering to the SBC 306 text. Illustrations are provided to aid understanding; they do not necessarily illustrate the only methods of achieving *code* compliance.

The format of the Commentary includes the section, table and figure which is applicable to the same section in the SBC 306C. The numbers of the section, table and figure in the commentary begin with the letter R. The Commentary reflects the most up-to-date text of the 2007 *Saudi Building Code steel structural requirements* (SBC 306C). American Concrete Institute (ACI) grants permission to the SBCNC to include all or portions of ACI codes and standards in the SBC, and ACI is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Readers should note that the Commentary (SBC 306C) is to be used in conjunction with the *Saudi Building Code steel structural requirements* (SBC 306) and not as a substitute for the code. **The Commentary is advisory only**; the code official alone possesses the authority and responsibility for interpreting the code.

Comments and recommendations are encouraged, for through your input, it can improve future editions.

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SBC 306C was reviewed and significantly revised by special technical committee during the period Aug.-Sept. 2007. The committee members are:

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CHAPTER 1
GENERAL PROVISIONS

SECTION C1.1
SCOPE

Load and Resistance Factor Design (LRFD) is an improved approach to the design of structural steel for buildings. It involves explicit consideration of limit states, multiple load factors, and resistance factors, and implicit probabilistic determination of reliability. The designation LRFD reflects the concept of factoring both loads and resistance. This type of factoring differs from the allowable stress design (ASD) Code requirements, where only the resistance is divided by a factor of safety (to obtain allowable stress) and from the plastic design portion of that Code requirements, where only the loads are multiplied by a common load factor. The LRFD method was devised to offer the designer greater flexibility, more rationality, and possible overall economy.

SECTION C1.2
TYPES OF CONSTRUCTION

The SBC 306 emphasizes the combined importance of stiffness, strength and ductility in connection design.

An important aspect of the nominal strength of a connection, M_n, is its relationship to the strength of the connected beam $M_{p,beam}$. A connection is full strength if

$M_n > M_{p,beam}$ otherwise the connection is partial strength.

A partial strength PR connection must be designed with sufficient ductility to permit the connection components to deform and to avoid any brittle failure modes. It is also useful to define a lower limit for the strength, below which the connection can be treated as simple. Connections that transmit less than $0.2M_{p,beam}$ at a rotation of 0.02 radians can be considered to have no flexural strength for design. It should be recognized, however, that the aggregate strength of many weak partial strength connections (e.g. those with a capacity less than $0.2M_{p,beam}$) can be significant when compared to that of a few strong connections.

Connection ductility. Connection ductility is a key parameter when the deformations are concentrated in the connection elements, as is the typical case in partial strength PR connections. The ductility required will depend on the flexibility of the connections and the particular application. For example, the ductility requirement for a braced frame in a non-seismic area will generally be less than for an unbraced frame in a high seismic area.

The available ductility, θ_u, should be compared with the required rotational ductility under the full factored loads, as determined by an analysis that takes into account the nonlinear behavior of the connection. In the absence of accurate analyses of the required rotation capacity, the connection ductility may be considered adequate when the available ductility is greater than 0.03 radians. This rotation is equal to the minimum beam-to-column connection ductility as specified in the seismic provisions for special moment frames (AISC, 1997 and 1999). Many types of partial strength PR connections, such as top and seat-angle details, meet this criterion.

Connection Stiffness. Because many PR connections manifest nonlinear behavior even at low force levels, the initial stiffness of the connection, K_i, does not characterize the connection response adequately.

SECTION C1.3
MATERIALS

C1.3.1.1 ASTM Designations. The grades of structural steel approved for use under the LRFD Code requirements, covered by ASTM standard code requirements, extend to a yield stress of 690 MPa. Some of these ASTM standards specify a minimum yield point, while others specify a minimum yield strength. The term “yield stress” is used in the Code requirements as a generic term to denote either the yield point or the yield strength.

Properties in the direction of rolling are of principal interest in the design of steel structures. Hence, yield stress as determined by the standard tensile test is the principal mechanical property recognized in the selection of the steels approved for use under the Code requirements. It must be recognized that other mechanical and physical properties of rolled steel, such as anisotropy, ductility, notch toughness, formability, corrosion resistance, etc., may also be important to the satisfactory performance of a structure.

C1.3.1.3 Heavy Shapes

The web-to-flange intersection and the web center of heavy hot-rolled shapes as well as the interior portions of heavy plates may contain a coarser grain structure and/or lower toughness material than other areas of these products. This is probably caused by ingot segregation, as well as somewhat less deformation during hot rolling, higher finishing temperature, and a slower cooling rate after rolling for these heavy sections. This characteristic is not detrimental to suitability for service for compression members, or for non-welded members.

However, when heavy cross sections are joined by splices or connections using complete-joint-penetration welds which extend through the coarser and/or lower notch-tough interior portions, tensile strains induced by weld shrinkage may result in cracking, for example in a complete-joint-penetration welded connection of a heavy cross section beam to any column section. When members of lesser thickness are joined by complete-joint-penetration welds, which induce smaller weld shrinkage strains, to the finer grained and/or more notch-tough surface material of ASTM A6/A6M Group 4 and 5 shapes and heavy built-up cross sections, the potential for cracking is significantly lower, for example in a complete-joint-penetration groove welded connection of a non-heavy cross-section beam to a heavy cross-section column. For critical applications such as primary tension members, material should be specified to provide adequate toughness at service temperatures. Because of differences in the strain rate between the Charpy V-notch (CVN) impact test and the strain rate experienced in actual structures, the CVN test is conducted at a temperature higher than the anticipated service temperature for the structure. The location of the CVN test is shown in Figure C1.3-1.

The toughness requirements of Section 1.3.1.3 are intended only to provide material of reasonable toughness for ordinary service applications. For unusual applications and/or low temperature service, more restrictive requirements and/or toughness requirements for other section sizes and thicknesses may be appropriate. To minimize the potential for fracture, the notch toughness requirements of Section 1.3.1.3 must be used in conjunction with good design and fabrication procedures. Specific requirements are given in Sections 10.1.5, 10.1.6, 10.2.6, 10.2.8, and 13.2.2.

C1.3.3 **Bolts, Washers, and Nuts**. The ASTM standard for A307 bolts covers two grades of fasteners. Either grade may be used under the LRFD Code requirements; however, it should be noted that Gr. B is intended for pipe flange bolting and Gr. A is the grade long in use for structural applications.

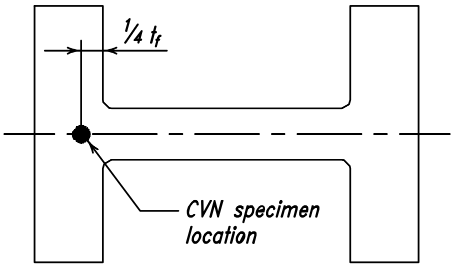


Figure. C1.3-1 Location from which Charpy impact specimen shall be taken.

C1.3.4 **Anchor Rods and Threaded Rods.** Since there is a limit on the maximum available length of A325 or A325M and A490 or A490M bolts, the attempted use of these bolts for anchor rods with design lengths longer than the maximum available lengths has presented problems in the past. The inclusion of A449 and A354 materials in these Code requirements allows the use of higher strength material for bolts longer than A325 or A325M and A490 or A490M bolts. The designer should be aware that pretensioning of anchor rods is not recommended due to relaxation and the potential for stress corrosion after pretensioning.

The designer should specify the appropriate thread and SAE fit for threaded rods used as load-carrying members.

C1.3.5 **Filler Metal and Flux for Welding**. The filler metal code requirements issued by the American Welding Society (AWS) are general code requirements which include filler metals suitable for building construction, as well as consumables that would not be suitable for building construction. For example, some electrodes covered by the code requirements are specifically limited to single pass applications, while others are restricted to sheet metal applications. Many of the filler metals listed are "low hydrogen," that is, they deposit filler metal with low levels of diffusible hydrogen. Other materials are not. Filler metals listed under the various AWS A5 code requirements may or may not have required impact toughness, depending on the specific electrode classification. Section 10.2.6 has identified certain welded joints where notch toughness of filler metal is needed in building construction. However, on structures subject to dynamic loading, filler metals may be required to deliver notch-tough weld deposits in other joints. Filler metals may be classified in either the as-welded or post weld heat-treated (stress-relieved) condition. Since most structural applications will not involve stress relief, it is important to utilize filler materials that are classified in conditions similar to those experienced by the actual structure.

When specifying filler metal and/or flux by AWS designation, the applicable standard code requirements should be carefully reviewed to assure a complete understanding of the designation reference. This is necessary because the AWS designation systems are not consistent. For example, in the case of electrodes for shielded metal arc welding (AWS A5.1), the first two or three digits indicate the nominal tensile strength classification, in ksi, of the filler metal and the final two digits indicate the type of coating. For metric designations, the first two digits times 10 indicate the nominal tensile strength classification in MPa. In the case of mild steel electrodes for submerged arc welding (AWS A5.17), the first one or two digits times 10 indicate the nominal tensile strength classification for both U.S. Customary and metric units, while the final digit or digits times 10 indicate the testing temperature in degrees Celsius, for filler metal impact tests. In the case of low-alloy steel covered arc welding electrodes (AWS A5.5), certain portions of the designation indicate a requirement for stress relief, while others indicate no stress relief requirement.

Engineers do not, in general, specify the exact filler metal to be employed on a particular structure. Rather, the decision as to which welding process and which filler metal is to be utilized, is usually left with the fabricator or erector. To ensure that the proper filler metals are used, codes restrict the usage of certain filler materials, or impose qualification testing to prove the suitability of the specific electrode.

SECTION C1.4
LOADS AND LOAD COMBINATIONS

The load factors and load combinations are developed based on the recommended minimum loads given in SBC-301.

The load factors and load combinations recognize that when several loads act in combination with the dead load (e.g., dead plus live plus wind), only one of these takes on its maximum lifetime value, while the other load is at its "arbitrary point-in-time value" (i.e., at a value which can be expected to be on the structure at any time). For example, under dead, live, and wind loads the following combinations are appropriate:

$$\gamma_D D + \gamma_L L \qquad \text{(C1.4-1)}$$

$$\gamma_D D + \gamma_{La} L_a + \gamma_W W \qquad \text{(C1.4-2)}$$

$$\gamma_D D + \gamma_L L + \gamma_{W_a} W_a \qquad \text{(C1.4-3)}$$

Where γ is the appropriate load factor as designated by the subscript symbol. Subscript, "a" refers to an "arbitrary point-in-time" value.

The mean value of arbitrary point-in-time live load L_a is on the order of 0.24 to 0.4 times the mean maximum lifetime live load L for many occupancies, but its dispersion is far greater. The arbitrary point-in-time wind load W_a, acting in conjunction with the maximum lifetime live load, is the maximum daily wind. It turns out that $\gamma_{W_a} W_a$ is a negligible quantity so only two load combinations remain:

$$1.2D + 1.6L \qquad \text{(C1.4-4)}$$

$$1.2D + 0.5L + 1.3W \qquad \text{(C1.4-5)}$$

The load factor 0.5 assigned to L in the second formula reflects the statistical properties of L_a, but to avoid having to calculate yet another load, it is reduced so it can be combined with the maximum lifetime wind load.

SECTION C1.5
DESIGN BASIS

C1.5.1 **Required Strength at Factored Loads**. LRFD permits the use of both elastic and plastic structural analyses. LRFD provisions result in essentially the same methodology for, and end product of, plastic design except that the LRFD provisions tend to be slightly more liberal, reflecting added experience and the results of further research.

In some circumstances, as in the proportioning of the bracing members that carry no calculated forces (see Section 3.3) and of connection components (see Item 10.1.7), the required strength is explicitly stated in the Code requirements.

C1.5.2 **Limit States.** A limit state is a condition which represents the limit of structural usefulness. Limit states may be dictated by functional requirements, such as maximum deflections or drift; they may be conceptual, such as plastic hinge or mechanism formation; or they may represent the actual collapse of the whole or part of the structure, such as fracture or instability. Design criteria ensure that a limit state is violated only with an acceptably small probability by selecting the combination of load and resistance factors and nominal load and resistance values which will never be exceeded under the design assumptions.

Two kinds of limit states apply for structures: limit states of strength which define safety against extreme loads during the intended life of the structure, and limit states of serviceability which define functional requirements. The LRFD Code requirements, like other structural codes, focuses on the limit states of strength because of overriding considerations of public safety for the life, limb, and property of human beings. This does not mean that limit states of serviceability are not important to the designer, who must equally ensure functional performance and economy of design. However, these latter considerations permit more exercise of judgment on the part of designers. Minimum considerations of public safety, on the other hand, are not matters of individual judgment and, therefore, code requirements dwell more on the limit states of strength than on the limit states of serviceability.

Limit states of strength vary from member to member, and several limit states may apply to a given member. The following limit states of strength are the most common: onset of yielding, formation of a plastic hinge, formation of a plastic mechanism, overall frame or member instability, lateral-torsional buckling, local buckling, tensile fracture, development of fatigue cracks, deflection instability, alternating plasticity, and excessive deformation. The most common serviceability limit states include unacceptable elastic deflections and drift, unacceptable vibrations, and permanent deformations.

C1.5.3 **Design for Strength**. The general format of the LRFD Code requirements is given by the formula:

$$\Sigma\gamma_i Q_i \leq \phi R_n \qquad (C1.5\text{-}1)$$

where

Σ = summation

i = type of load, i.e., dead load, live load, wind, etc.

Q_i = nominal load effect

γ_i = load factor corresponding to Q_i

$\Sigma\gamma_i Q_i$ = required strength

R_n = nominal strength

ϕ = resistance factor corresponding to R_n

ϕR_n = design strength

The left side of Equation *(C1.5-1)* represents the required resistance computed by structural analysis based upon assumed loads, and the right side of Equation *(C1.5-1)* represents a limiting structural capacity provided by the selected members. In LRFD, the designer compares the effect of factored loads to the strength actually provided. The term design strength refers to the resistance or strength ϕR_n that must be provided by the selected member. The load factors γ and the resistance factors reflect the fact that loads, load effects (the computed forces and moments in the structural elements), and the resistances can be determined only to imperfect degrees of accuracy. The resistance factor ϕ is equal to or less than 1.0 because there is always a chance for the actual resistance to be less than the nominal value R_n computed by the equations given in Chapters 4 through 11. Similarly, the load factors γ reflect the fact that the actual load effects may deviate from the nominal values of Q_i computed from the specified nominal loads. These factors account for unavoidable inaccuracies in the theory, variations in the material properties and dimensions, and uncertainties in the determination of loads. They provide a margin of reliability to account for unexpected loads. They do not account for gross error or negligence. The LRFD Code requirements is based on (1) probabilistic models of loads and resistance, (2) a calibration of the LRFD criteria to the ASD Code requirements for selected members, and (3) the evaluation of the resulting criteria by judgment and past experience aided by comparative design office studies of representative structures.

C1.5.4 **Design for Serviceability and Other Considerations**: Nominally, serviceability should be checked at the unfactored loads. For combinations of gravity and wind or seismic loads some additional reduction factor may be warranted.

CHAPTER 2
DESIGN REQUIREMENTS

SECTION C2.5
LOCAL BUCKLING

For the purposes of these Code requirements, steel sections are divided into compact sections, non-compact sections, and sections with slender compression elements. Compact sections are capable of developing a fully plastic stress distribution and they possess a rotational capacity of approximately 3 before the onset of local buckling (Yura, Galambos, and Ravindra, 1978). Non-compact sections can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender compression elements buckle elastically before the yield stress is achieved.

The dividing line between compact and non-compact sections is the limiting width-thickness ratio λ_p. For a section to be compact, all of its compression elements must have width-thickness ratios equal to or smaller than the limiting λ_p.

TABLE C2.5-1
Limiting Width-Thickness Ratios for Compression Elements

Description of Element	Width-Thickness Ratio	Limiting Width-thickness Ratios λ_p	
		Non-seismic	Seismic
Flanges of I-shaped sections (including hybrid sections) and channels in flexure [a]	b/t	$0.38\sqrt{E/F_y}$	$0.31\sqrt{E/F_y}$
Webs in combined flexural and axial compression	h/t_w	For $P_u/\phi_b P_y \le 0.125$	
		$3.76\sqrt{\frac{E}{F_y}}\left(1-\frac{2.75P_u}{\phi_b P_y}\right)$	$3.05\sqrt{\frac{E}{F_y}}\left(1-\frac{1.54P_u}{\phi_b P_y}\right)$
		For $P_u/\phi_b P_y > 0.125$	
		$1.12\sqrt{\frac{E}{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right)\ge 1.49\sqrt{\frac{E}{F_y}}$	
[a] For hybrid beams use F_{yf} in place of F_y			

A greater inelastic rotation capacity than provided by the limiting values λ_p given in Table C2.5-1 may be required for some structures in areas of high seismicity. It has been suggested that in order to develop a ductility of from 3 to 5 in a structural member, ductility factors for elements would have to lie in the range of 5 to 15. Thus, in this case it is prudent to provide for an inelastic rotation of 7 to 9 times the elastic rotation (Chopra and Newmark, 1980). In order to provide for this rotation capacity, the limits λ_p for local flange and web buckling would be as shown in Table C2.5-1 (Galambos, 1976).

More information on seismic design is contained in the AISC Seismic Provisions for Structural Steel Buildings (AISC, 1997) and the Seismic Provisions for Structural Steel Buildings Supplement No. 1 (AISC, 1999).

Another limiting width-thickness ratio is λ_r, representing the distinction between non-compact sections and sections with slender compression elements. As long as the width-thickness ratio of a compression element does not exceed the limiting value λ_r, local elastic buckling will not govern its strength. However, for those cases where the width-thickness ratios exceed λ_r, elastic buckling strength must be considered. A design procedure for such slender-element compression sections, based on elastic buckling of plates, is given in Section 2.5.3. The effective width Equation 2.5-12 applies strictly to stiffened elements under uniform compression. It does not apply to cases where the compression element is under stress gradient. A method of dealing with the stress gradient in a compression element is provided in Section 2.2 of the AISI Code requirements for the Design of Cold-Formed Steel Structural Members (1996). Exceptions are girders with slender webs. Such plate girders are capable of developing post-buckling strength in excess of the elastic buckling load. A design procedure for plate girders including tension field action is given in Section 7.

The values of the limiting ratios λ_p and λ_r specified in Table 2.5-1 are similar to those in AISC (1989) and Table 2.3.3.3 of Galambos (1976), except that: (1) λ_p = $0.38\sqrt{E/F_y}$, limited in Galambos (1976) to indeterminate beams when moments are determined by elastic analysis and to determinate beams, was adopted for all conditions on the basis of Yura et al. (1978); and (2) $\lambda_p = 0.045E/F_y$, for plastic design of circular hollow sections was obtained from Sherman (1976).

The high shape factor for circular hollow sections makes it impractical to use the same slenderness limits to define the regions of behavior for different types of loading. In Table 2.5-1, the values of λ_p for a compact shape that can achieve the plastic moment, and λ_r for bending, are based on an analysis of test data from several projects involving the bending of pipes in a region of constant moment (Sherman and Tanavde, 1984, and Galambos, 1998). The same analysis produced the equation for the inelastic moment capacity in Table 6.1-1 in Section 6.1. However, a more restrictive value of λ_p is required to prevent inelastic local buckling from limiting the plastic hinge rotation capacity needed to develop a mechanism in a circular hollow beam section (Sherman, 1976).

The values of λ_r for axial compression and for bending are both based on test data. The former value has been used in building code requirements since 1968 (Winter, 1970). Sections 2.5 and 6.1 also limit the diameter-to-thickness ratio for any circular section to $0.45E/F_y$. Beyond this, the local buckling strength decreases rapidly, making it impractical to use these sections in building construction.

Following the SSRC recommendations (Galambos, 1998) and the approach used for other shapes with slender compression elements, a Q factor is used for circular sections to account for interaction between local and column buckling. The Q factor is the ratio between the local buckling stress and the yield stress. The local buckling stress for the circular section is taken from the inelastic AISI criteria (Winter, 1970) and is based on tests conducted on fabricated and manufactured cylinders. Subsequent tests on fabricated cylinders (Galambos, 1998) confirm that this equation is conservative.

The definitions of the width and thickness of compression elements agree with the 1978 AISC ASD Code requirements with minor modifications. Their applicability extends to sections formed by bending and to unsymmetrical and hybrid sections.

For built-up I-shaped sections under axial compression, modifications have been made to the flange local buckling criterion to include web-flange interaction. The k_c in the λ_r limit, in Equations 2.5-7 and 2.5-8 and the elastic buckling Equation 2.5-8 are the same that are used for flexural members. Theory indicates that the web-flange interaction in axial compression is at least as severe as in flexure. Rolled shapes are excluded from this criterion because there are no standard sections with proportions where the interaction would occur. In built-up sections where the interaction causes a reduction in the flange local buckling strength, it is likely that the web is also a thin stiffened element.

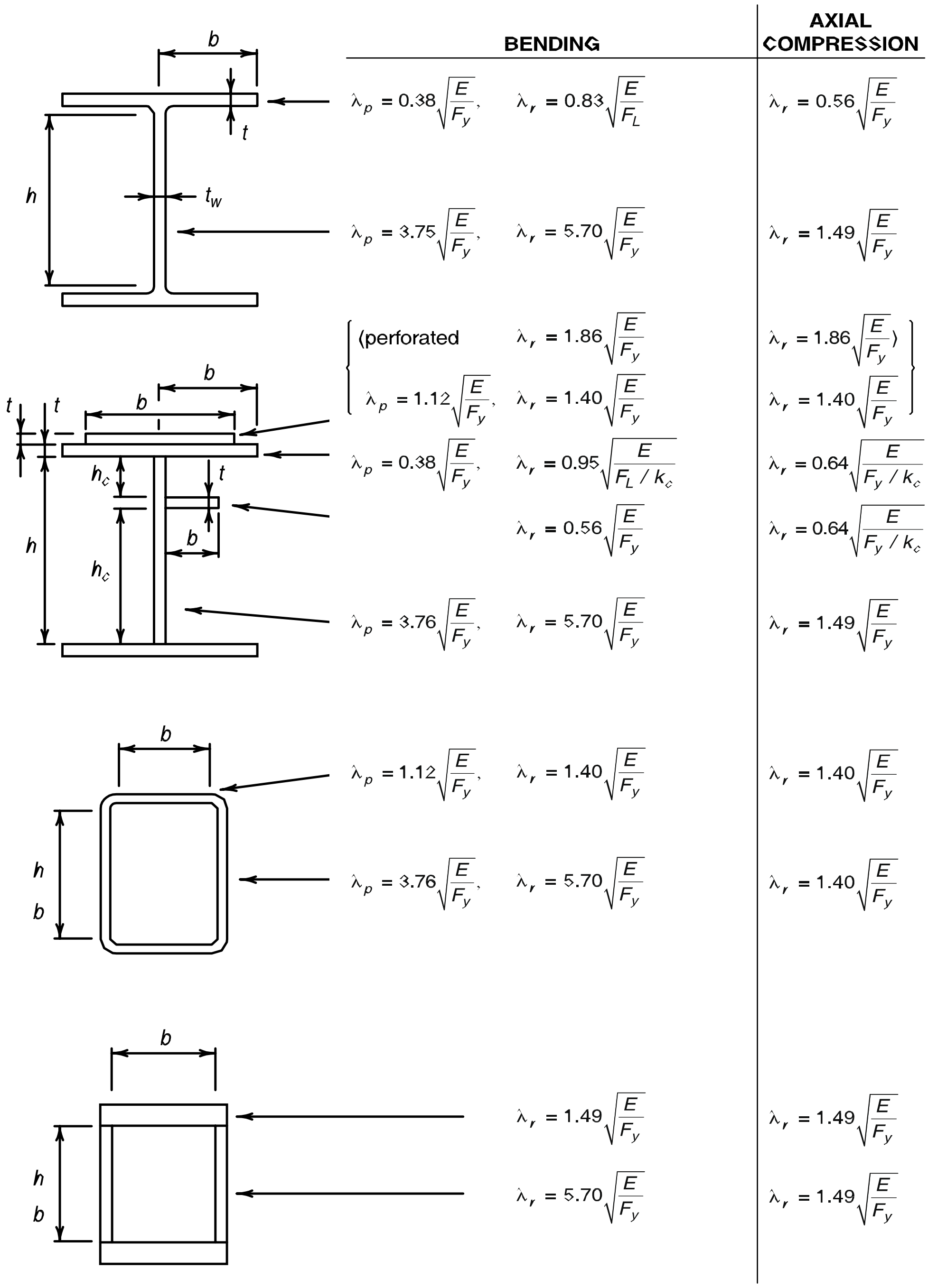


Figure. C2.5-1 Selected examples of Table 2.5-1 requirements.

The k_c factor accounts for the interaction of flange and web local buckling demonstrated in experiments conducted by Johnson (1985). The maximum limit of 0.763 corresponds to $F_{cr} = 0.69E / \lambda^2$ which was used as the local buckling strength in earlier editions of both the ASD and LRFD Code requirements. An

$h/t_w = 27.5$ is required to reach $k_c = 0.763$. Fully fixed restraint for an unstiffened compression element corresponds to $k_c = 1.3$ while zero restraint gives $k_c = 0.42$. Because of web-flange interactions it is possible to get $k_c < 0.42$ from the new k_c formula. If $h/t_w > 5.70\sqrt{E/F_y}$ use $h/t_w = 5.70\sqrt{E/F_y}$ in the k_c equation, which corresponds to the 0.35 limit.

Illustrations of some of the requirements of Table 2.5-1 of SBC 306 are shown in Figure C2.5-1.

SECTION C2.7
LIMITING SLENDERNESS RATIOS

Chapters 4 and 5 provide reliable criteria for resistance of axially loaded members based on theory and confirmed by tests for all significant parameters including slenderness. The advisory upper limits on slenderness contained in Section 2.7 are based on professional judgment and practical considerations of economics, ease of handling, and care required to minimize inadvertent damage during fabrication, transport, and erection. Out-of-straightness within reasonable tolerances does not affect the strength of tension members, and the effect of out-of-straightness within specified tolerances on the strength of compression members is accounted for in formulas for resistance. Applied tension tends to reduce, whereas compression tends to amplify, out-of-straightness. Therefore, more liberal criteria are suggested for tension members, including those subject to small compressive forces resulting from transient loads such as earthquake and wind. For members with slenderness ratios greater than 200, these compressive forces correspond to $\phi_c F_{cr}$ less than 18 MPa.

CHAPTER 3
FRAMES AND OTHER STRUCTURES

SECTION C3.1
SECOND ORDER EFFECTS

For frames under combined gravity and lateral loads, drift Δ (horizontal deflection caused by applied loads) occurs at the start of loading. In un-braced frames, additional secondary bending moments, known as the $P\Delta$ moments, may be developed in the columns and beams of the lateral load-resisting systems in each story. P is the total gravity load above the story and Δ is the story drift. As the applied load increases, the $P\Delta$ moments also increase. Therefore, the $P\Delta$ effect must often be accounted for in frame design. Similarly, in braced frames, increases in axial forces occur in the members of the bracing systems; however, such effects are usually less significant. The designer should consider these effects for all types of frames and determine if they are significant. Since $P\Delta$ effects can cause frame drifts to be larger than those calculated by ignoring them, they should also be included in the service load drift analysis when they are significant.

In un-braced frames designed by plastic analysis, the limit of $0.75\phi_c P_y$ on column axial loads has been retained to help ensure stability.

The designer may use second-order elastic analysis to compute the maximum factored forces and moments in a member. These represent the required strength. Alternatively, for structures designed on the basis of elastic analysis, the designer may use first order analysis and the amplification factors B_1 *and* B_2.

In the general case, a member may have first order moments not associated with sidesway which are multiplied by B_1, and first order moments produced by forces causing sidesway which are multiplied by B_2.

The factor B_2 applies only to moments caused by forces producing sidesway and is calculated for an entire story. In building frames designed to limit Δ_{oh}/L to a predetermined value, the factor B_2 may be found in advance of designing individual members.

Drift limits may also be set for design of various categories of buildings so that the effect of secondary bending can be insignificant. It is conservative to use the B_2 factor with the sum of the sway and the no-sway moments, i.e., with $M_{lt} + M_{nt}$.

The two kinds of first order moment M_{nt} and M_{lt} may both occur in sidesway frames from gravity loads. M_{nt} is defined as a moment developed in a member with frame sidesway prevented. If a significant restraining force is necessary to prevent sidesway of an unsymmetrical structure (or an un-symmetrically loaded symmetrical structure), the moments induced by releasing the restraining force will be M_{lt} moments, to be multiplied by B_2. In most reasonably symmetric frames, this effect will be small. If such a moment B_2M_{lt} is added algebraically to the B_1M_{nt} moment developed with sidesway prevented, a fairly accurate value of M_u will result. End moments produced in sidesway frames by lateral loads from wind or earthquake will always be M_{lt} moments to be multiplied by B_2.

When first order end moments in members subjected to axial compression are magnified by B_1 and B_2 factors, equilibrium requires that they be balanced by moments in connected members. Connections shall also be designed to resist the magnified end moments.

For beam columns with transverse loadings, the second-order moment can be approximated by using the following equation

$$C_m = 1 + \psi P_u / P_{e1} \qquad \text{(C3.1-1)}$$

for simply supported members

where

ψ = $\dfrac{\pi^2 \delta_o EI}{M_o L^2} - 1$

δ_o = maximum deflection due to transverse loading, mm

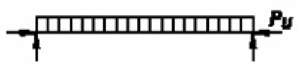
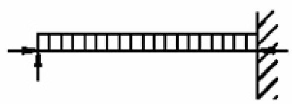
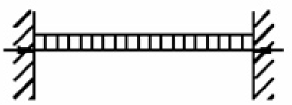
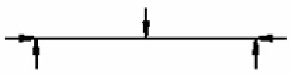
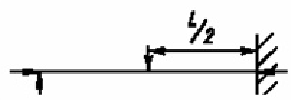
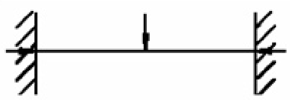
M_o = maximum factored design moment between supports due to transverse loading, N-mm

For restrained ends, some limiting cases are given in Table C3.1-1 together with two cases of simply supported beam-columns. These values of C_m are always used with the maximum moment in the member. For the restrained-end cases, the values of B_1 will be most accurate if values of $K < 1.0$ corresponding to the end boundary conditions are used in calculating P_{e1}. In lieu of using the equations above, $C_m = 1.0$ can be used conservatively for transversely loaded members with unrestrained ends and 0.85 for restrained ends.

If, as in the case of a derrick boom, a beam-column is subject to transverse (gravity) load and a calculable amount of end moment, the value δ_0 should include the deflection between supports produced by this moment.

Stiffness reduction adjustment due to column inelasticity is permitted.

TABLE C3.1-1
Amplification Factors for ψ and C_m

Case	Ψ	C_m
	0	1.0
	−0.4	$1 - 0.4\dfrac{P_u}{P_{e1}}$
	−0.4	$1 - 0.4\dfrac{P_u}{P_{e1}}$
	−0.2	$1 - 0.2\dfrac{P_u}{P_{e1}}$
	−0.3	$1 - 0.3\dfrac{P_u}{P_{e1}}$
	−0.2	$1 - 0.2\dfrac{P_u}{P_{e1}}$

SECTION C3.2
FRAME STABILITY

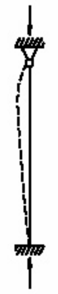
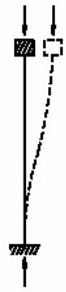
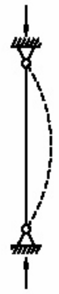
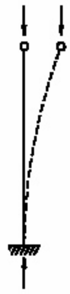
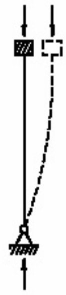
The stability of structures must be considered from the standpoint of the structure as a whole, including not only the compression members, but also the beams, bracing system, and connections (Galambos, 1998). The stability of individual elements must also be provided.

The effective length concept is one method of estimating the interaction effects of the total frame on a compression element being considered. This concept uses K factors to equate the strength of a framed compression element of length L to an equivalent pin-ended member of length KL subject to axial load only. Other rational methods are available for evaluating the stability of frames subject to gravity and side loading and individual compression members subject to axial load and moments.

The ratio *K*, effective column length to actual unbraced length, may be greater or less than 1.0, depending upon if the column is part of an unbraced frame or braced frame. The theoretical *K* values for six idealized conditions in which joint rotation and translation are either fully realized or nonexistent are tabulated in Table C3.2-1.

Also shown are suggested design values recommended by the Structural Stability Research Council (SSRC) for use when these conditions are approximated in actual design. In general, these suggested values are slightly higher than their theoretical equivalents, since joint fixity is seldom fully realized.

TABLE C3.2-1
K values for Columns

Buckled shape of column is shown by dashed line.	(a) 	(b) 	(c) 	(d) 	(e) 	(f) 
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	 Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free					

While in some cases masonry walls provide enough lateral support for building frames to control lateral deflection, light curtain wall construction and wide column spacing can create a situation where only the bending stiffness of the frame provides this support. In this case the effective length factor K for an unbraced length of column L is dependent upon the bending stiffness provided by the other in-plane members entering the joint at each end of the unbraced segment. If the combined stiffness provided by the beams is sufficiently small, relative to that of the unbraced column segments, KL could exceed two or more story heights.

Translation of the joints in the plane of a truss is inhibited and, due to end restraint, the effective length of compression members might be assumed to be less than the distance between panel points. However, it is usual practice to take K as equal to 1.0. If all members of the truss reached their ultimate load capacity simultaneously, the restraints at the ends of the compression members would be greatly reduced.

Once a trial selection of framing members has been made, the use of the alignment chart in Figures C3.2-1a and b affords a fairly rapid method for determining adequate *K* values. However, it should be noted that this alignment chart is based upon assumptions of idealized conditions which seldom exist in real structures (ASCE Task Committee on Effective Length, 1997).

The alignment chart for sidesway uninhibited shown in Figure C3.2-1b is based on the following equation:

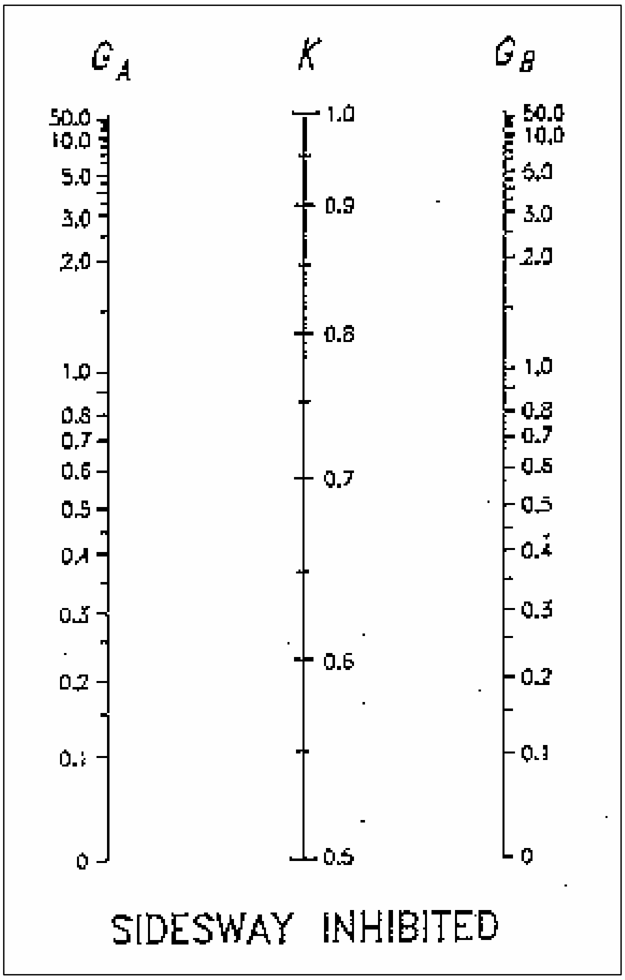
$$\frac{G_A G_B (\pi / K)^2 - 36}{6(G_A + G_B)} - \frac{(\pi / K)}{\tan(\pi / K)} = 0 \qquad \text{(C3.2-1)}$$

with *G* defined as

$$G = \frac{\Sigma(EI/L)_c}{\Sigma(EI/L)_g} \qquad \text{(C3.2-2)}$$

The expression for *G* given in the footnote of the alignment chart has assumed that E of the beams and columns are the same. However, the alignment chart is valid for different materials if Equation C3.2-2 is used. An equation for the sidesway-inhibited chart can be found in ASCE Task Committee on Effective Length (1997).

The theoretical K-factors that are less than 1.0 (Cases (a) and (b) in Table C3.2-1 and the sidesway inhibited alignment chart in Figure C3.2-1a, are based on the assumption that there is no relative lateral movement of the ends of the column. When bracing is proportioned by the requirements of Section 3.3, *K* equal to 1.0 should be used, not values less than 1.0, because a small relative movement of the brace points is anticipated.



Notes for Figure C3.2-1a and b: The subscripts A and B refer to the joints at the two ends of the column section being considered. G is defined as

$$G = \frac{\Sigma(I_c / L_c)}{\Sigma(I_g / L_g)}$$

in which Σ indicates a summation of all members rigidly connected to that joint and lying on the plane in which buckling of the column is being considered. I_c *is the moment of inertia and* L_c *the unsupported length of a column section, and* I_g *is the moment of inertia and* L_g *the unsupported length of a girder or other restraining member.* I_c *and* I_g *are taken about axes perpendicular to the plane of buckling being considered.*

For column ends supported by but not rigidly connected to a footing or foundation, G is theoretically infinity, but, unless actually designed as a true friction-free pin, may be taken as "10" for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as zero.

Figure. C3.2-1(a) Alignment chart for effective length of columns in continuous frames – Sidesway Inhibited.

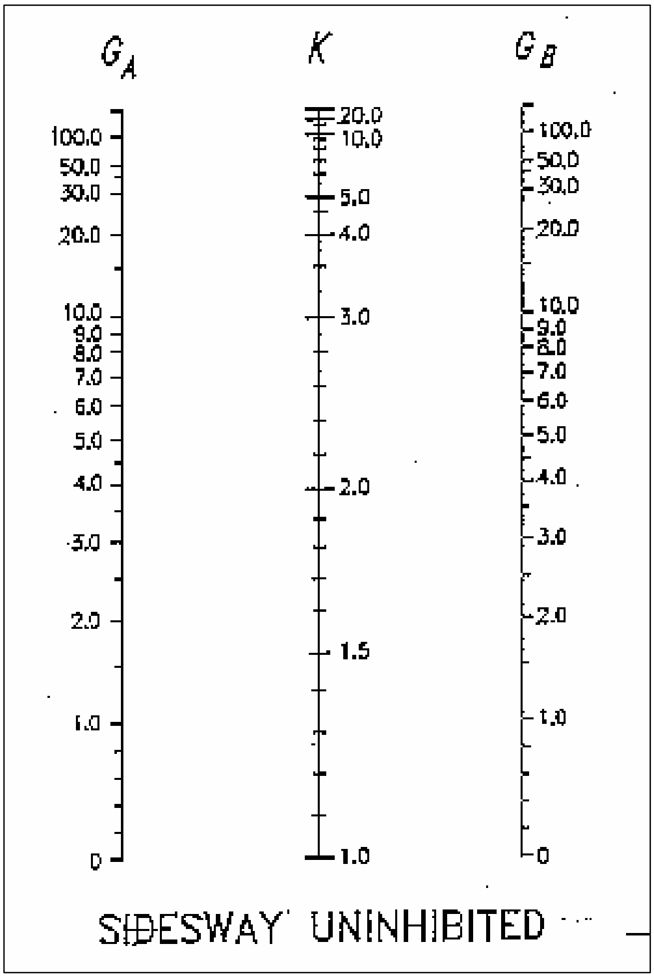


Figure. C3.2-1(b) Alignment chart for effective length of columns in continuous frames – Sidesway Uninhibited.

SECTION C3.3 STABILITY BRACING

C3.3.1 **Scope.** The design requirements consider two general types of bracing systems, relative and nodal, as shown in Figure C3.3-1 A relative column brace system (such as diagonal bracing or shear walls) is attached to two locations along the length of the column that defines the unbraced length. The relative brace system shown consists of the diagonal and the strut that controls the movement at one end of the unbraced length, A, with respect to the other end of the unbraced length, *B*. The diagonal and the strut both contribute to the strength and stiffness of the relative brace system. However, when the strut is a floor beam, its stiffness is large compared to the diagonal so the diagonal controls the strength and stiffness of the relative brace. A nodal brace controls the movement only at the particular brace point, without direct interaction with adjacent braced points. Therefore, to define an unbraced length, there must be additional adjacent brace points as shown in Figure C3.3-1. The two nodal column braces at *C* and *D* that are attached to the rigid abutment define the unbraced length for which K = 1.0 can be used. For beams, a cross frame between two adjacent beams at mid-span is a nodal brace because it prevents twist of the beams only at the particular cross frame location. The unbraced length is half the span length. The twist at the ends of the two beams is prevented by the beam-to-column connections at the end supports. Similarly, a nodal lateral brace attached at mid-span to the top flange of the beams and a rigid support assumes that there is no lateral movement at the column locations.

The brace requirements will enable a member to potentially reach a maximum load based on the unbraced length between the brace points and $K = 1.0$.

Winter (1958 and 1960) developed the concept of dual criterion for bracing design, strength and stiffness. The brace force is a function of the initial column out-of-straightness, Δ_o, and the brace stiffness, β. For a relative brace system, the relationship between column load, brace stiffness and sway displacement is shown in Figure C3.3-2. If $\beta = \beta_i$, the critical brace stiffness for a perfectly plumb member, then $P = P_e$ only if the sway deflection gets very large. Unfortunately, such large displacements produce large brace forces. For practical design, Δ must be kept small at the factored load level.

The brace stiffness requirements, β_{br}, for frames, columns, and beams were chosen as twice the critical stiffness. The $\phi = 0.75$ specified for all brace stiffness requirements is consistent with the implied resistance factor for elastic Euler column buckling, i.e. $0.877 \times \phi_c = 0.75$. For the relative brace system shown in Figure C3.3-2, $\beta_{br} = 2\beta_i$ gives $P_{br} = 0.4\% \ P_e$ for $\Delta_o = 0.002L$. If the brace stiffness provided, β_{act}, is different from the requirement, then the brace force or brace moment can be multiplied by the following factor:

$$\frac{1}{2 - \dfrac{\beta_{br}}{\beta_{act}}} \qquad \text{(C3.3-1)}$$

No ϕ is specified in the brace strength requirements since ϕ is included in the component design strength provisions in other chapters of this Code requirements.

The initial displacement, Δ_o, for relative and nodal braces is defined with respect to the distance between adjacent braces. The initial Δ_o is a displacement from the straight position at the brace points caused by sources other than brace elongations from gravity loads or compressive forces, such as displacements caused by wind or other lateral forces, erection tolerances, column shortening, etc. The brace force recommendations for frames, columns and beam lateral bracing are based on an assumed $\Delta_o = 0.002L$, where L is the distance between adjacent brace points. For torsional bracing of beams, an initial twist angle, θ_o, is assumed where $\theta_o = 0.002L/h_o$, and h_o is the distance between flange centroids. For other Δ_o and θ_o values, use direct proportion to modify the brace strength requirements, P_{br} and M_{br}. For cases where it is unlikely that all columns in a story are out-of-plumb in the same direction, Chen and Tong (1994) recommend an average $\Delta_o = 0.002L/\sqrt{n_o}$ where n_o columns, each with a random Δ_o, are to be stabilized by the brace system. This reduced Δ_o would be appropriate when combining the stability brace forces with wind and seismic forces.

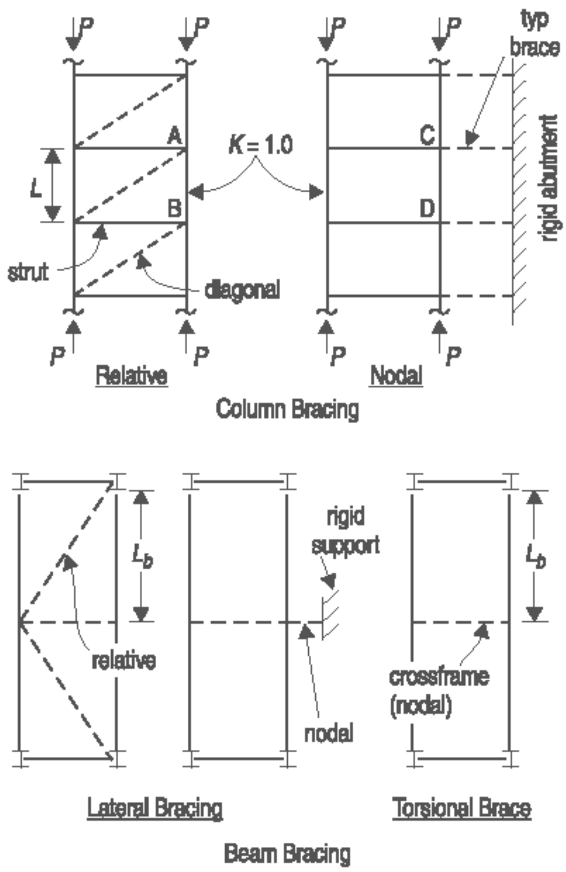


Figure. C3.3-1. Types of bracing

Brace connections, if they are flexible or can slip, should be considered in the evaluation of the bracing stiffness as follows:

$$\frac{1}{\beta_{act}} = \frac{1}{\beta_{conn}} + \frac{1}{\beta_{brace}} \qquad \text{(C3.3-2)}$$

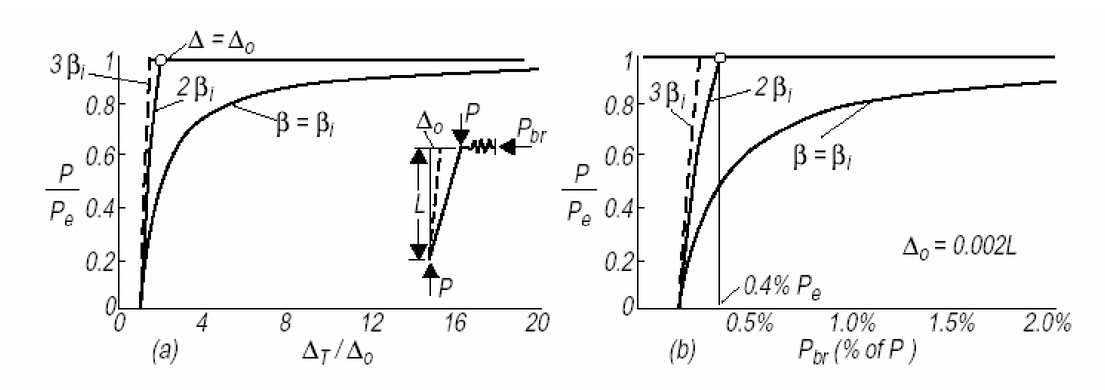


Figure. C3.3-2. Effect of initial out-of-plumbness.

The brace system stiffness, β_{act}, is less than the smaller of the connection stiffness, β_{conn}, or the stiffness of the brace, β_{brace}. Slip in connections with standard holes need not be considered except when only a few bolts are used. When evaluating the bracing of rows of columns or beams, consideration must be given to the accumulation of the brace forces along the length of the brace that results in a different displacement at each beam or column location. In general, brace forces can be minimized by increasing the number of braced bays and using stiff braces.

C3.3.3 **Columns.** For nodal column bracing, the critical stiffness is a function of the number of intermediate braces (Winter, 1958 and 1960). For one intermediate brace, $\beta_i = 2P/L_b$, and for many braces $\beta_i = 4P/L_b$. The relationship between the critical stiffness and the number of braces, n, can be approximated (Yura, 1995) as $\beta_i = N_iP/L_b$, where $N_i = 4 - 2/n$. The most severe case (many braces) was adopted for the brace stiffness requirement $\beta_{br} = 2 \times 4P/L_b$. The brace stiffness, Equation C3.3-6, can be reduced by the ratio, $N_i/4$, to account for the actual number of braces.

The unbraced length, L_b, in Equations C3.3-4 and C3.3-6 is assumed to be equal to the length L_q that enables the column to reach P_u. When the actual bracing spacing is less than L_q, the calculated required stiffness may become quite conservative since the stiffness equations are inversely proportional to L_b. In such cases, L_q can be substituted for L_b.

Winter's rigid model would derive a brace force of 0.8 percent P_u which accounts only for lateral displacement force effects. To account for the additional force due to member curvature, this theoretical force has been increased to one percent P_u.

C3.3.4 **Beams.** Beam bracing must prevent twist of the section, not lateral displacement. Both lateral bracing (for example, joists attached to the compression flange of a simply supported beam) and torsional bracing (for example, a cross frame or diaphragm between adjacent girders) can effectively control twist. Lateral bracing systems that are attached near the beam centroid are ineffective. For beams with double curvature, the inflection point cannot be considered a brace point because twist occurs at that point (Galambos, 1998). A lateral brace on one flange near the inflection point also is ineffective. In double curvature cases, the lateral brace near the inflection point must be attached to both flanges to prevent twist, or torsional bracing must be used. The beam brace requirements are based on the recommendations by Yura (1993).

C3.3.4.1 **Lateral Bracing.** For lateral bracing, the following stiffness requirement was derived following Winter's approach:

$$\beta_{br} = 2N_i\,(C_bP_f)\,C_tC_d\,/\,\phi L_b \tag{C3.3-3}$$

where

N_i = 1.0 for relative bracing

= $(4-2/n)$ for discrete bracing

n = number of intermediate braces

P_f = beam compressive flange force

= $\pi^2 EI_{yc}/L_b^2$

I_{yc} = out-of-plane moment of inertia of the compression flange

C_b = moment modifier from Chapter 6

C_t = accounts for top flange loading (use C_t = 1.0 for centroidal loading)

= $1 + (1.2/n)$

C_d = double curvature factor (compression in both flanges)

= $1 + (M_S/M_L)^2$

M_S = smallest moment causing compression in each flange

M_L = largest moment causing compression in each flange

The C_d factor varies between 1.0 and 2.0 and is applied only to the brace closest to the inflection point. The term $(2N_iC_t)$ can be conservatively approximated as 10 for any number of nodal forces and 4 for relative bracing and (C_bP_f) can be approximated by M_u / h which simplifies Equation C3.3-3 to the stiffness requirements given by Equations 3.3-8 and 3.3-10. Equation C3.3-3 can be used in lieu of Equations 3.3-8 and 3.3-10.

The brace strength requirement for relative bracing is

$$P_{br} = 0.004 M_u C_t C_d / h_o \qquad \text{(C3.3-4a)}$$

And for nodal bracing

$$P_{br} = 0.01 M_u C_t C_d / h_o \qquad \text{(C3.3-4b)}$$

They are based on an assumed initial lateral displacement of the compression flange of $0.002L_b$. The brace strength requirements of Equations 3.3-7 and 3.3-9 are derived from Equations C3.3-4a and C3.3-4b assuming top flange loading $(C_t = 2)$. Equations C3.3-4a and C3.3-4b can be used in lieu of Equations 3.3-7 and 3.3-9, respectively.

C3.3.4.2 **Torsional Bracing.** Torsional bracing can either be attached continuously along the length of the beam (for example, metal deck or slabs) or be located at discrete points along the length of the member (for example, cross frames). Torsional bracing attached to the tension flange is just as effective as a brace attached at mid depth or the compression flange. Partially restrained connections can be used if their stiffness is considered in evaluating the torsional brace stiffness.

The torsional brace requirements are based on the buckling strength of a beam with a continuous torsional brace along its length developed by Taylor and Ojalvo (1966) and modified for cross-section distortion by Yura (1993).

$$M_u \le M_{cr} = \sqrt{(C_{bu} M_o)^2 + \frac{C_b^2 EI_y \overline{\beta}_T}{2C_{tt}}} \qquad \text{(C3.3-5)}$$

The term $(C_{bu}M_o)$ is the buckling strength of the beam without torsional bracing. $C_{tt} = 1.2$ when there is top flange loading and $C_{tt} = 1.0$ for centroidal loading. $\overline{\beta}_T = n\beta_T / L$ is the continuous torsional brace stiffness per unit length or its equivalent when n nodal braces, each with a stiffness β_T, are used along the span L and the 2 accounts for initial out-of-straightness. Neglecting the un-braced beam buckling term gives a conservative estimate of the torsional brace stiffness requirement (Equation 3.3-13). A more accurate estimate of the brace requirements can be obtained by replacing M_u with $(M_u - C_{bu}M_o)$ in Equations 3.3-11 and 3.3-13. The β_{sec} term in Equations 3.3-12, 3.3-14 and 3.3-15 accounts for cross-section distortion. A web stiffener at the brace point reduces cross-sectional distortion and improves the effectiveness of a torsional brace. When a cross frame is attached near both flanges or a diaphragm is approximately the same depth as the girder, then web distortion will be insignificant so β_{sec} equals infinity. The required bracing stiffness, β_{Tb}, given by Equation 3.3-12 was obtained by solving the following expression that represents the brace system stiffness including distortion effects:

$$\frac{1}{\beta_T} = \frac{1}{\beta_{Tb}} + \frac{1}{\beta_{sec}} \qquad \text{(C3.3-6)}$$

The brace moment requirements are based on an assumed initial twist of $0.002L_b/h_o$.

Parallel chord trusses with both chords extended to the end of the span and attached to supports can be treated like beams. In Equations 3.3-7 through 3.3-11, M_u may be taken as the maximum compressive chord force times the depth of the truss to determine the brace strength and stiffness requirements. Cross-section distortion effects, β_{sec}, need not be considered when full-depth cross frames are used for bracing. When either chord does not extend to the end of the span, consideration should be given to control twist near the ends of the span by the use of cross frames or ties.

CHAPTER 4
TENSION MEMBERS

SECTION C4.1
DESIGN TENSILE STRENGTH

Due to strain hardening, a ductile steel bar loaded in axial tension can resist, without fracture, a force greater than the product of its gross area and its coupon yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness, but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the reduction of area and other mechanical properties of the steel, the member can fail by fracture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and fracture of the net area both constitute failure limit states.

The length of the member in the net area is negligible relative to the total length of the member. As a result, the strain hardening condition is quickly reached and yielding of the net area at fastener holes does not constitute a limit state of practical significance.

SECTION C4.2
BUILT-UP MEMBERS

The slenderness ratio L/r of tension members other than rods, HSS, or straps should preferably not exceed the limiting value of 300. This slenderness limit recommended for tension members is not essential to the structural integrity of such members; it merely assures a degree of stiffness such that undesirable lateral movement (“slapping” or vibration) will be unlikely.

SECTION C4.3
PIN-CONNECTED MEMBERS AND EYEBARS

Forged eyebars have generally been replaced by pin-connected plates or eyebars thermally cut from plates. Provisions for the proportioning of eyebars contained in the LRFD Code requirements are based upon standards evolved from long experience with forged eyebars. Through extensive destructive testing, eyebars have been found to provide balanced designs when they are thermally cut instead of forged. The somewhat more conservative rules for pin-connected members of non-uniform cross section and those not having enlarged “circular” heads are likewise based on the results of experimental research.

Somewhat stockier proportions are provided for eyebars and pin-connected members fabricated from steel having a yield stress greater than 485 MPa, in order to eliminate any possibility of their “dishing” under the higher design stress.

CHAPTER 5
COLUMN AND OTHER COMPRESSION MEMBERS

SECTION C5.1
EFFECTIVE LENGTH AND SLENDERNESS LIMITATIONS

C5.1.1 **Effective Length.** The Commentary on Section 3.2 regarding frame stability and effective length factors applies here. Further analytic methods, formulas, charts, and references for the determination of effective length are provided in Chapter 15 of the SSRC Guide (Galambos, 1998).

SECTION C5.2
DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING

Equations 5.2-2 and 5.2-3 are based on a reasonable conversion of research data into design equations and are essentially the same curve as column-strength curve 2P of the Structural Stability Research Council which is based on an initial out-of-straightness curve of $l/1500$ (Bjorhovde, 1972 and 1988; Galambos, 1998; Tide, 1985).

Equations 5.2-2 and 5.2-3 can be restated in terms of the more familiar slenderness ratio Kl/r. First, Equation 5.2-2 is expressed in exponential form as:

$$F_{cr} = [\exp(-0.419\lambda_c^2)]F_y \qquad \text{(C5.2-1)}$$

Note that exp(x) is identical to e^x. Substitution of λ_c according to definition of λ_c in Section 5.2 gives,

For $\dfrac{Kl}{r} \leq 4.71\sqrt{\dfrac{E}{F_y}}$

$$F_{cr} = \left\{\exp\left[-0.0424\frac{F_y}{E}\left(\frac{Kl}{r}\right)^2\right]\right\}F_y \qquad \text{(C5.2-2)}$$

For $\dfrac{Kl}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$

$$F_{cr} = \frac{0.877\pi^2 E}{\left(\dfrac{Kl}{r}\right)^2} \qquad \text{(C5.2-3)}$$

SECTION C5.3
DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL-TORSIONAL BUCKLING

Torsional buckling of symmetric shapes and flexural-torsional buckling of unsymmetric shapes are failure modes usually not considered in the design of hot-rolled columns. They generally do not govern, or the critical load differs very little from the weak axis planar buckling load. Such buckling loads may, however, control the capacity of symmetric columns made from relatively thin plate elements and unsymmetric columns. The AISC Design Guide, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997) provides an overview of the fundamentals and basic theory of torsional loading for structural steel members. Design examples are also included.

Tees that conform to the limits in Table C5.3-1 need not be checked for flexural-torsional buckling.

A simpler and more accurate design strength for the special case of tees and double-angles is based on Galambos (1991) wherein the y-axis of symmetry flexural-buckling strength component is determined directly from the column formulas.

The separate AISC *Specification for Load and Resistance Factor Design of Single-Angle Members* contains detailed provisions not only for the limit state of compression, but also for tension, shear, flexure, and combined forces.

The equations in 5.3-2 for determining the flexural-torsional elastic buckling loads of columns are derived in texts on structural stability. Since these equations for flexural-torsional buckling apply only to elastic buckling, they must be modified for inelastic buckling when $F_{cr} > 0.5F_y$. This is accomplished through the use of the equivalent slenderness factor $\lambda_e = \sqrt{F_y / F_e}$.

TABLE C5.3-1
Limiting Proportions for Tees

Shape	Ratio of Full Flange Width to Profile Depth	Ratio of Flange Thickness to Web or Stem Thickness
Built-up tees	≥ 0.50	≥ 1.25
Rolled tees	≥ 0.50	≥ 1.10

SECTION C5.4
BUILT-UP MEMBERS

Requirements for detailing and design of built-up members, which cannot be stated in terms of calculated stress, are based upon judgment and experience.

The longitudinal spacing of connectors connecting components of built-up compression members must be such that the slenderness ratio l/r of individual shapes does not exceed three-fourths of the slenderness ratio of the entire member. Additional requirements are imposed for built-up members consisting of angles. However, these minimum requirements do not necessarily ensure that the effective slenderness ratio of the built-up member is equal to that for the built-up member acting as a single unit. Section 5.4 gives formulas for modified slenderness ratios that are based on research and take into account the effect of shear deformation in the connectors. Equation 5.4-1 for snug tight intermediate connectors is empirically based on test results (Zandonini, 1985). Equation 5.4-2 is derived from theory and verified by test data. In both cases the end connection must be welded or slip-critical bolted (Aslani and Goel, 1991). The connectors must be designed to resist the shear forces which develop in the buckled member. The shear stresses are highest where the slope of the buckled shape is maximum (Bleich, 1952).

Maximum fastener spacing less than that required for strength may be needed to ensure a close fit over the entire faying surface of components in continuous contact. Specific requirements are given for weathering steel members exposed to atmospheric corrosion (Brockenbrough, 1983).

The provisions governing the proportioning of perforated cover plates are based upon extensive experimental research (Stang and Jaffe, 1948).

CHAPTER 6
BEAMS AND OTHER FLEXURAL MEMBERS

SECTION C6.1
DESIGN FOR FLEXURE

C6.1.1 **Yielding**. The bending strength of a laterally braced compact section is the plastic moment M_p. If the shape has a large shape factor (ratio of plastic moment to the moment corresponding to the onset of yielding at the extreme fiber), significant inelastic deformation may occur at service load if the section is permitted to reach M_p at factored load. The limit of $1.5M_y$ at factored load will control the amount of inelastic deformation for sections with shape factors greater than 1.5. This provision is not intended to limit the plastic moment of a hybrid section with a web yield stress lower than the flange yield stress. Yielding in the web does not result in significant inelastic deformations. In hybrid sections, $M_y = F_{yf} S$.

Lateral-torsional buckling cannot occur if the moment of inertia about the bending axis is equal to or less than the moment of inertia out of plane. Thus, for shapes bent about the minor axis and shapes with $I_x = I_y$, such as square or circular shapes, the limit state of lateral-torsional buckling is not applicable and yielding controls if the section is compact.

Three limit states must be investigated to determine the moment capacity of flexural members: lateral-torsional buckling (LTB), local buckling of the compression flange (FLB), and local buckling of the web (WLB). These limit states depend, respectively, on the beam slenderness ratio L_b/r_y, the width-thickness ratio b/t of the compression flange and the width-thickness ratio h/t_w of the web. For convenience, all three measures of slenderness are denoted by (λ).Variations in M_n with L_b are shown in Figure C6.1-1. Values of λ_p for FLB and WLB produce a compact section with a rotation capacity of about three (after reaching M_p) before the onset of local buckling, and therefore meet the requirements for plastic analysis of load effects (Commentary Section 2.5). On the other hand, values of λ_p for LTB do not allow plastic analysis because they do not provide rotation capacity beyond that needed to develop M_p. Instead $L_b \leq L_{pd}$ (Section 6.1.3) must be satisfied. Analyses to include restraint effects of adjoining elements are discussed in Galambos (1998). Analysis of the lateral stability of members with shapes not covered in this Chapter must be performed according to the available literature (Galambos, 1998).

See the Commentary for Section 2.5 for the discussion of the equation regarding the bending capacity of circular sections.

C6.1.2.1 **Doubly Symmetric Shapes and Channels with $L_b \leq L_r$**. The basic relationship between nominal moment M_n and unbraced length L_b is shown in Figure C6.1-1 for a compact section with $C_b = 1.0$. There are four principal zones defined on the basic curve by L_{pd}, L_p, *and* L_r. Equation 6.1-4 defines the maximum unbraced length L_p to reach M_p with uniform moment. Elastic lateral-torsional buckling will occur when the unbraced length is greater than L_r given by Equation 6.1-6. Equation 6.1-2 defines the inelastic lateral-torsional buckling as a straight line between the defined limits L_p *and* L_r. Buckling strength in the elastic region $L_b > L_r$ is given by Equation 6.1-13 for I-shaped members.

For other moment diagrams, the lateral buckling strength is obtained by multiplying the basic strength by C_b as shown in Figure C6.1-1. The maximum M_n, however, is limited to M_p. Note that L_p given by Equation 6.1-4 is merely a definition which has physical meaning when $C_b = 1.0$. For C_b greater than 1.0, larger unbraced lengths are permitted to reach M_p as shown by the curve for $C_b > 1.0$. For design, this length could be calculated by setting Equation 6.1-2 equal to M_p and solving this equation for L_b using the desired C_b value.

The equation

$$C_b = 1.75+1.05(M_1/M_2) +0.3(M_1/M_2)^2 \leq 2.3 \qquad \text{(C6.1-1)}$$

has been used since 1961 to adjust the flexural-torsional buckling equation for variations in the moment diagram within the unbraced length. This equation is applicable only to moment diagrams that are straight lines between braced points. Another equation

$$C_b = \frac{1}{0.6 - 0.4\dfrac{M_1}{M_2}} \leq 2.5 \qquad \text{(C6.1-2)}$$

fits the average value theoretical solutions when the beams are bent in reverse curvature and also provides a reasonable fit to the theory. If the maximum moment within the unbraced segment is equal to or larger than the end moment, $C_b = 1.0$ is used.

The equations above can be easily misinterpreted and misapplied to moment diagrams that are not straight within the unbraced segment. Kirby and Nethercot (1979) presented an equation which applies to various shapes of moment diagrams within the unbraced segment. Their equation has been adjusted slightly to the following

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \qquad \text{(C6.1-3)}$$

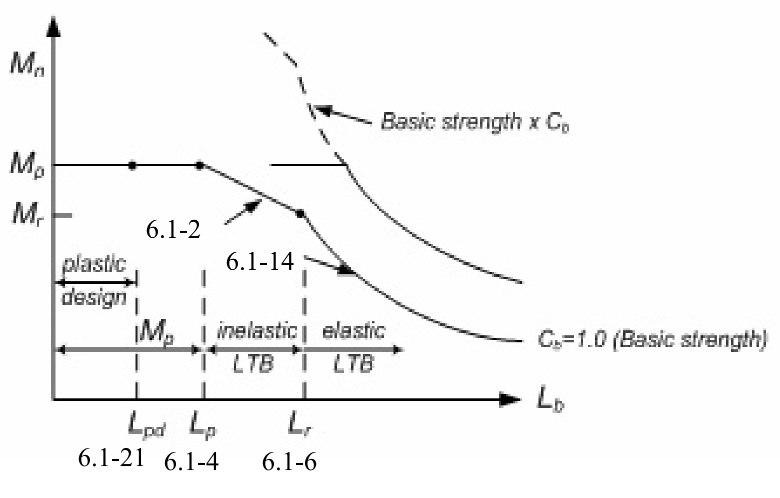


Figure. C6.1-1. Nominal moment as a function of unbraced length and moment gradient.

This equation gives more accurate solutions for fixed-end beams, and the adjusted equation reduces exactly to Equation C6.1-2 for a straight line moment diagram in single curvature. The C_b equation used in the SBC 306 is shown in Figure C6.1-2 for straight line moment diagrams. Other moment diagrams along with exact theoretical solutions in the SSRC Guide (Galambos, 1998) show good comparison with the new equation. The absolute values of the three interior quarter-point moments plus the maximum moment, regardless of its location are used in the equation. The maximum moment in the unbraced segment is always used for comparison with the resistance. The length between braces, not the distance to inflection points and C_b are used in the resistance equation.

It is still satisfactory to use the former C_b factor, Equation C6.1-1, for straight line moment diagrams within the unbraced length.

The elastic strength of hybrid beams is identical to homogeneous beams. The strength advantage of hybrid sections becomes evident only in the inelastic and plastic slenderness ranges.

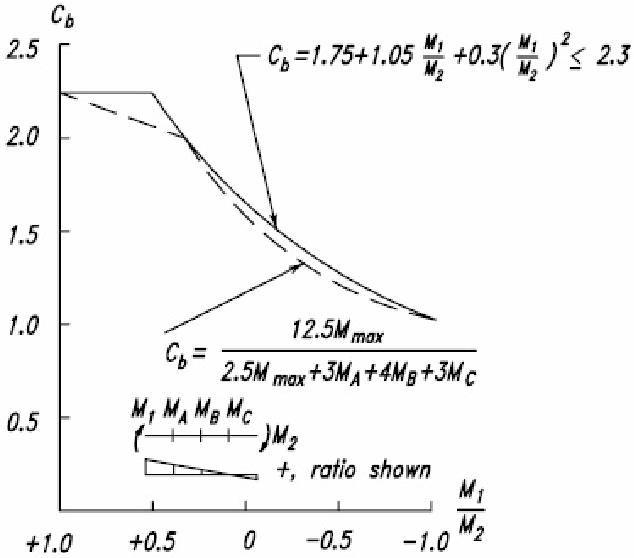


***Figure. C6.1-2.* C_b for a straight line moment diagram-prismatic beam.**

C6.1.2.2 Doubly Symmetric Shapes and Channels with $L_b > L_r$. The equation given in the SBC 306 assumes that the loading is applied along the beam centroidal axis. If the load is placed on the top flange and the flange is not braced, there is a tipping effect that reduces the critical moment; conversely, if the load is suspended from the bottom flange and is not braced, there is a stabilizing effect which increases the critical moment (Galambos, 1998). For unbraced top flange loading, the reduced critical moment may be conservatively approximated by setting the warping buckling factor X_2 to zero.

An effective length factor of unity is implied in these critical moment equations to represent a worst case pinned-pinned unbraced segment. Including consideration of any end restraint of the adjacent segments on the critical segment can increase its buckling capacity. The effects of beam continuity on lateral-torsional buckling have been studied and a simple and conservative design method, based on the analogy of end-restrained nonsway columns with an effective length factor less than one, has been proposed (Galambos, 1998).

C6.1.2.3 **Tees and Double-Angles**. The lateral-torsional buckling strength (LTB) of singly symmetric tee beams is given by a fairly complex formula (Galambos, 1998). Equation 6.1-15 is a simplified formulation based on Kitipornchai and Trahair (1980). See also Ellifritt, Wine, Sputo, and Samuel (1992).

The C_b used for I-shaped beams is unconservative for tee beams with the stem in compression. For such cases $C_b = 1.0$ is appropriate. When beams are bent in reverse curvature, the portion with the stem in compression may control the LTB resistance even though the moments may be small relative to other portions of the unbraced length with $C_b \approx 1.0$. This is because the LTB strength of a tee with the stem in compression may be only about one-fourth of the capacity for the stem in tension. Since the buckling strength is sensitive to the moment diagram, C_b has been conservatively taken as 1.0. In cases where the stem is in tension, connection details should be designed to minimize any end restraining moments which might cause the stem to be in compression.

C6.1.2.4 **Design by Plastic Analysis**. Equation 6.1-17 sets a limit on unbraced length adjacent to a plastic hinge for plastic analysis. There is a substantial increase in unbraced length for positive moment ratios (reverse curvature) because the yielding is confined to zones close to the brace points (Yura et al., 1978).

Equation 6.1-18 is an equation in similar form for solid rectangular bars and symmetric box beams. Equations 6.1-17 and 6.1-18 assume that the moment diagram within the unbraced length next to plastic hinge locations is reasonably linear. For nonlinear diagrams between braces, judgment should be used in choosing a representative ratio.

Equations 6.1-17 and 6.1-18 were developed to provide rotation capacities of at least 3.0, which are sufficient for most applications (Yura et al., 1978). When inelastic rotations of 7 to 9 are deemed appropriate in areas of high seismicity, as discussed in Commentary Section 2.5, Equation 6.1-17 would become:

$$L_{pd} = 0.086\left(\frac{E}{F_y}\right)r_y \qquad \text{(C6.1-4)}$$

SECTION C6.2
DESIGN FOR SHEAR

For unstiffened webs $k_v = 5.0$, therefore

$$1.10\sqrt{Ek_v / F_{yw}} = 2.45\sqrt{E / F_{yw}}, \text{and } 1.37\sqrt{Ek_v / F_{yw}} = 3.07\sqrt{E / F_{yw}}$$

For webs with $h / t_w \leq 1.10\sqrt{Ek_v / F_{yw}}$, the nominal shear strength V_n is based on shear yielding of the web, Equation 6.2-1. This h/t_w limit was determined by setting the critical stress causing shear buckling F_{cr} equal to the yield stress of the web F_{yw} in Equation 35 of Cooper, Galambos, and Ravindra (1978) and

Timoshenko and Gere (1961). When, $h/t_w > 1.10\sqrt{Ek_v/F_{yw}}$, the web shear strength is based on buckling. Basler (1961) suggested taking the proportional limit as 80 percent of the yield stress of the web. This corresponds to $h/t_w = (1.10/0.8)\sqrt{Ek_v/F_{yw}}$. Thus, when $h/t_w > 1.10\sqrt{Ek_v/F_{yw}}$, the web strength is determined from the elastic buckling stress given by Equation 6 of Cooper et al. (1978) and Timoshenko and Gere (1961):

$$F_{cr} = \frac{\pi^2 E k_v}{12(1-\nu^2)(h/t_w)^2} \qquad \text{(C6.2-1)}$$

The nominal shear strength, given by Equation 6.2-3, was obtained by multiplying F_{cr} by the web area and using $E = 200000\ MPa$ and $\nu = 0.3$. A straight line transition, Equation 6.2-2 is used between the limits

$$1.10\sqrt{Ek_v/F_{yw}} \text{ and } 1.37\sqrt{Ek_v/F_{yw}}$$

When designing plate girders, thicker unstiffened webs will frequently be less costly than lighter stiffened web designs because of the additional fabrication. If a stiffened girder design has economic advantages, the tension field method in Chapter 7 will require fewer stiffeners.

The equations in this section were established assuming monotonically increasing loads. If a flexural member is subjected to load reversals causing cyclic yielding over large portions of a web, such as may occur during a major earthquake, special design considerations may apply (Popov, 1980).

SECTION C6.3
WEB-TAPERED MEMBERS

C6.3.1 **General Requirements**. The design of wide-flange columns with a single web taper and constant flanges follows the same procedure as for uniform columns according to Section 5.2, except the column slenderness parameter λ_c for major axis buckling is determined for a slenderness ratio $K_\gamma L/r_{ox}$, and for minor axis buckling for K_L/r_{oy}, where K_γ is an effective length factor for tapered members, K is the effective length factor for prismatic members, and r_{ox} *and* r_{oy} are the radii of gyration about the x and the y axes, respectively, taken at the smaller end of the tapered member.

For stepped columns or columns with other than a single web taper, the elastic critical stress is determined by analysis or from data in reference texts or research reports (Chapters 11 and 13 in Timoshenko and Gere (1961), Bleich (1952), and Kitipornchai and Trahair (1980)), and then the same procedure of using λ_{eff} is utilized in calculating the factored resistance.

This same approach is recommended for open section built-up columns (columns with perforated cover plates, lacing, and battens) where the elastic critical buckling stress determination must include a reduction for the effect of shear. Methods for calculating the elastic buckling strength of such columns are given in Chapter 12 of the SSRC Guide (Galambos, 1998) and in Timoshenko and Gere (1961) and Bleich (1952).

C6.3.3 **Design Compressive Strength**. The approach in formulating $F_{a\gamma}$ of tapered columns is based on the concept that the critical stress for an axially loaded tapered column is equal to that of a prismatic column of different length, but of the same cross section as the smaller end of the tapered column. This has resulted in an equivalent effective length factor K_γ for a tapered member subjected to axial compression (Lee, Morrell, and Ketter, 1972). This factor, which is used to determine the value of S in Equations 6.3-2 and λ_c in Equation 5.2-3, can be determined accurately for a symmetrical rectangular rigid frame comprised of prismatic beams and tapered columns.

With modifying assumptions, such a frame can be used as a mathematical model to determine with sufficient accuracy the influence of the stiffness $\Sigma (I / b)_g$ of beams and rafters which afford restraint at the ends of a tapered column in other cases such as those shown in Figure C6.3-1 from Equations 6.3-2 and 5.2-3, the critical load P_{cr} can be expressed as $\pi^2 EI_\circ /(K_\gamma l)^2$. The value of K_γ can be obtained by interpolation, using the appropriate chart from Lee et al. (1972) and restraint modifiers G_T *and* G_B. In each of these modifiers the tapered column, treated as a prismatic member having a moment of inertia I_o, computed at the smaller end, and its actual length l, is assigned the stiffness I_o / l, which is then divided by the stiffness of the restraining members at the end of the tapered column under consideration.

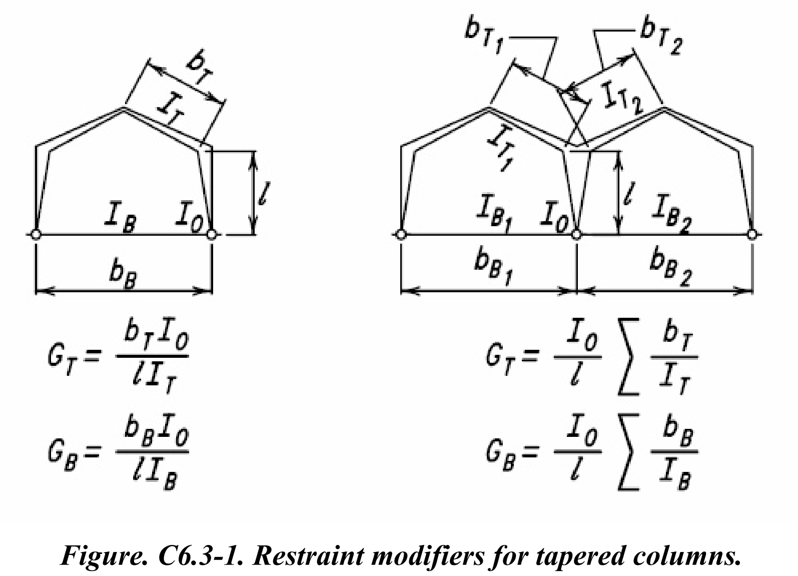


Figure. C6.3-1. Restraint modifiers for tapered columns.

C6.3.4 **Design Flexural Strength**. The development of the design bending stress for tapered beams follows closely with that for prismatic beams. The basic concept is to replace a tapered beam by an equivalent prismatic beam with a different length, but with a cross section identical to that of the smaller end of the tapered beam (Lee et al., 1972). This has led to the modified length factors h_s and h_w in Equations 6.3-6 and 6.3-7.

Equations 6.3-6 and 6-3-7 are based on total resistance to lateral buckling, using both St. Venant and warping resistance. The factor B modifies the basic $F_{b\gamma}$ to members, which are continuous past lateral supports. Categories a, b, and c of Section 6.3.4 usually apply; however, it is to be noted that they apply only when the axial force is small and adjacent unbraced segments are approximately equal in length. For a single member, or segments which do not fall into category a, b, c, or d, the recommended value of B is unity. The value of B should also be taken as unity when computing the value of $F_{b\gamma}$ to obtain M_n to be used in Equations 8.1-1 and 3.1-1, since the effect of moment gradient is provided for by the factor C_m. The background material is given in WRC Bulletin No. 192 (Morrell and Lee, 1974).

SECTION C6.4
BEAMS AND GIRDERS WITH WEB OPENINGS

Web openings in structural floor members may be necessary to accommodate various mechanical, electrical, and other systems. Strength limit states, including local buckling of the compression flange, web, and tee-shaped compression zone above or below the opening, lateral buckling and moment-shear interaction, or serviceability may control the design of a flexural member with web openings. The location, size, and number of openings are important and empirical limits for them have been identified. One general procedure for assessing these effects and the design of any needed reinforcement for both steel and composite beams is given in Darwin (1990) and in ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (1992 and 1992a).

CHAPTER 7
PLATE GIRDERS

SECTION C7.2
DESIGN FLEXURAL STRENGTH

In previous versions of the AISC Specification a coefficient of $0.0005a_r$ was used in R_{PG} based on the work of Basler (1961). This value is valid for $a_r \leq 2$. In that same paper, Basler developed a more general coefficient, applicable to all ratios of A_w/A_f which has been adopted because application of the previous equation to sections with large a_r values gives unreasonable results. An arbitrary limit of $a_r \leq 10$ is imposed so that the R_{PG} expression is not applied to sections approaching a tee shape.

CHAPTER 8
MEMBERS UNDER COMBINED FORCES AND TORSION

SECTION C8.1
SYMMETRIC MEMBERS SUBJECT TO BENDING AND AXIAL FORCE

Equations 8.1-1a and 8.1-1b are simplifications and clarifications of similar equations used in the AISC ASD Specification since 1961. Previously, both equations had to be checked. In the new formulation the applicable equation is governed by the value of the first term, $P_u / \phi P_n$. For bending about one axis only, the equations have the form shown in Figure C8.1-1.

The first term $P_u/\phi P_n$ has the same significance as the axial load term f_a / F_a in Equations 8.1-1 of the SBC 306. This means that for members in compression P_n must be based on the largest effective slenderness ratio Kl / r. In the development of Equations 8.1-1a and 8.1-1b, a number of alternative formulations were compared to the exact inelastic solutions of 82 side-sway cases reported in Kanchanalai (1977). In particular, the possibility of using Kl/r as the actual column length *(K = 1)* in determining P_n, combined with an elastic second order moment Mu, was studied. In those cases where the true P_n based on Kl/r, with $K = 1.0$, was in the inelastic range, the errors proved to be unacceptably large without the additional check that $P_u \le \phi_c P_n$, P_n being based on effective length. Although deviations from exact solutions were reduced, they still remained high.

In summary, it is not possible to formulate a safe general interaction equation for compression without considering effective length directly (or indirectly by a

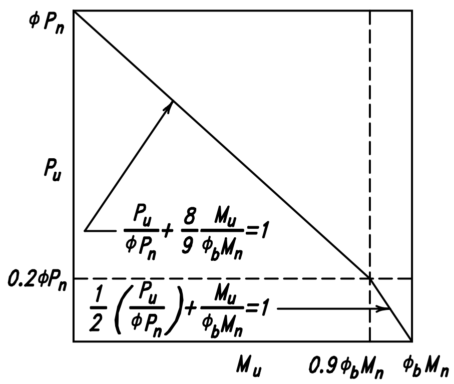


Figure. C8.1-1. Beam-column interaction equations

second equation). Therefore, the requirement that the nominal compressive strength P_n be based on the effective length KL in the general equation is continued in the LRFD Specification as it has been in the AISC ASD Specification since 1961. It is not intended that these provisions be applicable to limit nonlinear secondary flexure that might be encountered in large amplitude earthquake stability design (ATC, 1978).

The defined term M_u is the maximum moment in a member. In the calculation of this moment, inclusion of beneficial second order effects of tension is optional.

But consideration of detrimental second order effects of axial compression and translation of gravity loads is required. Provisions for calculation of these effects are given in Chapter 3.

The interaction equations in Section 8.3 have been recommended for bi-axially loaded H and wide flange shapes in Galambos (1998) and Springfield (1975). These equations which can be used only in braced frames represent a considerable liberalization over the provisions given in Section 8.1; it is, therefore, also necessary to check yielding under service loads, using the appropriate load and resistance factors for the serviceability limit state in Equation 8.1-1a or 8.1-1b with $M_{ux} = S_x F_y$ and $M_{uy} = S_y F_y$. Section 8.3 also provides interaction equations for rectangular box-shaped beam-columns. These equations are taken from Zhou and Chen (1985).

SECTION C8.2 UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

This section deals with types of cross sections and loadings not covered in Section 8.1, especially where torsion is a consideration. For such cases it is recommended to perform an elastic analysis based on the theoretical numerical methods available from the literature for the determination of the maximum normal and shear stresses, or for the elastic buckling stresses. In the buckling calculations an equivalent slenderness parameter is determined for use in Equation 5.2-2 or 5.2-3, as follows:

$$\lambda_e = \sqrt{F_y / F_e}$$

where F_e is the elastic buckling stress determined from a stability analysis. This procedure is similar to that of Section 5.3.

For the analysis of members with open sections under torsion refer to AISC (1997).

CHAPTER 9
COMPOSITE MEMBERS

SECTION C9.1
DESIGN ASSUMPTIONS AND DEFINITIONS

Force Determination. Loads applied to an unshored beam before the concrete has hardened are resisted by the steel section alone, and only loads applied after the concrete has hardened are considered as resisted by the composite section. It is usually assumed for design purposes that concrete has hardened when it attains 75 percent of its design strength. In beams properly shored during construction, all loads may be assumed as resisted by the composite cross section. Loads applied to a continuous composite beam with shear connectors throughout its length, after the slab is cracked in the negative moment region, are resisted in that region by the steel section and by properly anchored longitudinal slab reinforcement.

For purposes of plastic analysis all loads are considered resisted by the composite cross section, since a fully plastic strength is reached only after considerable yielding at the locations of plastic hinges.

Elastic Analysis. The use of constant stiffness in elastic analyses of continuous beams is analogous to the practice in reinforced concrete design. The stiffness calculated using a weighted average of moments of inertia in the positive moment region and negative moment regions may take the following form:

$$I_t = aI_{pos} + bI_{neg}$$

where

I_{pos} = effective moment of inertia for positive moment, mm^4

I_{neg} = negative moment of inertia for negative moment, mm^4

The effective moment of inertia shall be based on the cracked transformed section considering degree of composite actions. For continuous beams subjected to gravity loads only, the value of a may be taken as 0.6 and the value of b may be taken as 0.4. For the case of composite beams in moment resisting frames, the value of *a* and *b* may be taken as 0.5.

Plastic Analysis. For composite beams with shear connectors, plastic analysis may be used only when the steel section in the positive moment region has a compact web, i.e., $h/t_w \leq 3.76\sqrt{E/F_{yf}}$, and when the steel section in the negative moment region is compact, as required for steel beams alone. No compactness limitations are placed on encased beams, but plastic analysis is permitted only if the direct contribution of concrete to the strength of sections is neglected; the concrete is relied upon only to prevent buckling.

Plastic Stress Distribution for Positive Moment. Plastic stress distributions are described in Commentary Section C9.3, and a discussion of the composite participation of slab reinforcement is presented.

Plastic Stress Distribution for Negative Moment. Plastic stress distributions are described in Commentary Section C9.3.

Elastic Stress Distribution. The strain distribution at any cross section of a composite beam is related to slip between the structural steel and concrete elements. Prior to slip, strain in both steel and concrete is proportional to the distance from the neutral axis for the elastic transformed section. After slip, the strain distribution is discontinuous, with a jump at the top of the steel shape. The strains in steel and concrete are proportional to distances from separate neutral axes, one for steel and the other for concrete.

Partially Composite Beam.

Fully Composite Beam. Either the tensile yield strength of the steel section or the compressive stress of the concrete slab governs the maximum flexural strength of a fully composite beam subjected to a positive moment. The tensile yield strength of the longitudinal reinforcing bars in the slab governs the maximum flexural strength of a fully composite beam subjected to a negative moment. When shear connectors are provided in sufficient numbers to fully develop this maximum flexural strength, any slip that occurs prior to yielding is minor and has negligible influence both on stresses and stiffness.

Concrete-Encased Beam. When the dimensions of a concrete slab supported on steel beams are such that the slab can effectively serve as the flange of a composite T-beam, and the concrete and steel are adequately tied together so as to act as a unit, the beam can be proportioned on the assumption of composite action.

Two cases are recognized: fully encased steel beams, which depend upon natural bond for interaction with the concrete, and those with mechanical anchorage to the slab (shear connectors), which do not have to be encased.

SECTION C9.2
COMPRESSION MEMBERS

C9.2.1 Limitations.

(1) The lower limit of four percent on the cross-sectional area of structural steel differentiates between composite and reinforced concrete columns. If the area is less than four percent, a column with a structural steel core should be designed as a reinforced concrete column.

(2) The specified minimum quantity of transverse and longitudinal reinforcement in the encasement should be adequate to prevent severe spalling of the surface concrete during fires.

(3) Very little of the supporting test data involved concrete strengths in excess of 41 MPa, even though the cylinder strength for one group of four columns was 66 MPa. Normal weight concrete is believed to have been used in all tests. Thus, the upper limit of concrete strength is specified as 55 MPa for normal weight concrete. A lower limit of 21 MPa is specified for normal weight concrete and 28 MPa for lightweight concrete to encourage the use of good quality, yet readily available, grades of structural concrete.

(4) In addition to the work of Bridge and Roderick (1978), SSRC Task Group 20 (1979), and Galambos and Chapuis (1980), recent work by Kenny, Bruce, and Bjorhovde (1994) has shown that due to concrete confinement effects, the previous limitation of 380 MPa for the maximum steel yield stress is highly restrictive. Further, the most commonly used reinforcing steel grade has a yield stress of 415 MPa. The increase is therefore a rational recognition of material properties and structural behavior.

The 415 MPa limitations for the yield stress is very conservative for tubular composite columns, where the concrete confinement provided by the tube walls is very significant. Kenny et al. have proposed raising the value of F_y for such columns to whatever the yield stress is for the steel grade used, but not higher than 550 MPa.

(5) The specified minimum wall thicknesses are identical to those in the SBC-304. The purpose of this provision is to prevent buckling of the steel pipe or HSS before yielding.

SECTION C9.3
FLEXURAL MEMBERS

C9.3.2 **Design Strength of Beams with Shear Connectors.** This section applies to simple and continuous composite beams with shear connectors, constructed with or without temporary shores.

Positive Flexural Design Strength. Flexural strength of a composite beam in the positive moment region may be limited by the plastic strength of the steel section, the concrete slab, or shear connectors. In addition, web buckling may limit flexural strength if the web is slender and a significantly large portion of the web is in compression.

According to Table 2.5-1, local web buckling does not reduce the plastic strength of a bare steel beam if the beam depth-to-web thickness ratio is not larger than $3.76\sqrt{E/F_y}$. In the absence of web buckling research on composite beams, the same ratio is conservatively applied to composite beams. Furthermore, for more slender webs, the SBC 306 conservatively adopts first yield as the flexural strength limit. In this case, stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section.

When first yield is the flexural strength limit, the elastic transformed section is used to calculate stresses on the composite section. The modular ratio $n = E/E_c$ used to determine the transformed section depends on the specified unit weight and strength of concrete.

Plastic Stress Distribution for Plastic Moment. When flexural strength is determined from the plastic stress distribution shown in Figure C9.3-1, the compression force C in the concrete slab is the smallest of:

$$C = A_{sw} F_{yw} + 2A_{sf} F_{yf} \quad \text{(C9.3-1)}$$

$$C = 0.85 f_c' A_c \quad \text{(C9.3-2)}$$

$$C = \Sigma Q_n \quad \text{(C9.3-3)}$$

For a non-hybrid steel section, Equation C9.3-1 becomes $C = A_s F_y$

where

f_c' = specified compressive strength of concrete, MPa

A_c = area of concrete slab within effective width, mm^2

A_s = area of steel cross section, mm^2

A_{sw} = area of steel web, mm^2

A_{sf} = area of steel flange, mm^2

F_y = minimum specified yield stress of steel, MPa

F_{yw} = minimum specified yield stress of web steel, MPa

F_{yf} = minimum specified yield stress of flange steel, MPa

ΣQ_n = sum of nominal strengths of shear connectors between the point of maximum positive moment and point of zero moment to either side, N

Longitudinal slab reinforcement makes a negligible contribution to the compression force, except when Equation C9.3-2 governs. In this case, the area of longitudinal reinforcement within the effective width of the concrete slab times the yield stress of the reinforcement may be added in determining C.

The depth of the compression block is

$$a = \frac{C}{0.85 f_c' b} \quad \text{(C9.3-4)}$$

where

b = effective width of concrete slab, mm

A fully composite beam corresponds to the case of C governed by the yield strength of the steel beam or the compressive strength of the concrete slab, as in Equation C9.3-1 or C9.3-2. The number and strength of shear connectors govern C for a partially composite beam as in Equation C9.3-3.

The plastic stress distribution may have the plastic neutral axis (PNA) in the web, in the top flange of the steel section or in the slab, depending on the value of C.

The nominal plastic moment resistance of a composite section in positive bending is given by the following equation and Figure C9.3-1:

$$M_n = C(d_1 + d_2) + P_y(d_3 - d_2) \quad \text{(C9.3-5)}$$

where

P_y = tensile strength of the steel section; for a non-hybrid steel section, $P_y = A_sF_y$, N.

d_1 = distance from the centroid of the compression force C in concrete to the top of the steel section, mm.

d_2 = distance from the centroid of the compression force in the steel section to the top of the steel section, mm. For the case of no compression in the steel section $d_2 = 0$.

d_3 = distance from P_y to the top of the steel section, mm.

Equation C9.3-5 is generally applicable including both non-hybrid and hybrid steel sections symmetrical about one or two axes.

Negative Flexural Design Strength. The flexural strength in the negative moment region is the strength of the steel beam alone or the plastic strength of the composite section made up of the longitudinal slab reinforcement and the steel section.

Plastic Stress Distribution for Negative Moment. When an adequately braced compact steel section and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from the plastic stress distributions as shown in Figure C9.3-2. The tensile force T in the reinforcing bars is the smaller of:

$$T = A_r F_{yr} \tag{C9.3-6}$$

$$T = \Sigma Q_n \tag{C9.3-7}$$

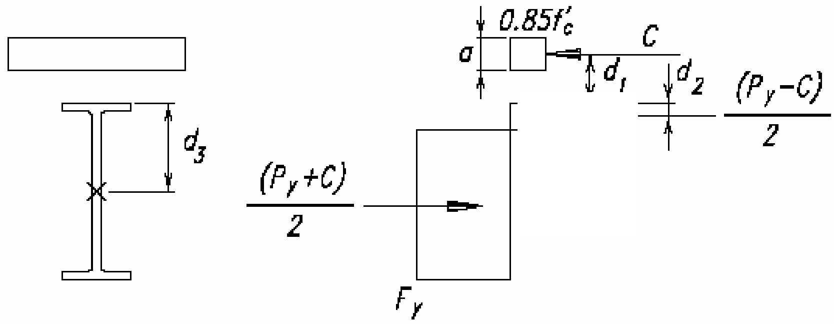


Figure. C9.3-1. Plastic stress distribution for positive moment in composite beams.

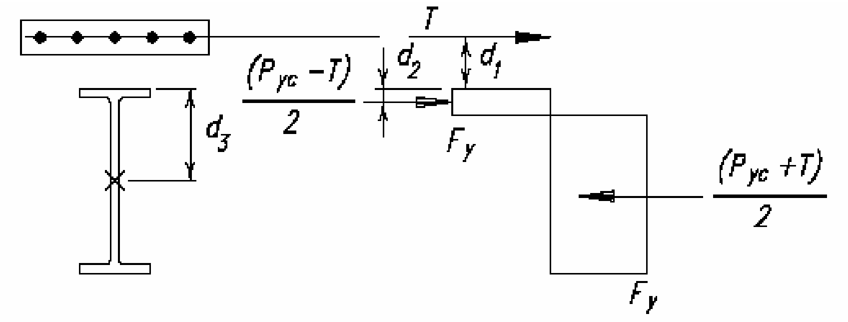


Figure. C9.3-2. Plastic stress distribution for negative moment.

where

A_r = area of properly developed slab reinforcement parallel to the steel beam and within the effective width of the slab, mm^2

F_{yr} = specified yield stress of the slab reinforcement, MPa

ΣQ_n = sum of the nominal strengths of shear connectors between the point of maximum negative moment and point of zero moment to either side, N

A third theoretical limit on T is the product of the area and yield stress of the steel section. However, this limit is redundant in view of practical limitations on slab reinforcement.

The nominal plastic moment resistance of a composite section in negative bending is given by the following equation:

$$M_n = T(d_1 + d_2) + P_{yc}(d_3 - d_2) \quad \text{(C9.3-8)}$$

where

P_{yc} = the compressive strength of the steel section; for a non-hybrid section, $P_{yc} = A_s F_y$, N

d_1 = distance from the centroid of the longitudinal slab reinforcement to the top of the steel section, mm

d_2 = distance from the centroid of the tension force in the steel section to the top of the steel section, mm

d_3 = distance from P_{yc} to the top of the steel section, mm

Transverse Reinforcement for the Slab. Where experience has shown that longitudinal cracking detrimental to serviceability is likely to occur, the slab should be reinforced in the direction transverse to the supporting steel section. It is recommended that the area of such reinforcement should be at least 0.002 times the concrete area in the longitudinal direction of the beam and should be uniformly distributed.

C9.3.3 **Design Strength of Concrete-Encased Beams.** Tests of concrete-encased beams demonstrated that (1) the encasement drastically reduces the possibility of lateral-torsional instability and prevents local buckling of the encased steel, (2) the restrictions imposed on the encasement practically prevent bond failure prior to first yielding of the steel section, and (3) bond failure does not necessarily limit the moment capacity of an encased steel beam (ASCE, 1979).

Accordingly, the SBC 306 permits three alternative design methods: one based on the first yield in the tension flange of the composite section; one based on the plastic moment capacity of the steel beam alone; and a third method based upon the plastic moment capacity of the composite section applicable only when shear connectors are provided along the steel section and reinforcement of the concrete encasement meets the specified detailing requirements. No limitations are placed on the slenderness of either the composite beam or the elements of the steel section, since the encasement effectively inhibits both local and lateral buckling.

In the method based on first yield, stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of the concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section. Complete interaction (no slip) between the concrete and steel is assumed.

The contribution of concrete to the strength of the composite section is ordinarily larger in positive moment regions than in negative moment regions. Accordingly, design based on the composite section is more advantageous in the regions of positive moments.

C9.3.4 **Strength During Construction.** When temporary shores are not used during construction, the steel beam alone must resist all loads applied before the concrete has hardened enough to provide composite action. Unshored beam deflection caused by wet concrete tends to increase slab thickness and dead load. For longer spans this may lead to instability analogous to roof ponding. An excessive increase of slab thickness may be avoided by beam camber.

When forms are not attached to the top flange, lateral bracing of the steel beam during construction may not be continuous and the un-braced length may control flexural strength, as defined in Section 6.1.

The SBC 306 does not include special requirements for a margin against yield during construction. According to Section 6.1, maximum factored moment during construction is $0.90F_yZ$ where F_yZ is the plastic moment $0.90F_yZ \approx 0.90 \times 1.1F_yS$. This is equivalent to approximately the yield moment, F_yS. Hence, required flexural strength during construction prevents moment in excess of the yield moment.

Load factors for construction loads should be determined for individual projects according to local conditions, using the factors stipulated in SBC 301 as a guide. As a minimum it is suggested that 1.2 be the factor for the loading from steel framing plus concrete plus formed steel deck, and a factor of 1.6 be used for the live load of workmen plus equipment which should not be taken as less than 950 N/m^2 (unfactored).

C9.3.5 **Formed Steel Deck.** Figure C9.3-3 is a graphic presentation of the terminology used in Section 9.3.5.

When studs are used on beams with formed steel deck, they may be welded directly through the deck or through pre-punched or cut-in-place holes in the deck. The usual procedure is to install studs by welding directly through the deck; however, when the deck thickness is greater than 16 gage (1.5 mm) for single thickness, or 18 (1.2 mm) gage for each sheet of double thickness, or when the total thickness of galvanized coating is greater than 0.38 kg/m^2, special precautions and procedures recommended by the stud manufacturer should be followed.

As shown in Figure C9.3-4, modern steel deck profiles with stiffeners (reinforcing rib) located along the centerline of the rib require that studs be placed off-center in the rib. Depending on the location of the stud relative to the direction of the shear transfer, for studs in the “weak position”, the resulting reduction in edge distance between the stud and rib wall can lead to premature failure accompanied by punching of the stud through the steel deck. Therefore, in addition to applying the required cap of 0.75 on the reduction factor (Equation 9.3-1) for single studs in a rib, it is recommended to avoid situations where all studs may be located in the “weak position” by either alternating stud placement between the “weak” and “strong” positions or coordinating placement of studs to ensure they are all installed in the strong position.

Based on the Lehigh test data (Grant et al., 1977), the maximum spacing of steel deck anchorage to resist uplift was increased from 405 mm to 460 mm in order to accommodate current production profiles.

When metal deck includes units for carrying electrical wiring, crossover headers are commonly installed over the cellular deck perpendicular to the ribs. They create trenches which completely or partially replace sections of the concrete slab above the deck. These trenches, running parallel to or transverse to a composite beam, may reduce the effectiveness of the concrete flange. Without special provisions to replace the concrete displaced by the trench, the trench should be considered as a complete structural discontinuity in the concrete flange.

When trenches are parallel to the composite beam, the effective flange width should be determined from the known position of the trench.

Trenches oriented transverse to composite beams should, if possible, be located in areas of low bending moment and the full required number of studs should be placed between the trench and the point of maximum positive moment. Where the trench cannot be located in an area of low moment, the beam should be designed as non-composite.

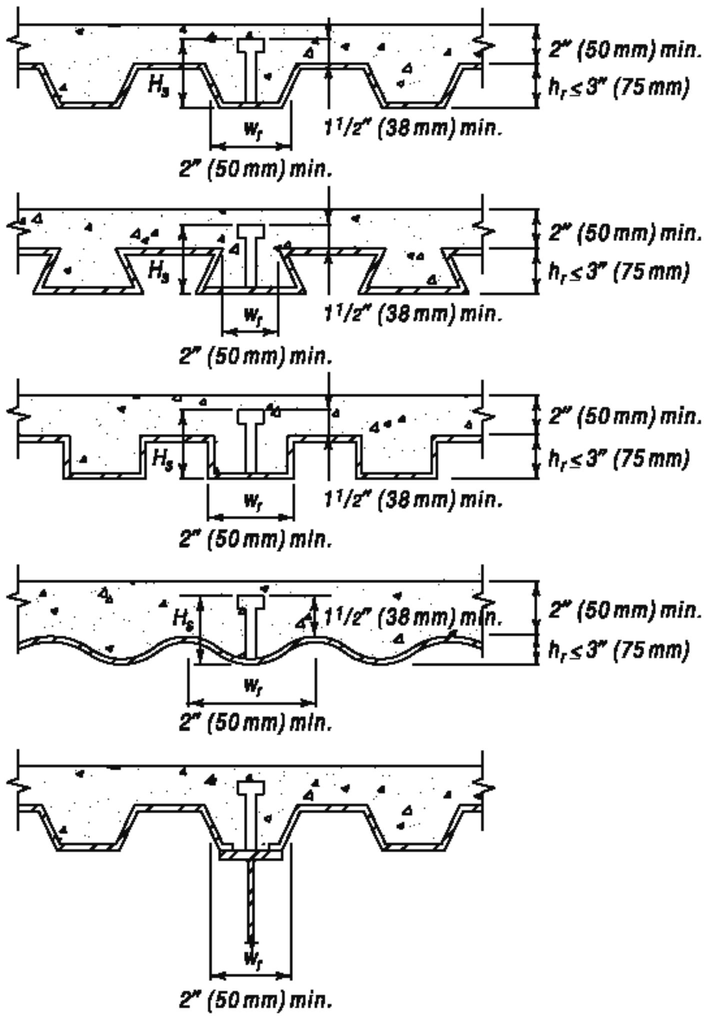


Figure. C9.3-3 Steel deck limits.

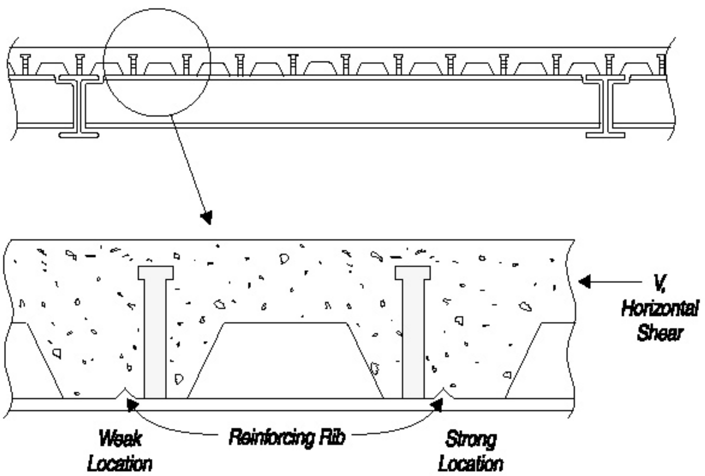


Figure. C9.3-4 Alternative shear stud positions in rib decked profiles.

SECTION C9.4
COMBINED COMPRESSION AND FLEXURE

The last paragraph in Section 9.4 provides a transition from beam-columns to beams. It involves bond between the steel section and concrete. Section 9.3 for beams requires either shear connectors or full, properly reinforced encasement of the steel section. Furthermore, even with full encasement, it is assumed that bond is capable of developing only the moment at first yielding in the steel of the composite section. No test data are available on the loss of bond in composite beam-columns. However, consideration of tensile cracking of concrete suggests $P_u/\phi_c P_n = 0.3$ as a conservative limit. It is assumed that when $P_u/\phi_c P_n$ is less than 0.3, the nominal flexural strength is reduced below that indicated by plastic stress distribution on the composite cross section unless the transfer of shear from the concrete to the steel is provided for by shear connectors.

SECTION C9.5
SHEAR CONNECTORS

C9.5.2 **Horizontal Shear Force.** Composite beams in which the longitudinal spacing of shear connectors was varied according to the intensity of the static shear, and duplicate beams in which the connectors were uniformly spaced, exhibited the same ultimate strength and the same amount of deflection at normal working loads. Only a slight deformation in the concrete and the more heavily stressed connectors is needed to redistribute the horizontal shear to other less heavily stressed connectors. The important consideration is that the total number of connectors be sufficient to develop the shear V_h on either side of the point of maximum moment. The provisions of the SBC 306 are based upon this concept of composite action.

In computing the design flexural strength at points of maximum negative bending, reinforcement parallel to the steel beam within the effective width of the slab may be included, provided such reinforcement is properly anchored beyond the region of negative moment. However, enough shear connectors are required to transfer the ultimate tensile force in the reinforcement, from the slab to the steel beam.

C9.5.3 **Strength of Stud Shear Connectors.** The SBC 306 does not specify a resistance factor for shear connector strength. The resistance factor for the flexural strength of a composite beam accounts for all sources of variability, including those associated with the shear connectors.

C9.5.6 **Shear Connector Placement and Spacing.** Uniform spacing of shear connectors is permitted except in the presence of heavy concentrated loads.

Studs not located directly over the web of a beam tend to tear out of a thin flange before attaining full shear-resisting capacity. To guard against this contingency, the size of a stud not located over the beam web is limited to 2½ times the flange thickness (Goble, 1968).

The minimum spacing of connectors along the length of the beam, in both flat soffit concrete slabs and in formed steel deck with ribs parallel to the beam, is six diameters; this spacing reflects development of shear planes in the concrete slab (Ollgaard et al., 1971). Since most test data are based on the minimum transverse spacing of four diameters, this transverse spacing was set as the minimum permitted. If the steel beam flange is narrow, this spacing requirement may be achieved by staggering the studs with a minimum transverse spacing of three diameters between the staggered rows of studs. The reduction in connector capacity in the ribs of formed steel decks is provided by the factor $0.85/\sqrt{N_r}$, which accounts for the reduced capacity of multiple connectors, including the effect of spacing. When deck ribs are parallel to the beam and the design requires more studs than can be placed in the rib, the deck may be split so that adequate spacing is available for stud installation. Figure C9.5-1 shows possible connector arrangements.

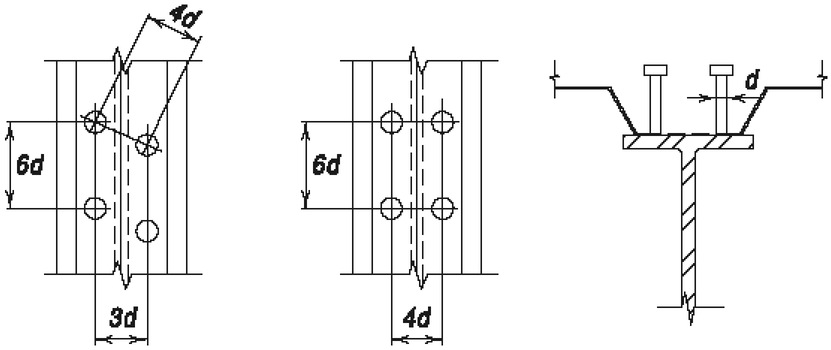


Figure. C9.5-1 Shear connector arrangements.

SECTION C9.6
SPECIAL CASES

Tests are required for construction that falls outside the limits given in the Specification. Different types of shear connectors may require different spacing and other detailing than stud and channel connectors.

CHAPTER 10
CONNECTIONS, JOINTS AND FASTENERS

SECTION C10.1
GENERAL PROVISIONS

C10.1.5 Splices in Heavy Sections. Solidified but still-hot filler metal contracts significantly as it cools to ambient temperature. Shrinkage of large welds between elements which are not free to move to accommodate the shrinkage causes strains in the material adjacent to the weld that can exceed the yield point strain. In thick material the weld shrinkage is restrained in the thickness direction, as well as in the width and length directions, causing triaxial stresses to develop that may inhibit the ability of ductile steel to deform in a ductile manner. Under these conditions, the possibility of brittle fracture increases.

When splicing ASTM A6/A6M Group 4 and 5 and equivalent rolled sections or heavy welded built-up members, the potentially harmful weld shrinkage strains can be avoided by using bolted splices or fillet-welded lap splices or splices that combine a welded and bolted detail (see Figure C10.1-1). Details and techniques that perform well for materials of modest thickness usually must be changed or supplemented by more demanding requirements when welding thick material. Also, the provisions of the *Structural Welding Code*, AWS D1.1, are minimum requirements that apply to most structural welding situations; however, when designing and fabricating welded splices of ASTM A6/A6M Group 4 and 5 and equivalent shapes and similar built-up cross sections, special consideration must be given to all aspects of the welded splice detail.

- Notch-toughness requirements should be specified for tension members. See Commentary Section C1.3.
- Generously sized weld access holes, Figure C10.1-2, are required to provide increased relief from concentrated weld shrinkage strains, to avoid close juncture of welds in orthogonal directions, and to provide adequate clearance for the exercise of high quality workmanship in hole preparation, welding, and ease of inspection.
- Preheating for thermal cutting is required to minimize the formation of a hard surface layer.
- Grinding to bright metal and inspection using magnetic particle or dye-penetrant methods is required to remove the hard surface layer and to assure smooth transitions free of notches or cracks.

In addition to tension splices of truss chord members and tension flanges of flexural members, other joints fabricated of heavy sections subject to tension should be given special consideration during design and fabrication.

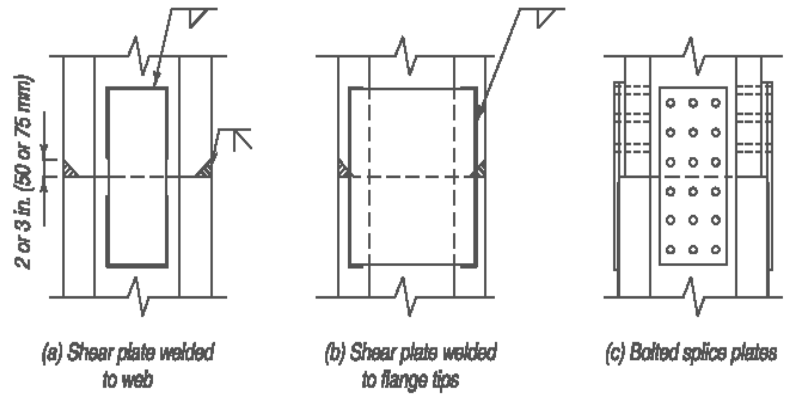
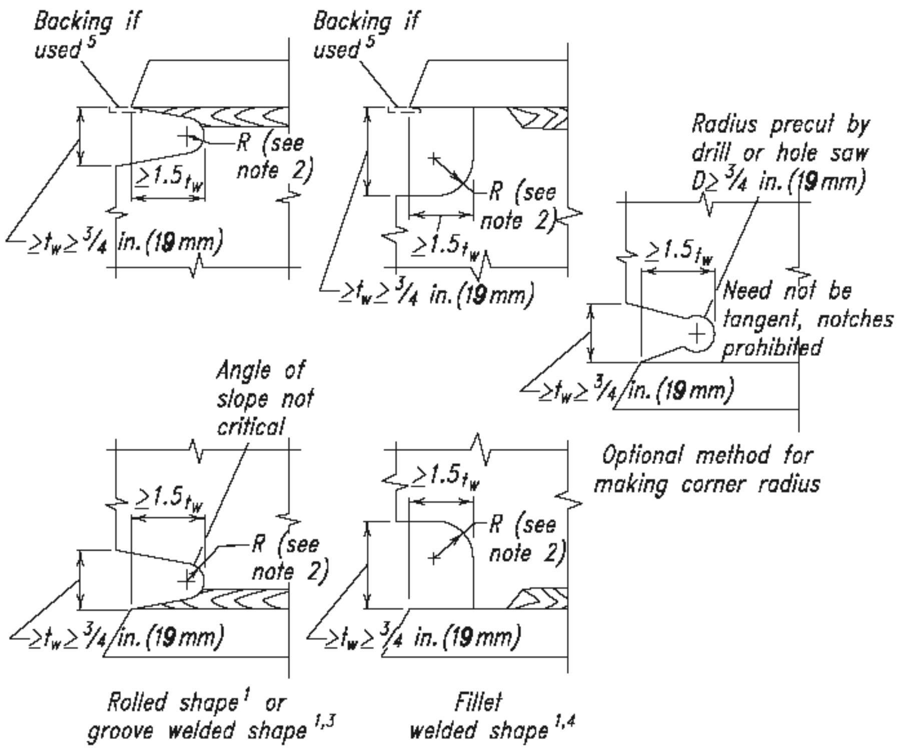


Figure. C10.1-1. Alternative splices that minimize weld restraint tensile stresses.



Notes:

1. *For ASTM A6 Group 4 and 5 and equivalent shapes and welded built-up shapes with plate thickness more than 50 mm, preheat to 65° C prior to thermal cutting, grind and inspect thermally cut edges of access hole using magnetic particle or dye penetration methods prior to making web and flange splice groove welds.*
2. *Radius shall provide smooth notch-free transition; R ≥ 10 mm (typical 13 mm).*
3. *Access opening made after welding web to flange.*
4. *Access opening made before welding web to flange.*
5. *These are typical details for joints welded from one side against steel backing. Alternative joint designs should be considered.*

Figure. C10.1-2. Weld access hole geometry

C10.1.8 Placement of Welds and Bolts. The fatigue life of eccentrically loaded welded angles has been shown to be very short (Kloppel and Seeger, 1964). Notches at the roots of fillet welds are harmful when alternating tensile stresses are normal to the axis of the weld, as could occur due to bending when axial cyclic loading is applied to angles with end welds not balanced about the neutral axis. Accordingly, balanced welds are indicated when such members are subjected to cyclic loading (see Figure C10.1-3).

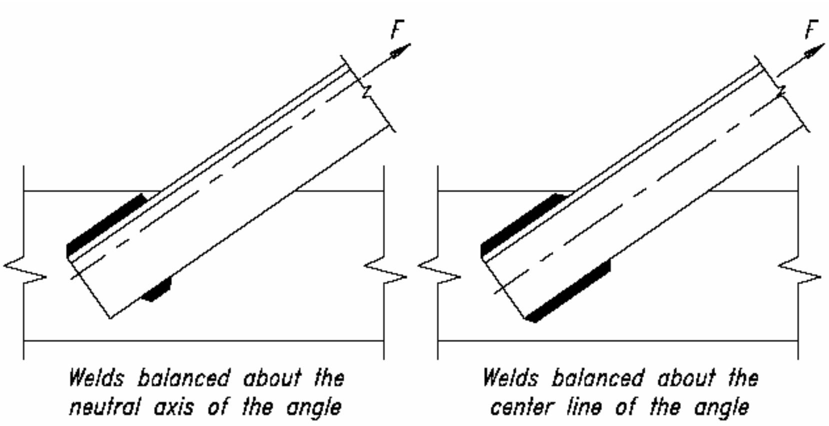


Figure C10.1-3. Balanced welds.

C10.1.9 Bolts in Combination with Welds. The sharing of load between welds and A307 bolts or high-strength bolts in a bearing-type connection is not recommended. For similar reasons, A307 bolts and rivets should not be assumed to share loads in a single group of fasteners.

For high-strength bolts in slip-critical connections to share the load with welds it is advisable to fully tension the bolts before the weld is made. If the weld is placed first, angular distortion from the heat of the weld might prevent the faying action required for development of the slip-critical force. When the bolts are fully tensioned before the weld is made, the slip-critical bolts and the weld may be assumed to share the load on a common shear plane (Kulak, Fisher, and Struik, 1987). The heat of welding near bolts will not alter the mechanical properties of the bolts.

In making alterations to existing structures, it is assumed that whatever slip is likely to occur in high-strength bolted bearing-type connections or riveted connections will have already taken place. Hence, in such cases the use of welding to resist all stresses, other than those produced by existing dead load present at the time of making the alteration, is permitted.

It should be noted that combinations of fasteners as defined herein does not refer to connections such as shear plates for beam-to-column connections which are welded to the column and bolted to the beam flange or web (Kulak et al., 1987) and other comparable connections.

C10.1.10 High-Strength Bolts in Combination with Rivets. When high-strength bolts are used in combination with rivets, the ductility of the rivets permits the direct addition of the strengths of both fastener types.

SECTION C10.2
WELDS

C10.2.1 **Groove Welds.** The engineer preparing contract design drawings cannot specify the depth of groove without knowing the welding process and the position of welding. Accordingly, only the effective throat for partial-joint-penetration groove welds should be specified on design drawings, allowing the fabricator to produce this effective throat with his own choice of welding process and position. The weld reinforcement is not used in determining the effective throat thickness of a groove weld (see Table 10.2-1).

C10.2.2 **Fillet Welds.**

C10.2.2.1 **Effective Area**. The effective throat of a fillet weld is based upon the root of the joint and the face of the diagrammatic weld; hence this definition gives no credit for weld penetration or reinforcement at the weld face. If the fillet weld is made by the submerged arc welding process, some credit for penetration is made. If the leg size of the resulting fillet weld exceeds 10 mm, then 3 mm is added to the theoretical throat. This increased weld throat is allowed because the submerged arc process produces deep penetration of welds of consistent quality. However, it is necessary to run a short length of fillet weld to be assured that this increased penetration is obtained. In practice, this is usually done initially by cross-sectioning the runoff plates of the joint. Once this is done, no further testing is required, as long as the welding procedure is not changed.

C10.2.2.2 **Limitations**. Table 10.2-4 provides a minimum size of fillet weld for a given thickness of the thicker part joined.

The requirements are not based upon strength considerations, but upon the quench effect of thick material on small welds. Very rapid cooling of weld metal may result in a loss of ductility. Further, the restraint to weld metal shrinkage provided by thick material may result in weld cracking. Because 8 mm fillet weld is the largest that can be deposited in a single pass by SMAW process, 8 mm applies to all material 19 mm and greater in thickness, but minimum preheat and inter-pass temperature are required by AWS D1.1 (See Table 10.2-4). Both the design engineer and the shop welder must be governed by the requirements.

Table 10.2-3 gives the minimum effective throat of a partial-joint-penetration groove weld. Notice that Table 10.2-3 for partial-joint-penetration groove welds goes up to a plate thickness of over 150 mm and a minimum weld throat of 16 mm, whereas, for fillet welds Table 10.2-4 goes up to a plate thickness of over 19 mm and a minimum leg size of fillet weld of only 8 mm. The additional thickness for partial-joint-penetration groove welds is to provide for reasonable proportionality between weld and material thickness.

For plates of 6 mm or more in thickness, it is necessary that the inspector be able to identify the edge of the plate to position the weld gage. This is assured if the weld is kept back at least 2 mm from the edge, as shown in Figure C10.2-1.

Where longitudinal fillet welds are used alone in a connection (see Figure C10.2-2), Section 10.2.2.2 requires the length of each weld to be at least equal to the width of the connecting material because of shear lag (Freeman, 1930).

By providing a minimum lap of five times the thickness of the thinner part of a lap joint, the resulting rotation of the joint when pulled will not be excessive, as shown in Figure C10.2-3. Fillet welded lap joints under tension tend to open and apply a tearing action at the root of the weld as shown in Figure C10.2-4(b), unless restrained by a force F as shown in Figure C10.2-4(a).

End returns are not essential for developing the capacity of fillet welded connections and have a negligible effect on their strength. Their use has been encouraged to insure that the weld size is maintained over the length of the weld, to enhance the fatigue resistance of cyclically loaded flexible end connections, and to increase the plastic deformation capability of such connections.

The weld capacity database on which the SBC 306 was developed had no end returns. This includes the study by Higgins and Preece (1968), seat angle tests by Lyse and Schreiner (1935), the seat and top angle tests by Lyse and Gibson (1937), beam webs welded directly to column or girder by fillet welds by Johnston and Deits (1942), and the eccentrically loaded welded connections reported by Butler, Pal, and Kulak (1972). Hence, the current design-resistance values and joint-capacity models do not require end returns when the required weld size is provided. Johnston and Green (1940) noted that movement consistent with the design assumption of no end restraint (i.e., joint flexibility) was enhanced without end returns. They also verified that greater plastic deformation of the connection was achieved when end returns existed, although the strength was not significantly different.

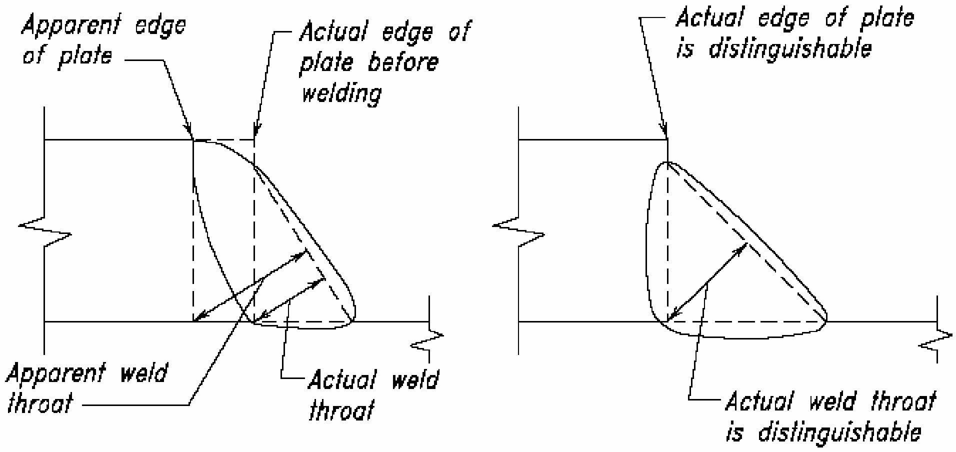


Figure. C10.2-1. Identification of plate edge.

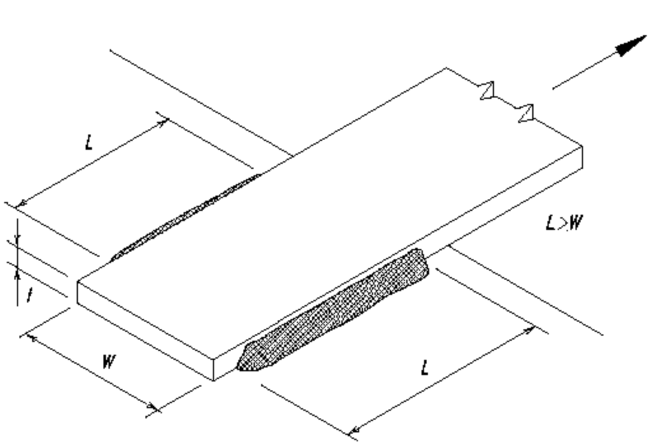


Figure. C10.2-2. Longitudinal fillet welds.

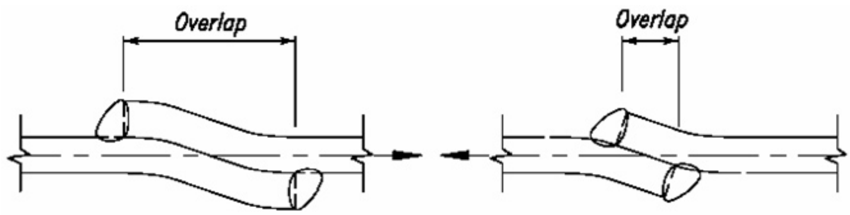


Figure. C10.2-3. Minimum lap.

When longitudinal fillet welds parallel to the stress are used to transmit the load to the end of an axially loaded member, the welds are termed "end loaded". Typical examples of such welds would include, but are not necessarily limited to, longitudinally welded lap joints at the end of axially loaded members, welds attaching bearing stiffeners, and similar cases. Typical examples of longitudinally loaded fillet welds which are not considered end loaded include, but are not limited to, welds that connect plates or shapes to form built-up cross sections in which the shear force is applied to each increment of length of weld stress depending upon the distribution of shear load along the length of the member, welds attaching beam web connection angles and shear plates because the flow of shear force from the beam or girder web to the weld is essentially uniform throughout the weld length, that is, the weld is not end-loaded despite the fact that it is loaded parallel to the weld axis. Neither does the reduction factor apply to welds attaching stiffeners to webs because the stiffeners and welds are not subject to calculated axial stress but merely serve to keep the web flat.

The distribution of stress along the length of end loaded fillet welds is far from uniform and is dependent upon complex relationships between the stiffness of the longitudinal fillet weld relative to the stiffness of the connected materials. Beyond some length, it is non-conservative to assume that the average stress over the total length of the weld may be taken as equal to the full design strength. Experience has shown that when the length of the weld is equal to approximately 100 times the weld size or less, it is reasonable to assume the effective length is equal to the actual length. For weld lengths greater than 100 times the weld size, the effective length should be taken less than the actual length. The reduction coefficient, R, provided in Section 10.2.2.2 is the equivalent of Eurocode 3, which is a simplified approximation to exponential formulas developed by finite element studies and tests preformed in Europe over many years. The criterion is based upon combined consideration of nominal strength for fillet welds with leg size less than 6 mm and upon a judgment based serviceability limit of slightly less than 1 mm displacement at the end of the weld for welds with leg size 6 mm and larger. Mathematically, the application of the β factor implies that the minimum strength of an end-loaded weld is achieved when the length is approximately 300 times the leg size. Because it is illogical to conclude that the total strength of a weld longer than 300 times the weld size would be less than a shorter weld, the length reduction coefficient is taken as 0.6 when the weld length is greater than 300 times the leg size.

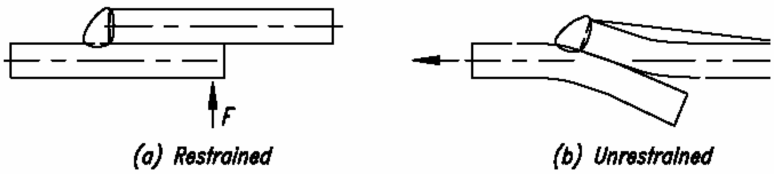


Figure. C10.2-4. Restraint of lap joints.

Fillet weld terminations do not affect the strength or serviceability of connections in most cases. However, in certain cases, the disposition of welds affect the planned function of connections, and notches may affect the static strength and/or the resistance to crack initiation if cyclic loads of sufficient magnitude and frequency occur. For these cases, limitations are specified to assure desired performance.

(a) At lapped joints where one part extends beyond the end or edge of the part to which it is welded and if the parts are subject to calculated tensile stress at the start of the overlap, it is important that the weld terminate a short distance from the stressed edge. For one typical example, the lap joint between the tee chord and the web members of a truss, the weld should not extend to the edge of the tee stem. See Figure C10.2-5. The best technique to avoid inadvertent notches at this critical location is to strike the welding arc at a point slightly back from the edge and proceed with welding in the direction away from the edge. See Figure C10.2-6. On the other hand, where framing angles extend beyond the end of the beam web to which they are welded, the free end of the beam web is subject to zero stress; thus, it is permissible for the fillet weld to extend continuously across the top end, along the side and along the bottom end of the angle to the extreme end of the beam. See Figure C10.2-7.

(b) For connections which are subject to maximum stress at the weld termination due to cyclic forces and/or moments of sufficient magnitude and frequency to initiate cracks emanating from unfilled start or stop craters or other discontinuities, at the end of the weld must be protected by boxing or returns. If the bracket is a plate projecting from the face of a support, extra care must be exercised in the deposition of the boxing weld across the thickness of the plate to assure that a fillet free of notches is provided.

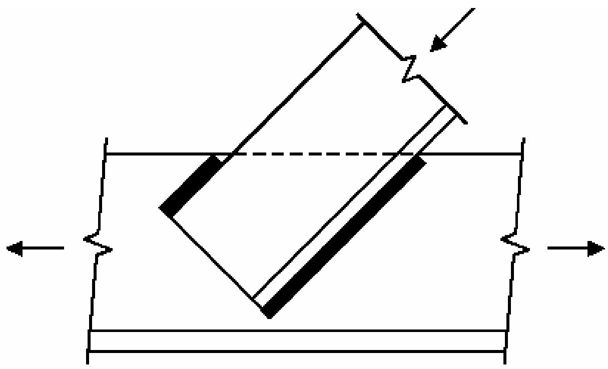


Figure. C10.2-5. Fillet welds near tension edges

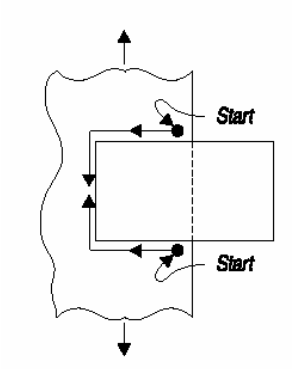


Figure. C10.2-6. Suggested direction of welding travel to avoid notches.

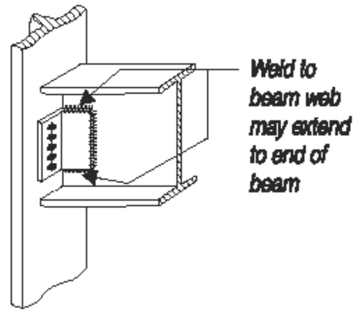


Figure. C10.2-7. Fillet weld details on framing angles.

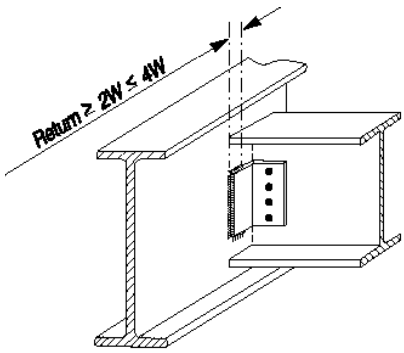


Figure. C10.2-8. Flexible connection returns optional unless subject to fatigue.

(c) For connections such as framing angles and simple end plates which are assumed in design of the structure to be flexible connections, the top and bottom edges of the outstanding legs must be left unwelded over a substantial portion of their length in order to assure flexibility of the connection. Research tests (Johnston and Green, 1940) have shown that the static strength of the connection is the same with or without end returns; therefore the use of returns is optional, but if used, their length must be restricted to not more than four times the weld size, see Figure C10.2-8.

(d) Experience has shown that when ends of intermediate transverse stiffeners on the webs of plate girders are not welded to the flanges (the usual practice), small torsional distortions of the flange which occur near shipping bearing points in the normal course of shipping by rail or truck may cause high out-of-plane bending stresses (yield point) and fatigue cracking at the toe of the web-to-flange welds. This has been observed even with closely fitted stiffeners. The intensity of these out-of-plane stresses may be effectively limited and cracking prevented if "breathing room" is provided by terminating web-to-flange welds. The unwelded distance should not exceed six times the web thickness to assure that column buckling of the web within the unwelded length does not occur.

(e) For fillet welds which occur on opposite sides of a common plane, it is not possible to deposit a weld continuously around the corner from one side to the other without causing a gouge in the corner of the parts joined; therefore the welds must be interrupted at the corner. See Figure C10.2-9.

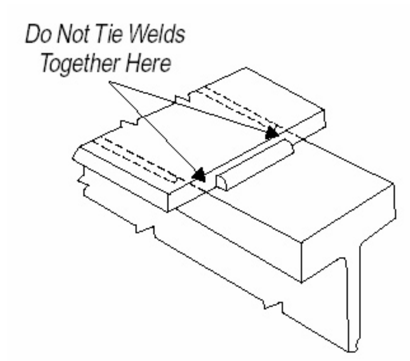


Figure. C10.2-9. Details for fillet welds which occur on opposite sides of a common plane.

C10.2.4 **Design Strength.** The strength of welds is governed by the strength of either the base material or the deposited weld metal. Table 10.2-5 contains the resistance factors and nominal weld strengths, as well as a number of limitations.

It should be noted that in Table 10.2-5 the nominal strength of fillet welds is determined from the effective throat area, whereas the strength of the connected parts is governed by their respective thicknesses. Figure C10.2-10 illustrates the shear planes for fillet welds and base material:

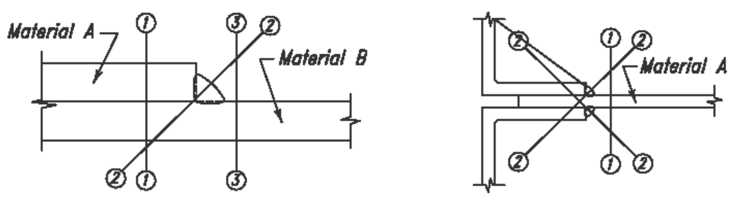


Figure. C10.2-10. Shear planes for fillet welds loaded in longitudinal shear

(a) Plane 1-1, in which the resistance is governed by the shear strength for material A.

(b) Plane 2-2, in which the resistance is governed by the shear strength of the weld metal.

(c) Plane 3-3, in which the resistance is governed by the shear strength of the material B.

The resistance of the welded joint is the lowest of the resistance calculated in each plane of shear transfer. Note that planes 1-1 and 3-3 are positioned away from the fusion areas between the weld and the base material. Tests have demonstrated that the stress on this fusion area is not critical in determining the shear strength of fillet welds (Preece, 1968).

The shear planes for plug and partial-joint-penetration groove welds are shown in Figure C10.2-12 for the weld and base metal. Generally the base metal will govern the shear strength.

When weld groups are loaded in shear by an external load that does not act through the center of gravity of the group, the load is eccentric and will tend to cause a relative rotation and translation between the parts connected by the weld. The point about which rotation tends to take place is called the instantaneous center of rotation. Its location is dependent upon the load eccentricity, geometry of the weld group, and deformation of the weld at different angles of the resultant elemental force relative to the weld axis.

The individual resistance force of each unit weld element can be assumed to act on a line perpendicular to a ray passing through the instantaneous center and that element's location (see Figure C10.2-11).

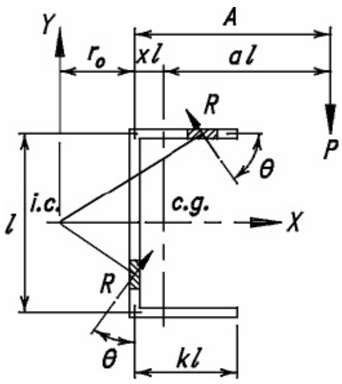


Figure. C10.2-11. Weld element nomenclature.

C10.2.5 **Combination of Welds**. This method of adding weld strengths does not apply to a welded joint using a partial-joint-penetration single bevel groove weld with a superimposed fillet weld. In this case, the effective throat of the combined joint must be determined and the design strength based upon this throat area.

C10.2.6 **Weld Metal Requirements**. Applied and residual stresses and geometrical discontinuities from back-up bars with associated notch effects contribute to sensitivity to fracture. Some weld metals in combination with certain procedures result in welds with low notch toughness. The Specification requires a minimum specified toughness for weld metals in those joints that are subject to more significant applied stresses and toughness demands.

The level of toughness required was selected as one level more conservative than the base metal requirement for Group 4 and 5 and equivalent shapes. Research continues on this subject.

C10.2.7 **Mixed Weld Metal.** Problems can occur when incompatible weld metals are used in combination and notch-tough composite weld metal is required. For instance, tack welds deposited using a self-shielded process with aluminum deoxidizers in the electrodes and subsequently covered by SAW weld passes can result in composite weld metal with low notch-toughness, despite the fact that each process by itself could provide notch-tough weld metal.

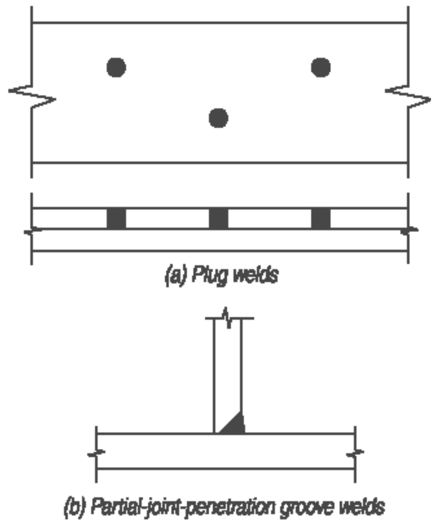


Figure. C10.2-12. Shear planes for plug and partial-joint-penetration groove welds.

SECTION C10.3
BOLTS AND THREADED PARTS

C10.3.1 **High-Strength Bolts.** In general, the use of high-strength bolts is required to conform to the provisions of the *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts* (RCSC, 1994) as approved by the Research Council on Structural Connections.

Occasionally the need arises for the use of high-strength bolts of diameters and lengths in excess of those available for A325 or A325M and A490 or A490M bolts, as for example, anchor rods for fastening machine bases. For this situation Section 1.3.3 permits the use of A449 bolts and A354 threaded rods.

With this edition of the Specification snug-tightened installation is permitted for static applications involving ASTM A325 or A325M bolts (only) in tension or combined shear and tension.

There are practical cases in the design of structures where slip of the connection is desirable in order to allow for expansion and contraction of a joint in a controlled manner. Regardless of whether force transfer is required in the directions normal to the slip direction, the nuts should be hand-tightened with a spud wrench and then backed off one-quarter turn. Furthermore, it is advisable to deform the bolt threads or use a locking nut or jamb nut to insure that the nut does not back off under service conditions. Thread deformation is commonly accomplished with a cold chisel and hammer applied at one location. Note that tack-welding of the nut to the bolt threads is discouraged.

C10.3.2 **Size and Use of Holes.** To provide some latitude for adjustment in plumbing up a frame during erection, three types of enlarged holes are permitted, subject to the approval of the designer. The nominal maximum sizes of these holes are given in Table 10.3-3. The use of these enlarged holes is restricted to connections assembled with bolts and is subject to the provisions of Sections 10.3.3 and 10.3.4.

C10.3.3 **Minimum Spacing.** The maximum factored strength R_n at a bolt or rivet hole in bearing requires that the distance between the centerline of the first fastener and the edge of a plate toward which the force is directed should not be less than $1\frac{1}{2}d$ where d is the fastener diameter (Kulak et al., 1987). By similar reasoning the distance measured in the line of force, from the centerline of any fastener to the nearest edge of an adjacent hole, should not be less than $3d$, to ensure maximum design strength in bearing. Plotting of numerous test results indicates that the critical bearing strength is directly proportional to the above defined distances up to a maximum value of $3d$, above which no additional bearing strength is achieved (Kulak et al., 1987). Table 10.3-6 lists the increments that must be added to adjust the spacing upward to compensate for an increase in hole dimension parallel to the line of force. Section 10.3.10 gives the bearing strength criteria as a function of spacing.

C10.3.4 **Minimum Edge Distance.** Critical bearing stress is a function of the material tensile strength, the spacing of fasteners, and the distance from the edge of the part to the center line of the nearest fastener. Tests have shown (Kulak et al., 1987) that a linear relationship exists between the ratio of critical bearing stress to tensile strength (of the connected material) and the ratio of fastener spacing (in the line of force) to fastener diameter. The following equation affords a good lower bound to published test data for single-fastener connections with standard holes, and is conservative for adequately spaced multi-fastener connections:

$$F_{pcr} / F_u = l_e / d \qquad \text{(C10.3-1)}$$

where

F_{pcr} = critical bearing stress, MPa

F_u = tensile strength of the connected material, MPa

l_e = distance, along a line of transmitted force, from the center of a fastener to the nearest edge of an adjacent fastener or to the free edge of a connected part (in the direction of stress), mm

d = diameter of fastener, mm

C10.3.5 **Maximum Spacing and Edge Distance.** Limiting the edge distance to not more than 12 times the thickness of an outside connected part, but not more than 150 mm, is intended to provide for the exclusion of moisture in the event of paint failure, thus preventing corrosion between the parts which might accumulate and force these parts to separate. More restrictive limitations are required for connected parts of unpainted weathering steel exposed to atmospheric corrosion.

C10.3.6 **Design Tension or Shear Strength.** Tension loading of fasteners is usually accompanied by some bending due to the deformation of the connected parts. Hence, the resistance factor ϕ, by which R_n is multiplied to obtain the design tensile strength of fasteners, is relatively low. The nominal tensile strength values in Table 10.3-2 were obtained from the equation

$$R_n = 0.75\, A_b F_u \tag{C10.3-2}$$

This tensile strength given by Equation C10.3-2 is independent of whether the bolt was initially installed pretensioned or snug-tightened.

In connections consisting of only a few fasteners, the effects of strain on the shear in bearing fasteners is negligible (Kulak et al., 1987 and Fisher et al., 1978). In longer joints, the differential strain produces an uneven distribution between fasteners (those near the end taking a disproportionate part of the total load), so that the maximum strength per fastener is reduced. The ASD-based Specifications permits connections up to 1270 mm in length without a reduction in maximum shear stress. With this in mind the resistance factor ϕ for shear in bearing-type connections has been selected to accommodate the same range of connections.

The values of nominal shear strength in Table 10.3-2 were obtained from the equation

$$R_n / mA_b = 0.50\, F_u \tag{C10.3-3}$$

when threads are excluded from the shear planes and

$$R_n / mA_b = 0.40\, F_u \tag{C10.3-4}$$

when threads are not excluded from the shear plane, where m is the number of shear planes (Kulak et al., 1987). While developed for bolted connections, the equations were also conservatively applied to threaded parts and rivets. The value given for A307 bolts was obtained from Equation C10.3-4 but is specified for all cases regardless of the position of threads. For A325 or A325M bolts, no distinction is made between small and large diameters, even though the minimum tensile strength F_u is lower for bolts with diameters in excess of one inch. It was felt that such a refinement of design was not justified, particularly in view of the low resistance factor ϕ, the increasing ratio of tensile area to gross area, and other compensating factors.

C10.3.7 **Combined Tension and Shear in Bearing-Type Connections.** Tests have shown that the strength of bearing fasteners subject to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse (Kulak et al., 1987). Such a curve can be replaced, with only minor deviations, by three straight lines as shown in Figure C10.3-1. This latter representation offers the advantage that no modification of either type stress is required in the presence of fairly large magnitudes of the other type. This linear representation was adopted for Table 10.3-5, giving a limiting tensile stress F_t as a function of the shearing stress f_v for bearing-type connections. Following a change in the 1994 RCSC *LRFD Specification for Structural Joints Using ASTM A325 or A490 Bolts,* the coefficients in the equations in Table 10.3-5 have been modified for consistency (Carter, Tide, and Yura, 1997).

C10.3.8 **High-Strength Bolts in Slip-Critical Connections.** Connections classified as slip-critical include those cases where slip could theoretically exceed an amount deemed by the Engineer of Record to affect the suitability for service of the structure by excessive distortion or reduction in strength or stability, even though the nominal strength of the connection may be adequate. Also included are those cases where slip of any magnitude must be prevented, for example, joints subject to fatigue, connectors between elements of built-up members at their ends (Sections 4.2 and 5.4), and bolts in combination with welds (Section 10.1.9).

The onset of slipping in a high-strength bolted, slip-critical connection is not an indication that the maximum strength of the connection has been reached. Its occurrence may be only a serviceability limit state. The design check for slip resistance can be made at two different load levels, factored loads (Sections 10.3.8.1 and 10.3.9.1) and service loads (not included here). The nominal slip resistances r_{str} and F_yA_b to be used with factored loads and service loads, respectively, are based on two different design concepts. The slip resistance r_{str} with factored loads is the mean resistance per bolt, which is a function of the mean slip coefficient and the clamping force. The 1.13 factor in (Equation 10.3-1) accounts for the expected 13 percent increase above the minimum specified preload provided by calibrated wrench tightening procedures. This was used to represent typical installations. The factored load resistance r_{str} uses the β reliability index approach that is used for the other design checks such as tension and bearing. The service load approach uses a probability of slip concept that implies a 90 percent reliability that slip will not occur if the calibrated wrench method of bolt installation is used.

The Engineer of Record must make the determination to use factored loads, service loads, or both in checking the slip resistance of a slip-critical connection. The following commentary is provided as guidance and an indication of the intent of the Specification.

In the case of slip-critical connections with three or more bolts in holes with only a small clearance, such as standard holes and slotted holes loaded transversely to the axis of the slot, the freedom to slip does not generally exist because one or more bolts are in bearing even before load is applied due to normal fabrication tolerances and erection procedures. If connections with standard holes have only one or two bolts in the direction of the applied force, a small slip may occur. In this case, slip-critical connections subjected to vibration or wind should be checked for slip at service-load levels. In built-up compression members, such as double-angle struts in trusses, a small slip in the end connections can significantly reduce the strength of the compression member so the slip-critical end connection should be checked for slip at the factored-load level, whether or not a slip-critical connection is required by a serviceability requirement.

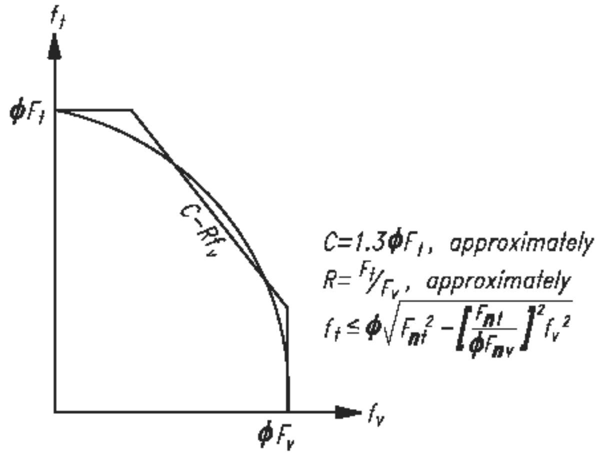


Figure. C10.3-1. Three straight line approximation.

In connections with long slots that are parallel to the direction of the applied load, slip of the connection prior to attainment of the factored load might be large enough to alter the usual assumption of analysis that the undeformed structure can be used to obtain the internal forces. The SBC 306 allows the designer two alternatives in this case. If the connection is designed so that it will not slip under the effects of service loads, then the effect of the factored loads acting on the deformed structure (deformed by the maximum amount of slip in the long slots at all locations) must be included in the structural analysis. Alternatively, the connection can be designed so that it will not slip at loads up to the factored load level.

Joints subjected to full reverse cyclical loading are clearly slip-critical joints since slip would permit back and forth movement of the joint and early fatigue. However, for joints subjected to pulsating load that does not involve reversal of direction, proper fatigue design could be provided either as a slip-critical joint on the basis of stress on the gross section, or as a non-slip-critical joint on the basis of stress on the net section. Because fatigue results from repeated application of the service load rather than the overload load, design should be based upon service-load criteria.

For high-strength bolts in combination with welds in statically loaded conditions and considering new work only, the nominal strength may be taken as the sum of the slip resistances provided by the bolts and the shear resistance of the welds. Section 10.1.9 requires that the slip resistance be determined at factored load levels. If one type of connector is already loaded when the second type of connector is introduced, the nominal strength cannot by obtained by adding the two resistances. The Guide (Kulak et al., 1987) should be consulted in these cases.

Slip of slip-critical connections is likely to occur at approximately 1.4 to 1.5 times the service loads. For standard holes, oversized holes, and short-slotted holes the connection is designed at factored loads (Section 10.3.8.1). The nominal loads and ϕ factors have been adjusted accordingly. The number of connectors will be essentially the same for the two procedures because they have been calibrated to give similar results. Slight differences will occur because of variation in the ratio of live load to dead load.

In connections containing long slots that are parallel to the direction of the applied load, slip of the connection prior to attainment of the factored load might be large enough to alter the usual assumption of analysis that the undeformed structure can be used to obtain the internal forces. To guard against this occurring, the design slip resistance is further reduced by setting ϕ to 0.60 in conjunction with factored loads.

While the possibility of a slip-critical connection slipping into bearing under anticipated service conditions is small, such connections must comply with the provisions of Section 10.3.10 in order to prevent connection failure at the maximum load condition.

C10.3.10 **Bearing Strength at Bolt Holes.** Provisions for bearing strength of pins differ from those for bearing strength of bolts; refer to Section 10.8.

Bearing values are provided as a measure of the strength of the material upon which a bolt bears, not as a protection to the fastener, which needs no such protection. Accordingly, the same bearing value applies to all joints assembled by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

Material bearing strength may be limited either by bearing deformation of the hole or by block shear rupture of the material upon which the bolt bears. Recent testing by Kim and Yura (1996) and Lewis and Zwerneman (1996) has confirmed the bearing strength provisions for the former case wherein the nominal bearing strength R_n is equal to $CdtF_u$ and C is 2.4, 3.0, or 2.0 depending upon hole type and/or acceptability of hole ovalization at ultimate load as indicated in LRFD Specification Section 10.3.10. However, this same research indicated the need for more accurate bearing strength provisions when block-shear-rupture-type failure would control. Appropriate equations for bearing strength as a function of clear distance L_c are therefore provided and this formulation is consistent with that adopted by RCSC in the Load and Resistance Factor Design Specification for *Structural Joints Using ASTM A325 or A490 Bolts* (RCSC, 1994).

Additionally, to simplify and generalize such bearing strength calculations, the current provisions have been based upon a clear-distance formulation. Previous provisions utilized edge distances and bolt spacings measured to hole centerlines with adjustment factors to account for varying hole type and orientation, as well as minimum edge distance requirements.

C10.3.11 **Long Grips.** Provisions requiring a decrease in calculated stress for A307 bolts having long grips (by arbitrarily increasing the required number in proportion to the grip length) are not required for high-strength bolts. Tests (Bendigo, Hansen, and Rumpf, 1963) have demonstrated that the ultimate shearing strength of high-strength bolts having a grip of eight or nine diameters is no less than that of similar bolts with much shorter grips.

SECTION C10.4
DESIGN RUPTURE STRENGTH

Tests (Birkemoe and Gilmor, 1978) on coped beams indicated that a tearing failure mode (rupture) can occur along the perimeter of the bolt holes as shown in Figure C10.4-1. This block shear mode combines tensile strength on one plane and shear strength on a perpendicular plane. The failure path is defined by the center lines of the bolt holes. The block shear failure mode is not limited to the coped ends of beams. Other examples are shown in Figure C10.4-1 and C10.4-2.

The block shear failure mode should also be checked around the periphery of welded connections. Welded connection block shear is determined using $\phi = 0.75$ in conjunction with the area of both the fracture and yielding planes (Yura, 1988).

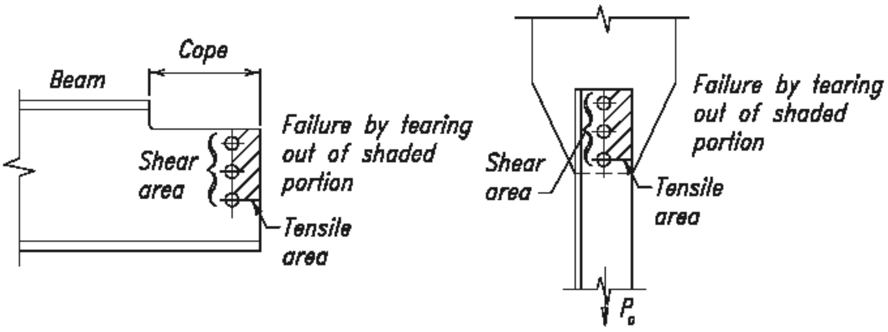


Figure. C10.4-1. Failure for block shear rupture limit state.

The LRFD Specification has adopted a conservative model to predict block shear strength. Test results suggest that it is reasonable to add the yield strength on one plane to the rupture strength of the perpendicular plane (Ricles and Yura, 1983, and Hardash and Bjorhovde, 1985). Therefore, two possible block shear strengths can be calculated; rupture strength F_u on the net tensile section along with shear yielding $0.6\ F_y$ on the gross section on the shear plane(s), or rupture $0.6\ F_u$ on the net shear area(s) combined with yielding F_y on the gross tensile area. This is the basis of Equations 10.4-1 and 10.4-2.

These equations are consistent with the philosophy in Chapter 4 for tension members, where gross area is used for the limit state of yielding and net area is used for rupture. The controlling equation is the one that produces the larger rupture force.

This can be explained by the two extreme examples given in Figure C10.4-2. In Case (a), the total force is resisted primarily by shear, so shear rupture, not shear yielding, should control the block shear tearing mode; therefore, use Equation 10.4-2. For Case (b), block shear cannot occur until the tension area ruptures as given by Equation 10.4-1. If Equation 10.4-2 (shear rupture on the small area and yielding on the large tension area) is checked for Case (b), a smaller P_o will result. In fact, as the shear area gets smaller and approaches zero, the use of Equation 10.4-2 for Case (b) would give a block shear strength based totally on yielding of the gross tensile area. Block shear is a rupture or tearing phenomenon not a yielding limit state. Therefore, the proper equation to use is the one with the larger rupture term.

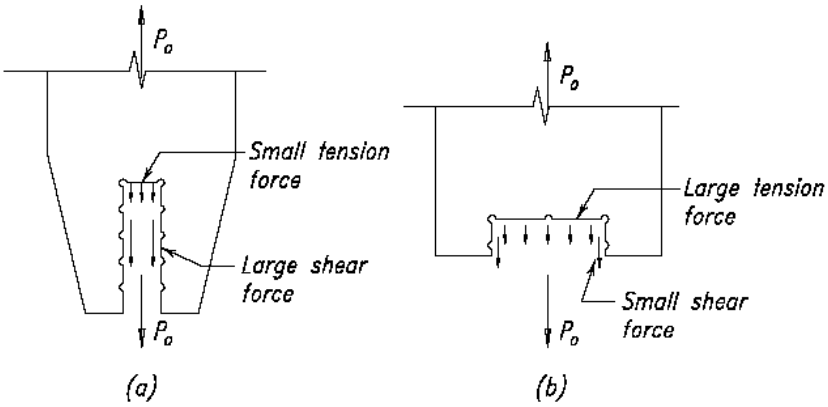


Figure. C10.4-2. Block shear rupture in tension.

SECTION C10.5
CONNECTING ELEMENTS

C10.5.2 **Design Strength of Connecting Elements in Tension.** Tests have shown that yield will occur on the gross section area before the tensile capacity of the net section is reached, if the ratio $A_n / A_g < 0.85$ (Kulak et al., 1987). Since the length of connecting elements is small compared to the member length, inelastic deformation of the gross section is limited. Hence, the effective net area A_n of the connecting element is limited to $0.85A_g$ in recognition of the limited inelastic deformation and to provide a reserve capacity.

SECTION C10.6
FILLERS

The practice of securing fillers by means of additional fasteners, so that they are, in effect, an integral part of a shear-connected component, is not required where a connection is designed to be a slip-critical connection using high-strength bolts. In such connections, the resistance to slip between filler and either connected part is comparable to that which would exist between the connected parts if no fill were present.

Filler plates may be used in lap joints of welded connections that splice parts of different thickness, or where there may be an offset in the joint.

SECTION C10.8
BEARING STRENGTH

The SBC 306 provisions for bearing on milled surfaces, Section 10.8, follow the same philosophy of ASD-based Specifications. In general, the design is governed by a deformation limit state at service loads resulting in stresses nominally at 9/10 of yield. Adequate safety is provided by post-yield strength as deformation increases. Tests on pin connections (Johnston, 1939) and on rockers (Wilson, 1934) have confirmed this behavior.

As used throughout the SBC 306, the terms “milled surface,” “milled,” and “milling” are intended to include surfaces which have been accurately sawed or finished to a true plane by any suitable means.

SECTION C10.9
COLUMN BASES AND BEARING ON CONCRETE

The equations for resistance of concrete in bearing are the same as SBC-304 except that this specification equations use $\phi = 0.60$ while SBC-304 uses $\phi = 0.70$, since SBC-304 specifies larger load factors than the ASCE load factors stipulated by this code requirement.

CHAPTER 11
CONCENTRATED FORCES, PONDING AND FATIGUE

SECTION C11.1
FLANGES AND WEBS WITH CONCENTRATED FORCES

C11.1.1 **Design Basis**. The SBC 306 separates flange and web strength requirements into distinct categories representing different limit state criteria, i.e., flange local bending (Section 11.1.2), web local yielding (Section 11.1.3), web crippling (Section 11.1.4), web sidesway buckling (Section 11.1.5), web compression buckling (Section 11.1.6), and web panel-zone shear (Section 11.1.7).

These criteria are applied to two distinct types of concentrated forces which act on member flanges. Single concentrated forces may be tensile, such as those delivered by tension hangers, or compressive, such as those delivered by bearing plates at beam interior positions, reactions at beam ends, and other bearing connections. Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member, such as that delivered to column flanges through welded and bolted moment connections. See Carter (1999) for guidelines on column stiffener design.

C11.1.2 **Flange Local Bending**. Where a tensile force is applied through a plate welded across a flange, that flange must be sufficiently rigid to prevent deformation of the flange and the corresponding high-stress concentration in the weld in line with the web.

The effective column flange length for local flange bending is $12t_f$ (Graham, et al., 1959). Thus, it is assumed that yield lines form in the flange at $6t_f$ in each direction from the point of the applied concentrated force. To develop the fixed edge consistent with the assumptions of this model, an additional $4t_f$ and therefore a total of $10t_f$, is required for the full flange-bending strength given by Equation 11.1-1. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the applied concentrated force is less than $10t_f$ from the member end.

This criterion given by Equation 11.1-1 was originally developed for moment connections, but it also applies to single concentrated forces such as tension hangers consisting of a plate welded to the bottom flange of a beam and transverse to the beam web.

C11.1.3 **Web Local Yielding**. The web strength criteria have been established to limit the stress in the web of a member into which a force is being transmitted. It should matter little whether the member receiving the force is a beam or a column; however, Galambos (1976) and AISC (1978), references upon which the SBC 306 is based, did make such a distinction. For beams, a 2:1 stress gradient through the flange was used, whereas the gradient through column flanges was 2½:1. In Section 11.1.3, the 2½:1 gradient is used for both cases.

This criterion applies to both bearing and moment connections.

C11.1.4 **Web Crippling**. The expression for resistance to web crippling at a concentrated force is a departure from earlier specifications (IABSE, 1968; Bergfelt, 1971; Hoglund, 1971; and Elgaaly, 1983). Equations 11.1-4 and 11.1-5 are based on research by Roberts (1981). The increase in Equation 11.1-5b for $N/d > 0.2$ was developed after additional testing (Elgaaly and Salkar, 1991) to better represent the effect of longer bearing lengths at ends of members. All tests were conducted on bare steel beams without the expected beneficial contributions of any connection or floor attachments. Thus, the resulting criteria are considered conservative for such applications.

These equations were developed for bearing connections, but are also generally applicable to moment connections. However, for the rolled shapes listed in Part 1 of the LRFD Manual with F_y not greater than 345 MPa, the web crippling criterion will never control the design in a moment connection except for a W12 x 50 (W310 x 74) or W10 x 33 (W250 x 49.1) column.

The web crippling phenomenon has been observed to occur in the web adjacent to the loaded flange. For this reason, a half-depth stiffener (or stiffeners) or a half-depth doubler plate is expected to eliminate this limit state.

C11.1.5 **Web Sidesway Buckling**. The web sidesway buckling criterion was developed after observing several unexpected failures in tested beams (Summers and Yura, 1982). In those tests the compression flanges were braced at the concentrated load, the web was squeezed into compression, and the tension flange buckled (see Figure C11.1-1).

Web sidesway buckling will not occur in the following cases. For flanges restrained against rotation:

$$\frac{h/t_w}{l/b_f} > 2.3 \qquad \text{(C11.1-1)}$$

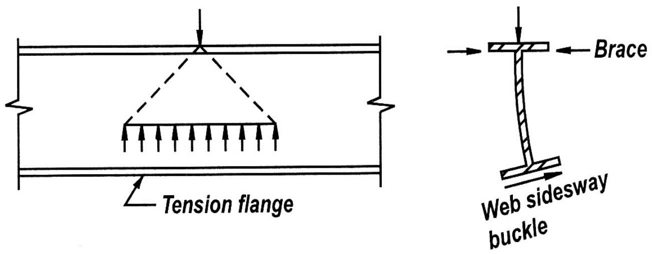


Figure. C11.1-1. Web sidesway buckling.

For flanges not restrained against rotation:

$$\frac{h/t_w}{l/b_f} > 1.7 \qquad \text{(C11.1-2)}$$

where l is as shown in Figure C11.1-2.

Web sidesway buckling can also be prevented by the proper design of lateral bracing or stiffeners at the load point. It is suggested that local bracing at both flanges be designed for one percent of the concentrated force applied at that point. Stiffeners must extend from the load point through at least one-half the beam or girder depth. In addition, the pair of stiffeners should be designed to carry the full load. If flange rotation is permitted at the loaded flange, neither stiffeners nor doubler plates will be effective.

In the 1st Edition LRFD Manual, the web sidesway buckling equations were based on the assumption that $h/t_f = 40$, a convenient assumption which is generally true for economy beams. This assumption has been removed so that the equations will be applicable to all sections.

This criterion was developed only for bearing connections and does not apply to moment connections.

C11.1.6 **Web Compression Buckling**. When compressive forces are applied to both flanges of a member at the same location, as by moment connections at both flanges of a column, the member web must have its slenderness ratio limited to avoid the possibility of buckling. This is done in the SBC 306 with Equation 11.1-8. This equation is applicable to a pair of moment connections, and to other pairs of compressive forces applied at both flanges of a member, for which N/d is small (<1). When N/d is not small, the member web should be designed as a compression member in accordance with Chapter 5.

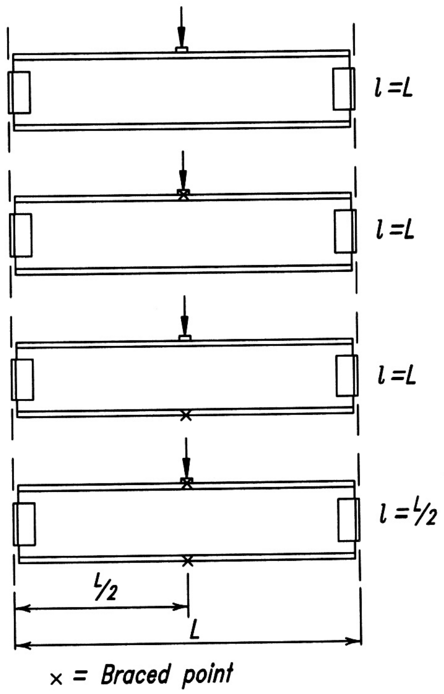


Figure. C11.1-2. Unbraced flange length.

Equation 11.1-8 is predicated on an interior member loading condition. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the compressive forces are close to the member end.

Equation 11.1-8 has also traditionally been applied when there is a moment connection to only one flange of the column and compressive force is applied to only one flange. Its use in this case is conservative.

C11.1.7 **Web Panel-Zone Shear.** The column web shear stresses may be high within the boundaries of the rigid connection of two or more members whose webs lie in a common plane. Such webs should be reinforced when the calculated factored force ΣF_u along plane A-A in Figure C11.1-3 exceeds the column web design strength ϕR_v, where

$$\Sigma F_u = \frac{M_{u1}}{d_{m1}} + \frac{M_{u2}}{d_{m2}} - V_u \qquad \text{(C11.1-3)}$$

and

$M_{u1} = M_{u1L} + M_{u1G}$ = the sum of the moments due to the factored lateral load M_{u1L} and the moments due to factored gravity load M_{u1G} on the windward side of the connection, N-mm

$M_{u2} = M_{u2L} + M_{u2G}$ = the difference between the moments due to the factored lateral load M_{u2L} and the moments due to factored gravity load M_{u2G} on the windward side of the connection, N-mm

d_{m1}, d_{m2} = distance between flange forces in a moment connection, mm

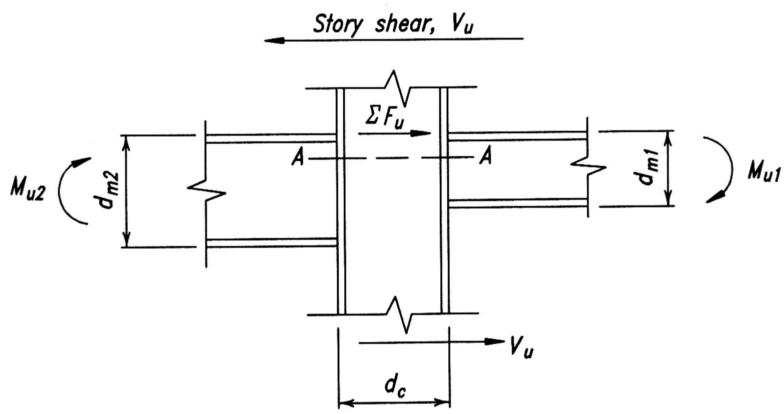


Figure. C11.1-3. Forces in panel zone.

Conservatively, 0.95 times the beam depth has been used for d_m in the past.

If $\Sigma F_u \leq \phi R_v$, no reinforcement is necessary, i.e., $t_{req} \leq t_w$, where t_w is the column web thickness.

Consistent with elastic first order analysis, Equations 11.1-9 and 11.1-10 limit panel-zone behavior to the elastic range. While such connection panels possess large reserve capacity beyond initial general shear yielding, the corresponding inelastic joint deformations may adversely affect the strength and stability of the frame or story (Fielding and Huang, 1971, and Fielding and Chen, 1973). Panel-zone shear yielding affects the overall frame stiffness and, therefore, the ultimate-strength second-order effects may be significant. The shear/axial interaction expression of Equation 11.1-10, as shown in Figure C11.1-4, is chosen to ensure elastic panel behavior.

If adequate connection ductility is provided and the frame analysis considers the inelastic panel-zone deformations, then the additional inelastic shear strength is recognized in Equations 11.1-11 and 11.1-12 by the factor

$$\left[1+\frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right]$$

This inelastic shear strength has been most often utilized for design of frames in high seismic zones and should be used when the panel zone is to be designed to match the strength of the members from which it is formed.

The shear/axial interaction expression incorporated in Equation 11.1-12 recognizes the observed fact that when the panel-zone web has completely yielded in shear, the axial column load is carried in the flanges.

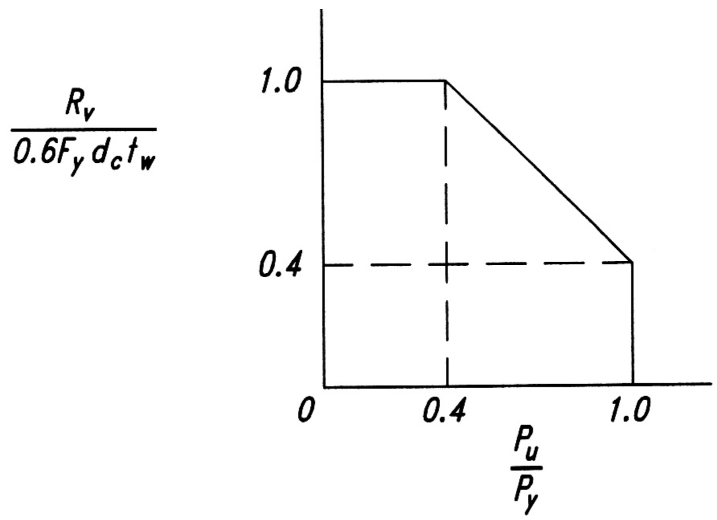


Figure. C11.1-4. Unbraced flange length.

SECTION C11.3
DESIGN FOR CYCLIC LOADING (FATIGUE)

In general, members or connections subject to less than a few thousand cycles of loading will not constitute a fatigue condition except possibly for cases involving full reversal of loading and particularly sensitive categories of details. This is because the admissible static design stress range will be limited by the admissible static design stress. At low levels of cyclic tensile stress, a point is reached where the stress range is so low that fatigue cracking will not initiate regardless of the number of cycles of loading. This level of stress is defined as the fatigue threshold, F_{TH}.

When fabrication details involving more than one category occur at the same location in a member, the stress range at that location must be limited to that of the most restrictive category. By locating notch-producing fabrication details in regions subject to a small range of stress, the need for a member larger than required by static loading will often be eliminated.

CHAPTER 12
SERVICEABILITY DESIGN CONSIDERATIONS

To satisfy the general design requirement for serviceability, the overall structure and the individual members, connections, and connectors shall be checked for serviceability.

Serviceability criteria are formulated to prevent disruptions of the functional use and damage to the structure during its normal everyday use. While malfunctions may not result in the collapse of a structure or in loss of life or injury, they can seriously impair the usefulness of the structure and lead to costly repairs. Neglect of serviceability may result in unacceptably flexible structures.

There are essentially three types of structural behavior which may impair serviceability:

(1) Excessive local damage (local yielding, buckling, slip, or cracking) that may require excessive maintenance or lead to corrosion.

(2) Excessive deflection or rotation that may affect the appearance, function, or drainage of the structure, or may cause damage to nonstructural components and their attachments.

(3) Excessive vibrations induced by wind or transient live loads which affect the comfort of occupants of the structure or the operation of mechanical equipment.

In allowable stress design, the Specification accounts for possible local damage with factors of safety included in the allowable stresses, while deflection and vibration are controlled, directly or indirectly, by limiting deflections and span-depth ratios. In the past, these rules have led to satisfactory performance of structures, with perhaps the exception of large open floor areas without partitions. In SBC 306 the serviceability checks should consider the appropriate loads, the response of the structure, and the reaction of the occupants to the structural response.

Examples of loads that may require consideration of serviceability include permanent live loads, wind, and earthquake; effects of human activities such as walking, dancing, etc.; temperature fluctuations; and vibrations induced by traffic near the building or by the operation of mechanical equipment within the building.

Serviceability checks are concerned with adequate performance under the appropriate load conditions. Elastic behavior can usually be assumed. However, some structural elements may have to be examined with respect to their long-term behavior under load.

It is difficult to specify limiting values of structural performance based on serviceability considerations because these depend to a great extent on the type of structure, its intended use, and subjective physiological reaction. For example, acceptable structural motion in a hospital clearly would be much less than in an ordinary industrial building. It should be noted that humans perceive levels of structural motion that are far less than motions that would cause any structural damage. Serviceability limits must be determined through careful consideration by the designer and client.

SECTION C12.1
CAMBER

The engineer should consider specifying camber when deflections at the appropriate load level present a serviceability problem.

SECTION C12.2
EXPANSION AND CONTRACTION

As in the case of deflections, the satisfactory control of expansion cannot be reduced to a few simple rules, but must depend largely upon the good judgment of qualified engineers.

The problem is more serious in buildings with masonry walls than with prefabricated units. Complete separation of the framing, at widely spaced expansion joints, is generally more satisfactory than more frequently located devices depend upon the sliding of parts in bearing, and usually less expensive than rocker or roller expansion bearings.

Creep and shrinkage of concrete and yielding of steel are among the causes, other than temperature, for dimensional changes.

SECTION C12.3
DEFLECTIONS, VIBRATION, AND DRIFT

C12.3.1 **Deflections.** Excessive transverse deflections or lateral drift may lead to permanent damage to building elements, separation of cladding, or loss of weather tightness, damaging transfer of load to non-load-supporting elements, disruption of operation of building service systems, objectionable changes in appearance of portions of the buildings, and discomfort of occupants.

The SBC 306 Specification does not provide specific limiting deflections for individual members or structural assemblies. Such limits would depend on the function of the structure. Provisions that limit deflections to a percentage of span may not be adequate for certain long-span floor systems; a limit on maximum deflection that is independent of span length may also be necessary to minimize the possibility of damage to adjoining or connecting nonstructural elements.

Deflection calculations for composite beams should include an allowance for slip for short-term deflection calculations, and for creep and shrinkage for long-term deflection calculations (see Commentary Section C9.3.2).

C12.3.2 **Floor Vibration**. The increasing use of high-strength materials and efficient structural schemes leads to longer spans and more flexible floor systems. Even though the use of a deflection limit related to span length generally precluded vibration problems in the past, some floor systems may require explicit consideration of the dynamic, as well as the static, characteristics of the floor system.

The dynamic response of structures or structural assemblies may be difficult to analyze because of difficulties in defining the actual mass, stiffness, and damping characteristics. Moreover, different load sources cause varying responses. For example, a steel beam-concrete slab floor system may respond to live loading as a non-composite system, but to transient excitation from human activity as an orthotropic composite plate. Nonstructural partitions, cladding, and built-in furniture significantly increase the stiffness and damping of the structure and frequently eliminate potential vibration problems. The damping can also depend on the amplitude of excitation.

The general objective in minimizing problems associated with excessive structural motion is to limit accelerations, velocities, and displacements to levels that would not be disturbing to the building occupants. Generally, occupants of a building find sustained vibrations more objectionable than transient vibrations.

The levels of peak acceleration that people find annoying depend on frequency of response. Thresholds of annoyance for transient vibrations are somewhat higher and depend on the amount of damping in the floor system. These levels depend on the individual and the activity at the time of excitation.

The most effective way to reduce effects of continuous vibrations is through vibration isolation devices. Care should be taken to avoid resonance, where the frequency of steady-state excitation is close to the fundamental frequency of the system. Transient vibrations are reduced most effectively by increasing the damping in the structural assembly. Mechanical equipment which can produce objectionable vibrations in any portion of a structure should be adequately isolated to reduce the transmission of such vibrations to critical elements of the structure.

C12.3.3 **Drift.** The SBC 306 does not provide specific limiting values for lateral drift. If a drift analysis is desired, the stiffening effect of non-load-supporting elements such as partitions and in filled walls may be included in the analysis of drift. Some irrecoverable inelastic deformations may occur at given load levels in certain types of construction. The effect of such deformations may be negligible or serious, depending on the function of the structure, and should be considered by the designer on a case by case basis.

The deformation limits should apply to structural assemblies as a whole. Reasonable tolerance should also be provided for creep. Where load cycling occurs, consideration should be given to the possibility of increases in residual deformation that may lead to incremental failure.

SECTION C12.5 CORROSION

Steel members may deteriorate in particular service environments. This deterioration may appear either in external corrosion, which would be visible upon inspection, or in undetected changes that would reduce its strength. The designer should recognize these problems by either factoring a specific amount of damage tolerance into the design or providing adequate protection systems (e.g., coatings, cathodic protection) and/or planned maintenance programs so that such problems do not occur.

CHAPTER 13
FABRICATION, ERECTION AND QUALITY CONTROL

SECTION C13.2
FABRICATION

C13.2.1 **Cambering, Curving, and Straightening.** The use of heat for straightening or cambering members is permitted for ASTM A514/A514M and ASTM A852/A852M steel, as it is for other steels. However, the maximum temperature permitted is 593°C compared to 649°C for other steels.

Cambering of flexural members, when required by the contract documents, may be accomplished in various ways. In the case of trusses and girders, the desired curvature can be built in during assembly of the component parts. Within limits, rolled beams can be cold-cambered at the producing mills.

Local application of heat has come into common use as a means of straightening or cambering beams and girders. The method depends upon an ultimate shortening of the heat-affected zones. A number of such zones, on the side of the member that would be subject to compression during cold-cambering or "gagging," are heated enough to be "upset" by the restraint provided by surrounding unheated areas. Shortening takes place upon cooling.

While the final curvature or camber can be controlled by these methods, it must be realized that some deviation, due to workmanship error and permanent change due to handling, is inevitable.

C13.2.2 **Thermal Cutting**. Preferably thermal cutting shall be done by machine. The requirement for a positive preheat of 66°C minimum when thermal cutting beam copes and weld access holes in ASTM A6/A6M Group 4 and 5 shapes, and in built-up shapes made of material more than 50 mm thick, tends to minimize the hard surface layer and the initiation of cracks.

C13.2.5 **Bolted Construction.** In the past, it has been required to tighten all ASTM A325 or A325M and A490 or A490M bolts in both slip-critical and bearing-type connections to a specified tension. The requirement was changed in 1985 to permit most bearing-type connections to be tightened to a snug-tight condition.

In a snug-tight bearing connection, the bolts cannot be subjected to tension loads, slip can be permitted and loosening or fatigues due to vibration or load fluctuations are not design considerations.

It is suggested that snug-tight bearing-type connections be used in applications where A307 bolts would be permitted.

This section provides rules for the use of oversized and slotted holes paralleling the provisions which have been in the RCSC Specification (RCSC, 1994) since 1972, extended to include A307 bolts which are outside the scope of the high-strength bolt specifications.

SECTION C13.3
SHOP PAINTING

The surface condition of steel framing disclosed by the demolition of long-standing buildings has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Even in the presence of leakage, the shop coat is found to be of minor influence.

The SBC 306 does not define the type of paint to be used when a shop coat is required. Conditions of exposure and individual preference with regard to finish paint are factors which bear on the selection of the proper primer. Hence, a single formulation would not suffice. For a comprehensive treatment of the subject, see SSPC (1989).

C13.3.5 **Surfaces Adjacent to Field Welds.** The SBC 306 allows for welding through surface materials, including appropriate shop coatings that do not adversely affect weld quality nor create objectionable fumes.

SECTION C13.4
ERECTION

C13.4.4 **Fit of Column Compression Joints and Base Plates.** Tests on spliced full-size columns with joints that had been intentionally milled out-of-square, relative to either strong or weak axis, demonstrated that the load-carrying capacity was the same as that for a similar unspliced column. In the tests, gaps of 2 mm were not shimmed; gaps of 6 mm were shimmed with non-tapered mild steel shims. Minimum size partial-joint-penetration welds were used in all tests. No tests were performed on specimens with gaps greater than 6 mm.

C13.4.5 **Field Welding.** The purpose of wire brushing shop paint on surfaces adjacent to joints to be field welded is to reduce the possibility of porosity and cracking and also to reduce any environmental hazard. Although there are limited tests which indicate that painted surfaces result in sound welds without wire brushing, other studies have resulted in excessive porosity and/or cracking when welding coated surfaces. Wire brushing to reduce the paint film thickness minimizes rejectable welds. Grinding or other procedures beyond wire brushing is not necessary.

CHAPTER 14
EVALUATION OF EXISTING STRUCTURES

SECTION C14.1
GENERAL PROVISIONS

The load combinations referred to in this chapter reflect gravity loading because it is the most prevalent condition encountered. If other loading conditions are a consideration, such as lateral loads, the appropriate load combination from SBC 301 or from the applicable building code should be used. The Engineer of Record for a project is generally established by the owner.

SECTION C14.2
MATERIALS PROPERTIES

C14.2.1 **Determination of Required Tests.** The extent of tests required depends on the nature of the project, the criticality of the structural system or member evaluated, and the availability of records pertinent to the project. Thus, the Engineer of Record is required to determine the specific tests required and the locations from which specimens are to be obtained.

C14.2.2 **Tensile Properties.** Samples required for tensile tests should be removed from regions of reduced stress, such as at flange tips at beam ends and external plate edges, to minimize the effects of the reduced area. The number of tests required will depend on whether they are conducted to merely confirm the strength of a known material or to establish the strength of some other steel. Guidance on the appropriate minimum number of tests is available (FEMA, 1997).

It should be recognized that the yield stress determined by standard ASTM methods and reported by mills and testing laboratories is somewhat greater than the static yield stress because of dynamic effects of testing. Also, the test specimen location may have an effect. These effects have already been accounted for in the nominal strength equations in the Specification. However, when strength evaluation is done by load testing, this effect should be accounted for in test planning because yielding will tend to occur earlier than otherwise anticipated. The static yield stress, F_{ys}, can be estimated from that determined by routine application of ASTM methods, F_y, by the following equation (Galambos, 1978 and 1998):

$$F_{ys} = R(F_y - 27) \qquad \text{(C14.2-1)}$$

where

F_{ys} = static yield stress (MPa)

F_y = reported yield stress (MPa)

R = 0.95 for tests taken from web specimens

R = 1.00 for tests taken from flange specimens

The R factor in Equation C14.2-1 accounts for the effect of the coupon location on the reported yield stress.

C14.2.4 **Base Metal Notch Toughness.** The Engineer of Record shall specify the location of samples. Samples shall be cored, flame cut, or saw cut. The Engineer of Record will determine if remedial actions are required, such as the possible use of bolted splice plates.

C14.2.5 **Weld Metal.** Because connections typically have a greater reliability index than structural members, strength testing of weld metal is not usually necessary. However, field investigations have sometimes indicated that complete-joint-penetration welds, such as at beam-to-column connections, were not made properly. The specified provisions in Section 14.2.4 provide a means for judging the quality of such a weld. Where feasible, any samples removed should be obtained from compression splices rather than tension splices, because the effects of repairs to restore the sampled area are less critical.

C14.2.6 **Bolts and Rivets.** Because connections typically have a greater reliability index than structural members, removal and strength testing of fasteners is not usually necessary. However, strength testing of bolts is required where they cannot be properly identified otherwise. Because removal and testing of rivets is difficult, assuming the lowest rivet strength grade simplifies the investigation.

SECTION C14.3
EVALUATION BY STRUCTURAL ANALYSIS

C14.3.2 **Strength Evaluation.** Resistance factors reflect variations in determining strength of members and connections, such as uncertainty in theory and variations in material properties and dimensions. If an investigation of an existing structure indicates that there are variations in material properties or dimensions significantly greater than those anticipated in new construction, the Engineer of Record should consider the use of more conservative values.

SECTION C14.4
EVALUATION BY LOAD TESTS

C14.4.1 **Determination of Live Road Rating by Testing.** Generally, structures that can be designed according to the provisions of this Specification need no confirmation of calculated results by test. However, special situations may arise when it is desirable to confirm by tests the results of calculations. Minimal test procedures are provided to determine the live load rating of a structure. However, in no case is the live load rating determined by test to exceed that which can be calculated using the provisions of the Code. This is not intended to preclude testing to evaluate special conditions or configurations that are not adequately covered by this Code.

It is essential that the Engineer of Record take all necessary precautions to ensure that the structure does not fail catastrophically during testing. A careful assessment of structural conditions before testing is a fundamental requirement. This includes accurate measurement and characterization of the size and strength of members, connections, and details. Shoring and scaffolding should be used as required in the proximity of the test area to mitigate against unexpected circumstances. Deformations must be carefully monitored and structural conditions must be continually evaluated. In some cases it may be desirable to monitor strains as well.

The Engineer of Record must use judgment to determine when deflections are becoming excessive and terminate the tests at a safe level even if the desired loading has not been achieved. Incremental loading is specified so that deformations can be accurately monitored and the performance of the structure carefully observed. Load increments should be small enough initially so that the onset of significant yielding can be determined. The increment can be reduced as the level of inelastic behavior increases, and the behavior at this level carefully evaluated to determine when to safely terminate the test. Periodic unloading after the onset of inelastic behavior will help the Engineer of Record determine when to terminate the test to avoid excessive permanent deformation or catastrophic failure.

It must be recognized that the margin of safety at the maximum load level used in the test may be very small, depending on such factors as the original design, the purpose of the tests, and the condition of the structure. Thus, it is imperative that all appropriate safety measures be adopted. It is recommended that the maximum live load used for load tests be selected conservatively. It should be noted that experience in testing more than one bay of a structure is limited.

Criteria limiting increases in deformations for a period of one hour have been given to ensure that the structure is stable at the loads evaluated.

C14.4.2 **Serviceability Evaluation.** In certain cases serviceability criteria must be determined by load testing. It should be recognized that complete recovery (i.e., return to initial deflected shape) after removal of maximum load is unlikely because of phenomena such as local yielding, slip at the slab interface in composite construction, creep in concrete slabs, localized crushing or deformation at shear connections in slabs, slip in bolted connections, and effects of continuity. Because most structures exhibit some slack when load is first applied, it is appropriate to project the load-deformation curve back to zero load to determine the slack and exclude it from the recorded deformations. Where desirable, the applied load sequence can be repeated to demonstrate that the structure is essentially elastic under service loads and that the permanent set is not detrimental.

SECTION C14.5
EVALUATION REPORT

Extensive evaluation and load testing of existing structures is often performed when appropriate documentation no longer exists or when there is considerable disagreement about the condition of a structure. The resulting evaluation is only effective if well documented, particularly when load testing is involved. Furthermore, as time passes, various interpretations of the results can arise unless all parameters of the structural performance, including material properties, strength, and stiffness, are well documented.

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