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	
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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 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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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) $f'_{cr} = 0.90 f'_c + 2.33s$ (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	$1.10 f'_c + 5.0$

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 $^{\circ}$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°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
$d \le 200\,mm$	±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.

$$U = 1.4\,(D + F) \quad (9\text{-}1)$$

$$U = 1.4\,(D + F + T) + 1.7(L + H) + 0.5\,(L_r \text{ or } R) \quad (9\text{-}2)$$

$$U = 1.2D + 1.6\,(L_r \text{ or } R) + (1.0\,L \text{ or } 0.8\,W) \quad (9\text{-}3)$$

$$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } R) \quad (9\text{-}4)$$

$$U = 1.2D + 1.0E + 1.0L \quad (9\text{-}5)$$

$$U = 0.9D + 1.6W + 1.6H \quad (9\text{-}6)$$

$$U = 0.9D + 1.0E + 1.6H \quad (9\text{-}7)$$

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 $\left(h-d^{'}-d_s\right)/h$ 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 $1.8f_{ct}$ for $\sqrt{f_c'}$, but the value of $1.8f_{ct}$ 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

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.

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

$$h = \frac{\ell_n\left(0.8 + \frac{f_y}{1500}\right)}{36 + 5\beta(\alpha_m - 0.2)} \qquad (9\text{-}12)$$

and not less than 120 mm;

(c) For α_m greater than 2.0, the thickness shall not be less than

$$h = \frac{\ell_n\left(0.8 + \frac{f_y}{1500}\right)}{36 + 9\beta} \qquad (9\text{-}13)$$

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, $N \leq \phi P_n$

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:

$$\phi P_{n,\max} = 0.85\phi\left[0.85 f_c^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad (10\text{-}1)$$

10.3.6.2 For nonprestressed members with tie reinforcement conforming to 7.10.5:

$$\phi P_{n,\max} = 0.80\phi\left[0.85 f_c^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad (10\text{-}2)$$

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

$$A_{s,\min} = \frac{\sqrt{f_c^{'}}}{4 f_y} b_w d \qquad (10\text{-}3)$$

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:

$$\frac{k\ell_u}{r} \leq 34 - 12(M_1 / M_2) \qquad (10\text{-}7)$$

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

$$\delta_{ns} = \frac{C_m}{1 - \dfrac{P_u}{0.75P_c}} \geq 1.0 \tag{10-9}$$

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \tag{10-10}$$

EI shall be taken as

$$EI = \frac{(0.2E_c I_g + E_s I_{se})}{1 + \beta_d} \tag{10-11}$$

$$EI = \frac{0.4E_c I_g}{1 + \beta_d} \tag{10-12}$$

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

$$M_1 = M_{1ns} + \delta_s M_{1s} \tag{10-15}$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \tag{10-16}$$

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

$$\delta_s M_s = \frac{M_s}{1-Q} \geq M_s \tag{10-17}$$

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

$$\delta_s M_s = \frac{M_s}{1 - \dfrac{\sum P_u}{0.75 \sum P_c}} \geq M_s \tag{10-18}$$

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.

$$\frac{\ell_u}{r} > \frac{35}{\sqrt{\dfrac{P_u}{f_c' A_g}}} \tag{10-19}$$

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

$$r = \sqrt{\frac{\left(E_c I_g / 5\right) + E_s I_t}{\left(E_c A_g / 5\right) + E_s A_t}} \tag{10-20}$$

and, as an alternative to a more accurate calculation, EI in Eq. (10-10) shall be taken either as Eq. (10-11) or

$$EI = \frac{\left(E_c I_g / 5\right)}{1 + \beta_d} + E_s I_t \tag{10-21}$$

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

$b\sqrt{\frac{f_y}{3E_s}}$ for each face of width b

or

$h\sqrt{\frac{f_y}{8E_s}}$ 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:

$$\phi V_n \geq V_u \tag{11-1}$$

where V_u is the factored shear force at the section considered and V_n is nominal shear strength computed by:

$$V_n = V_c + V_s \tag{11-2}$$

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,

$$V_c = \left(1 + \frac{N_u}{14A_g}\right)\left(\frac{\sqrt{f_c^{'}}}{6}\right) b_w d \qquad (11\text{-}4)$$

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

$$V_c = 0.3 \times \sqrt{f_c'} \cdot b_w d \sqrt{1 + \frac{0.3 N_u}{A_g}} \qquad (11\text{-}7)$$

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,

$$V_c = \left(1 + \frac{0.3 N_u}{A_g}\right) \frac{\sqrt{f_c'}}{6} b_w d \qquad (11\text{-}8)$$

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,

$$V_c = \left(\frac{\sqrt{f_c'}}{20} + 5 \frac{V_u d}{M_u}\right) b_w d \qquad (11\text{-}9)$$

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

$$V_{ci} = \frac{\sqrt{f_c'}}{20} b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad (11\text{-}10)$$

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

$$A_v = \frac{1}{16}\sqrt{f_c'}\frac{b_w s}{f_y} \tag{11-13}$$

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)

$$A_v = \frac{A_{ps} f_{pu} s}{80 f_y d} \sqrt{\frac{d}{b_w}} \qquad (11\text{-}14)$$

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,

$$V_s = \frac{A_v f_y d}{s} \qquad (11\text{-}15)$$

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,

$$V_s = \frac{A_v f_y (\sin\alpha + \cos\alpha)\, d}{s} \qquad (11\text{-}16)$$

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,

$$V_s = A_v f_y \sin\alpha \qquad (11\text{-}17)$$

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 $M_u /\left(0.9df_{y\ell}\right)$ 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

$$V_c = \left(\beta_p \sqrt{f_c'} + 0.3 f_{pc}\right) b_o d + V_p \qquad (11\text{-}36)$$

where β_p, is the smaller of 0.29 or $\left(\alpha_s d / b_o + 1.5\right)/12$, 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

$$M_p = \frac{V_u}{2\phi\eta}\left[h_v + \alpha_v\left(\ell_v - \frac{c_1}{2}\right)\right] \tag{11-37}$$

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

$$M_v = \frac{\phi\alpha_v V_u}{2\eta}\left(\ell_v - \frac{c_1}{2}\right) \tag{11-38}$$

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

$$\gamma_v = (1 - \gamma_f) \qquad (11\text{-}39)$$

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:

$$\phi v_n = \phi V_c / (b_0 d) \qquad (11\text{-}40)$$

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:

$$\phi v_n = \phi (V_c + V_s) / (b_o d) \qquad (11\text{-}41)$$

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:

$$\ell_d = \left(\frac{9}{10} \frac{f_y}{\sqrt{f_c^{'}}} \frac{\alpha\beta\gamma\lambda}{\left(\frac{c + k_{tr}}{d_b}\right)} \right) d_b \tag{12-1}$$

In which the term $(c + k_{tr})/d_b$ 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	$\alpha = 1.3$
Other reinforcement	$\alpha = 1.0$
Epoxy-coated bars or wires with cover less than $3d_b$ or clear spacing less than $6d_b$	$\beta = 1.5$
All other epoxy-coated bars or wires	$\beta = 1.2$
Uncoated reinforcement	$\beta = 1.0$

However, the product $\alpha\beta$ need not be taken greater than 1.7.

Dia 20 mm and smaller bars and deformed wires	$\gamma = 0.8$
Dia 22 mm and larger bars	$\gamma = 1.0$

When lightweight aggregate concrete is used	$\lambda = 1.3$
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	$\lambda = 1.0$

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 = $\frac{A_{tr} f_{yt}}{10sn}$

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.

$$\ell_d \leq \frac{M_n}{V_u} + \ell_a \tag{12-3}$$

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 $12d_b$, 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 $(0.13 f_y - 24) d_b$ 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. See13.5.3.2

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.

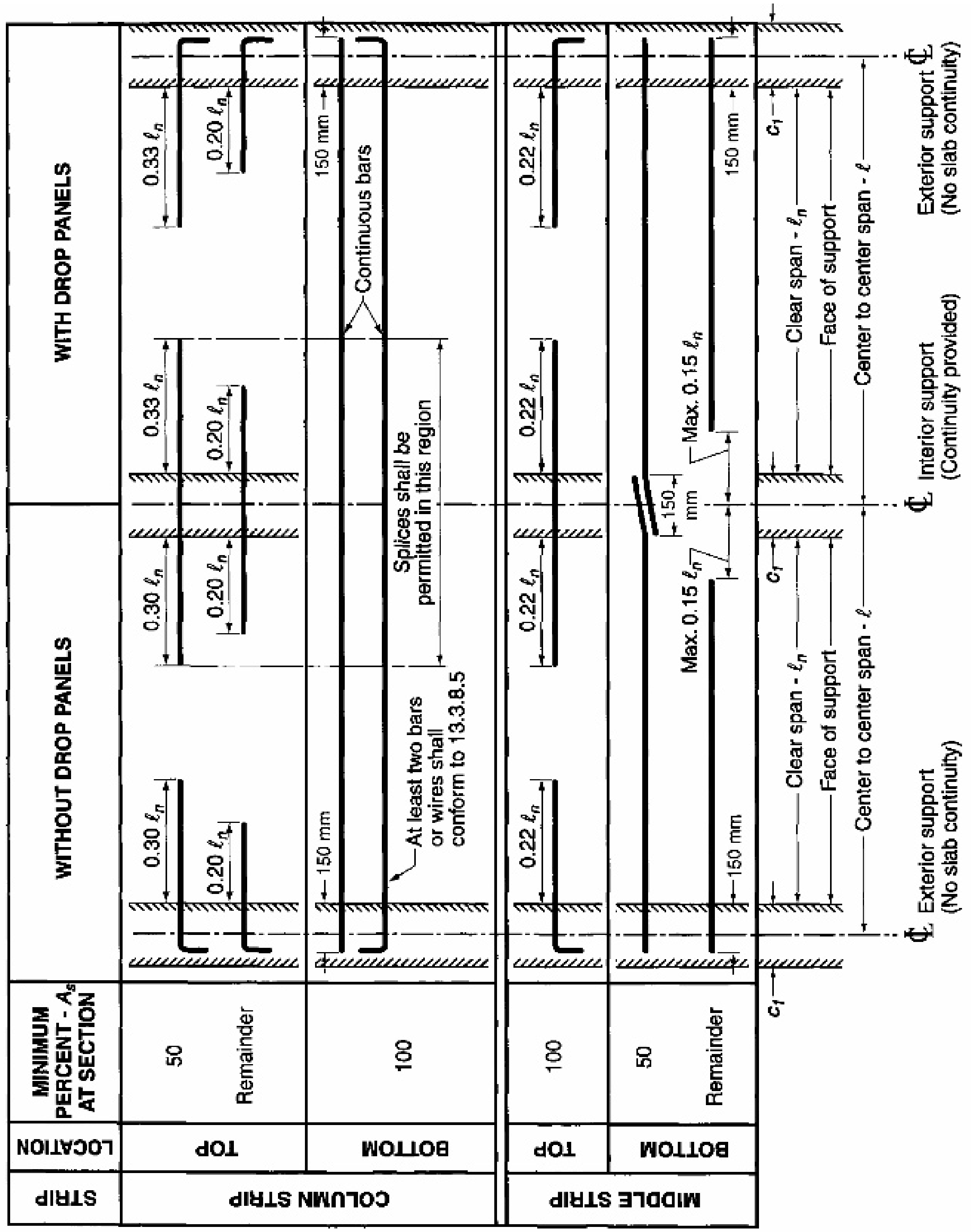


Fig. 13.3.8 – Minimum extensions for reinforcement in slabs without beams. (See 12.11.1 for reinforcement extension into supports)

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

$$\gamma_f = \frac{1}{1+(2/3)\sqrt{b_1/b_2}} \qquad (13\text{-}1)$$

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.

$$M = 0.07[(w_d + 0.5w_l)l_2 l_n^2 - w'_d l'_2 (l'_n)^2] \qquad (13\text{-}4)$$

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

$$\phi M_n \geq M_u \tag{14-3}$$

where:

$$M_u = M_{ua} + P_u \Delta_u \tag{14-4}$$

M_{ua} is the moment at the midheight section of the wall due to factored loads, and Δ_u is:

$$\Delta_u = \frac{5 M_u \ell_c^2}{(0.75) 48 E_c I_{cr}} \tag{14-5}$$

M_u shall be obtained by iteration of deflections, or by direct calculation using Eq. (14-6).

$$M_u = \frac{M_{ua}}{1 - \dfrac{5 P_u \ell_c^2}{(0.75) 48 E_c I_{cr}}} \tag{14-6}$$

where:

$$I_{cr} = n A_{se} (d - c)^2 + \frac{\ell_w c^3}{3} \tag{14-7}$$

and

$$A_{se} = \frac{P_u + A_s f_y}{f_y} \tag{14-8}$$

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:

$$\Delta_s = \frac{(5M) \ell_c^2}{48 E_c I_e} \tag{14-9}$$

$$M = \frac{M_{sa}}{1 - \dfrac{5 P_s \ell_c^2}{48 E_c I_e}} \tag{14-10}$$

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.

$$\frac{\begin{array}{c}\textit{Reinforcement in}\\ \textit{band width}\end{array}}{\begin{array}{c}\textit{Total reinforcement}\\ \textit{in short direction}\end{array}} = \frac{2}{(\beta+1)} \qquad (15\text{-}1)$$

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

$$V_u \leq \phi V_{nh} \qquad (17\text{-}1)$$

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

$$P_s = P_x e^{(K\ell_x + \mu\alpha)} \tag{18-1}$$

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 $2^{\circ}C$ and shall be maintained above $2^{\circ}C$ 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

$\Delta_{\max}$ = 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:

$$\Delta_{\max} \le \frac{\ell_t^2}{20{,}000h} \qquad (20\text{-}1)$$

$$\Delta_{r\max} \le \frac{\Delta_{\max}}{4} \qquad (20\text{-}2)$$

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:

$$\Delta_{r\max} \le \frac{\Delta_{f\max}}{5} \qquad (20\text{-}3)$$

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).

$$A_{sh} = 0.3\left(sh_c f_c^{'} / f_{yh}\right)\left(\left(A_g / A_{ch}\right) - 1\right) \tag{21-3}$$

$$A_{sh} = 0.09\, sh_c f_c^{'} / f_{yh} \tag{21-4}$$

(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).

$$\ell_{dh} = f_y d_b / \left(4.5\sqrt{f_c^{'}}\right) \tag{21-6}$$

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

$$V_n = A_{cv}\left(\alpha_c\sqrt{f_c^{'}} + \rho_n f_y\right) \quad (21\text{-}7)$$

where the coefficient α_c is 1/4 for $h_w/\ell_w \leq 1.5$, is 1/6 for $h_w/\ell_w \geq 2.0$, 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:

$$c \geq \frac{\ell_w}{600(\delta_u / h_w)} \qquad (21\text{-}8)$$

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

$$V_n = 2A_{vd} f_y \sin\alpha \le (5/6)\sqrt{f_c^{'}} A_{cp} \qquad (21\text{-}9)$$

(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

$$\phi F_n \geq F_u \qquad \text{(A-1)}$$

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

$$F_{ns} = f_{cu} A_c \qquad \text{(A-2)}$$

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

$$f_{cu} = 0.85 \beta_s f_c^{'} \qquad \text{(A-3)}$$

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

$$\sum \frac{A_{si}}{b s_i} \sin \gamma_i \geq 0.003 \qquad \text{(A-4)}$$

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 $0.1f_c'A_g$ 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

$$20\left(1 - \frac{\omega_p + \frac{d}{d_p}(\omega - \omega')}{0.36\beta_1}\right) \textit{percent}$$

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.

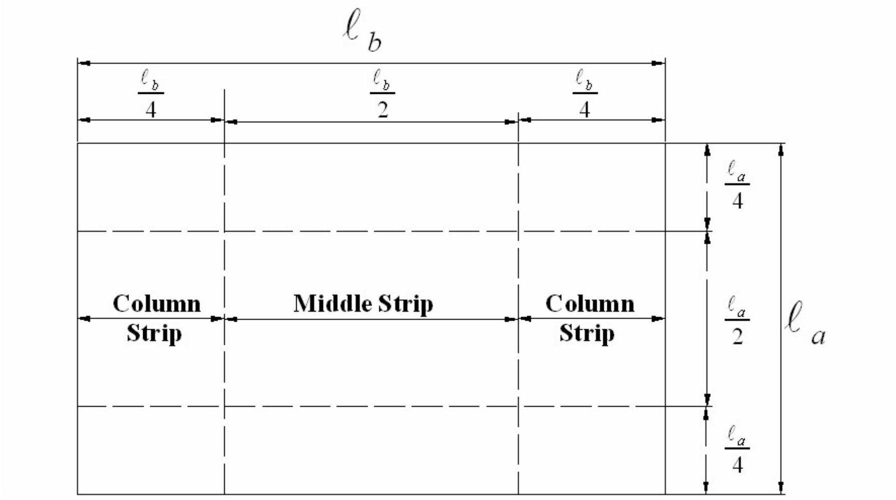
C.2.4 **Factored shear in slab systems with beams**

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.



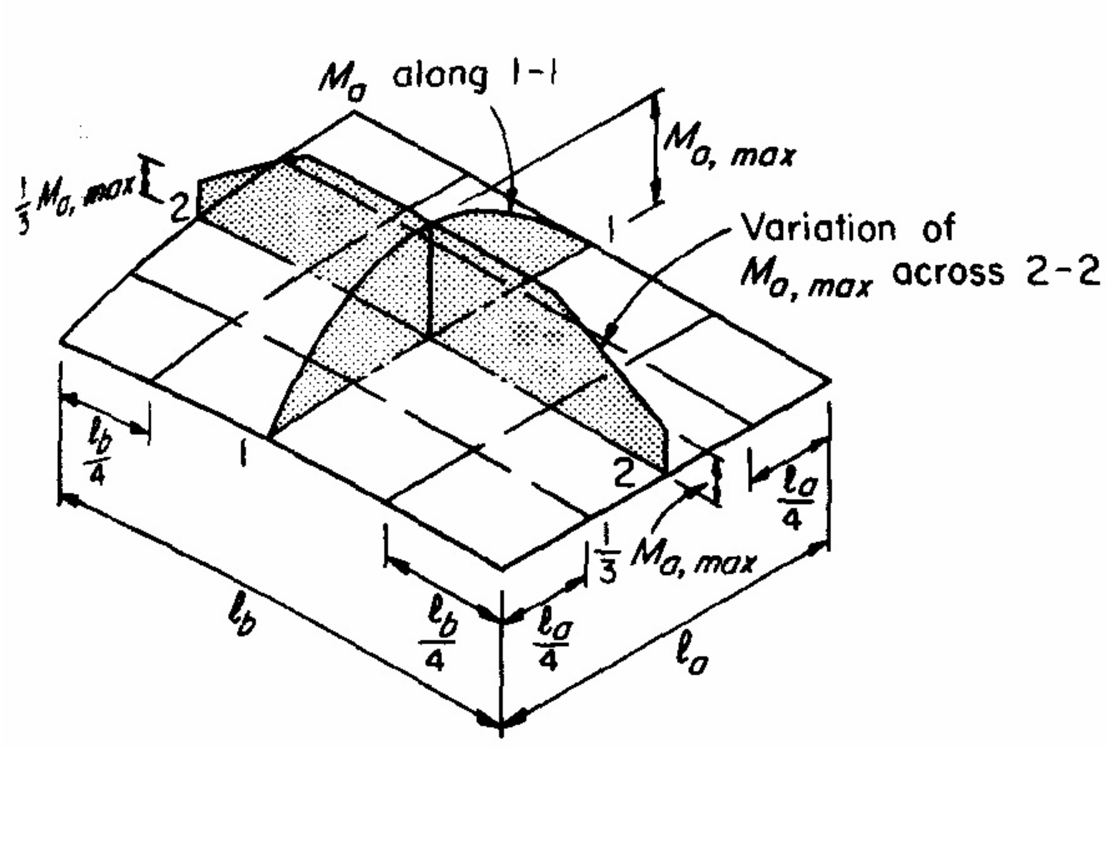


Table 2: Coefficients for negative moments in slabs (*)

$$M_{a,neg} = C_{a,neg} w \ell_a^2$$
$$M_{b,neg} = C_{b,neg} w \ell_b^2$$

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,

$\phi n N_{pn}$ 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

$$N_b = (3.8)\sqrt{f_c'}\;\; h_{ef}^{5/3} \qquad \text{(D-8)}$$

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:

$$\psi_1 = \frac{1}{\left(1 + \dfrac{2e_N'}{3h_{ef}}\right)} \le 1 \qquad \text{(D-9)}$$

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:

$$\psi_2 = 1 \;\; if \;\; \mathrm{c}_{\min} \ge 1.5h_{ef} \qquad \text{(D-10)}$$

$$\psi_2 = 0.7 + 0.3\frac{c_{\min}}{1.5h_{ef}} \;\; \text{if } \mathrm{c}_{\min} < 1.5h_{ef} \qquad \text{(D-11)}$$

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: $3d_o \le e_h \le 4.5 d_o$

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 $(c < 0.4 h_{ef})$ 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:

$$N_{sbg} = \left(1 + \frac{s_o}{6c}\right) N_{sb} \quad \text{(D-16)}$$

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

$$V_s = nA_{se}f_{ut} \quad \text{(D-17)}$$

where f_{ut} shall not be taken greater than $1.9f_y$ or 860 MPa

(b) for cast-in headed bolt and hooked bolt anchors

$$V_s = 0.6\, nA_{se}f_{ut} \quad \text{(D-18)}$$

where f_{ut} shall not be taken greater than $1.9f_y$ or 860 MPa.

(c) for post-installed anchors

$$V_s = n(0.6A_{se}f_{ut} + 0.4A_{si}f_{utsi}) \quad \text{(D-19)}$$

where f_{ut} shall not be taken greater than $1.9f_y$ 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:

$$V_{cb} = \frac{A_v}{A_{vo}}\psi_6\psi_7 V_b \quad \text{(D-20)}$$

(b) for shear force perpendicular to the edge on a group of anchors:

$$V_{cbg} = \frac{A_v}{A_{vo}}\psi_5\psi_6\psi_7 V_b \quad \text{(D-21)}$$

(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 $\left(f_t < f_r\right)$ 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} ≥ 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 $V_u \le 0.2\phi V_n$, then full strength in tension shall be permitted: $\phi N_n \ge N_u$.

D.7.2 If $N_u \le 0.2\phi N_n$, then full strength in shear shall be permitted: $\phi V_n \ge V_u$.

D.7.3 If $V_u > 0.2\phi V_n$ and $N_u > 0.2\phi N_n$, then:

$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \le 1.2 \qquad \text{(D-29)}$$

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

$\mathbf{A_n}$ = area of reinforcement in bracket or corbel resisting tensile force N_{uc} mm², Chapter 11

$\mathbf{A_n}$ = area of a face of a nodal zone or a section through a nodal zone, mm², Appendix A

$\mathbf{A_N}$ = 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

$\mathbf{A_{No}}$ = 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

$\mathbf{A_o}$ = gross area enclosed by shear flow path, mm², Chapter 11

$\mathbf{A_{oh}}$ = area enclosed by centerline of the outermost closed transverse torsional reinforcement, mm², Chapter 11

$\mathbf{A_{ps}}$ = area of prestressed reinforcement in a tie, mm², Appendix A

$\mathbf{A_{ps}}$ = area of prestressed reinforcement in tension zone, mm², Chapters 11, 18

$\mathbf{A_s}$ = area of longitudinal tension reinforcement in wall segment, mm^2, Chapter 14

$\mathbf{A_s}$ = area of nonprestressed tension reinforcement, mm², Chapters 8, 10, 11, 12 and 18

$\mathbf{A_s}$ = area of tension reinforcement, mm², Appendix B

$\mathbf{A_s'}$ = area of compression reinforcement, mm², Chapters 8, 9, 18, Appendix B

$\mathbf{A_s'}$ = area of compression reinforcement in a strut, mm², Appendix A

$\mathbf{A_{se}}$ = area of effective longitudinal tension reinforcement in wall segment, mm², as calculated by Eq. (14-8), Chapter 14

$\mathbf{A_{se}}$ = effective cross-sectional area of anchor, mm², Appendix D

$\mathbf{A_{sh}}$ = total cross-sectional area of transverse reinforcement (including crossties) within spacing "s" and perpendicular to dimension h_c mm², Chapter 21

$\mathbf{A}_{si}$ = area of surface reinforcement in the i th layer crossing a strut, mm², Appendix A

$\mathbf{A_{sl}}$ = effective cross-sectional area of expansion or undercut anchor sleeve, if sleeve is within shear plane, mm², Appendix D

$\mathbf{A_{s,min}}$ = minimum amount of flexural reinforcement, mm². See 10.5, Chapter 10

$\mathbf{A_{st}}$ = total area of longitudinal reinforcement, (bars or steel shapes), mm², Chapter 10

$\mathbf{A_{st}}$ = area of nonprestressed reinforcement in a tie, mm², Appendix A

$\mathbf{A_t}$ = area of structural steel shape, pipe, or tubing in a composite section, mm², Chapter 10

$\mathbf{A_t}$ = area of one leg of a closed stirrup resisting torsion within a distance, "s" mm², Chapter 11

$\mathbf{A_{tr}}$ = 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

$\mathbf{A_v}$ = 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

$\mathbf{A_v}$ = area of shear reinforcement within a distance "s", mm², Chapter 12

$\mathbf{A_v}$ = area of ties within a distance "s", mm², Chapter 17

$\mathbf{A_V}$ = 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

$\mathbf{A_{vd}}$ = total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam, mm², Chapter 21

$\mathbf{A_{vf}}$ = area of shear-friction reinforcement, mm², Chapter 11

$\mathbf{A_{vh}}$ = area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2, mm², Chapter 11

$\mathbf{A_{Vo}}$ = 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

$\mathbf{A_w}$ = area of an individual wire to be developed or spliced, mm², Chapter 12

$\mathbf{A_1}$ = loaded area, mm², Chapter 10

$\mathbf{A_2}$ = 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

$\boldsymbol{b}$ = width of compression face of member, mm, Chapters 8 through 11, 18

$\boldsymbol{b}$ = width of member, mm, Appendix A

$\boldsymbol{b}$ = effective compressive flange width of a structural member, mm, Chapter 21

$\boldsymbol{B_n}$ = nominal bearing load, N

$\boldsymbol{b_o}$ = perimeter of critical section for slabs and footings, mm, Chapter 11

$\boldsymbol{b_t}$ = width of that part of cross section containing the closed stirrups resisting torsion, mm, Chapter 11

$\boldsymbol{b_v}$ = width of cross section at contact surface being investigated for horizontal shear, mm, Chapter 17

$\boldsymbol{b_w}$ = web width, mm, Chapter 10

$\boldsymbol{b_w}$ = web width, or diameter of circular section, mm, Chapters 11, 12, 21

$\boldsymbol{b_1}$ = 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

$\boldsymbol{b_2}$ = Width of the critical section defined in 11.12.1.2 measured in the direction perpendicular to b_1, mm, Chapters 11, 13

$\boldsymbol{c}$ = distance from extreme compression fiber to neutral axis, mm, Chapter 9

$\boldsymbol{c}$ = distance from extreme compression fiber to neutral axis, mm, Chapters 10, 14, Appendix A

$\boldsymbol{c}$ = spacing or cover dimension, mm. See 12.2.4, Chapter 12

$\boldsymbol{c}$ = 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

$\boldsymbol{c}$ = 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

h = thickness of member in which an anchor is anchored, measured parallel to anchor axis, mm, Appendix D

h_t = effective height of tie, mm, Appendix A

H = loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces, Chapter 9

h_c = cross-sectional dimension of column core measured center-to-center of confining reinforcement, mm, Chapter 21

h_{ef} = effective anchor embedment depth, mm. (See D.8.5 and Fig. RD.1 in SBC 304C), Appendix D

h_v = total depth of shearhead cross section, mm, Chapter 11

h_w = total height of wall from base to top, mm, Chapter 11

h_w = height of entire wall or of the segment of wall considered, mm, Chapter 21

h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the column, mm, Chapter 21

I = moment of inertia of section resisting externally applied factored loads, mm^4,Chapter 11

I_b = moment of inertia about centroidal axis of gross section of beam as defined in 13.2.4, mm^4, Chapter 13

I_{cr} = moment of inertia of cracked section transformed to concrete, mm^4, Chapters 9, 14

I_e = effective moment of inertia for computation of deflection, mm^4, Chapters 9, 14

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4, Chapters 9, 10

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

I_{se} = moment of inertia of reinforcement about centroidal axis of member cross section, mm^4, Chapter 10

I_t = moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross section, mm^4, Chapter 10

k = effective length factor for compression members, Chapter 10

k = coefficient for basic concrete breakout strength in tension, Appendix D

k = effective length factor, Chapter 14

K = wobble friction coefficient per foot of tendon, Chapter 18

k_{cp} = coefficient for pryout strength, Appendix D

k_t = torsional stiffness of torsional member; moment per unit rotation. (See R13.75 in SBC 304C), Chapter 13

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

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

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
$A_s f_y (d - \mathrm{a}/2)$, 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

$M_{2,min}$ = minimum value of M_2, N-mm, Chapter 10

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

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

n = number of bars or wires being spliced or developed along the plane of splitting, Chapter 12

n = modular ratio of elasticity, but not less than 6
E_s / E_c ,Chapter 14

n = number of monostrand anchorage devices in a group, Chapter 18

n = number of anchors in a group, Appendix D

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

N_c = tensile force in concrete due to unfactored dead load plus live load $(D+L)$, N, Chapter 18

N_{cb} = nominal concrete breakout strength in tension of a single anchor, as defined in D.5.2.1, N, Appendix D

N_{cbg} = nominal concrete breakout strength in tension of a group of anchors, as defined in D.5.2.1, N, Appendix D

N_n = nominal strength in tension, N, Appendix D

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

N_{pn} = nominal pullout strength in tension of a single anchor, as defined in D.5.3.1, N, Appendix D

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

N_{sb} = side-face blowout strength of a single anchor,N, Appendix D

N_{sbg} = side-face blowout strength of a group of anchors, N, Appendix D

N_u = factored tensile load, N, Appendix D

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

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

P_b = nominal axial load strength at balanced strain conditions, N. See 10.3.2, Chapters 9, 10, Appendix B

P_c = critical load, N. See Eq. (10-10), Chapter 10

p_{cp} = outside perimeter of the concrete cross section, *mm.* See 11.6.1, Chapter 11

p_h = perimeter of centerline of outermost closed transverse torsional reinforcement, mm, Chapter 11

P_n = nominal axial load strength at given eccentricity, N, Chapters 9, 10, Appendix B

P_o = nominal axial load strength at zero eccentricity, N, Chapter 10

P_{nw} = nominal axial load strength of wall designed by 14.4, N, Chapter 14

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

T = cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete, Chapter 9
T_n = nominal torsional moment strength, N-mm, Chapter 11
T_u = factored torsional moment at section, N-mm, Chapter 11
U = required strength to resist factored loads or related internal moments and forces, Chapter 9
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
V_c = nominal shear strength provided by concrete, N, See 11.12.2.1, Chapters 8, 11, 13, 21
V_{cb} = nominal concrete breakout strength in shear of a single anchor, as defined in D.6.2.1, N, Appendix D
V_{cbg} = nominal concrete breakout strength in shear of a group of anchors, as defined in D.6.2.1, N, Appendix D
V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, N, Chapter 11
V_{cp} = nominal concrete pryout strength, as defined in D.6.3, N, Appendix D
V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web, N, Chapter 11
V_d = shear force at section due to unfactored dead load, N, Chapter 11
V_e = design shear force determined from 21.3.4.1 or 21.4.5.1, N, Chapter 21
V_i = factored shear force at section due to externally applied loads occurring simultaneously with $M_{\max}$, N, Chapter 11
V_n = nominal shear strength, N, Chapters 11, 21, Appendix C
v_n = nominal shear stress, MPa. See 11.12.6.2, Chapter 11
V_{nh} = nominal horizontal shear strength, N, Chapter 17
V_p = vertical component of effective prestress force at section, N, Chapter 11
V_s = nominal shear strength provided by shear reinforcement, N, Chapter 11
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
V_u = factored shear force at section, N, Chapters 11-13, 17, 21
V_u = factored shear load, N, Appendix D
V_u = factored horizontal shear in a story, N, Chapter 10
W = wind load, or related internal moments and forces, Chapter 9
w_c = unit weight of concrete, kg/m^3, Chapters 8, 9
w_d = factored dead load per unit area, Chapter 13
w_ℓ = factored live load per unit area, Chapter 13
w_u = factored load per unit length of beam or per unit area of slab, Chapter 8
w_u = factored load per unit area, Chapter 13
x = shorter overall dimension of rectangular part of cross section, mm, Chapter 13
y = longer overall dimension of rectangular part of cross section, mm, Chapter 13
y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, mm, Chapters 9, 11
α = angle between inclined stirrups and longitudinal axis of member, Chapter 11
α = 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 mm^2	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
WD 6.5	6.5	33.2	0.26	664	443	332	221	166	133	111	95	83
WD 7.0	7	38.5	0.302	770	513	385	257	193	154	128	110	96
WD 7.5	7.5	44.2	0.347	884	589	442	295	221	177	147	126	111
WD 8.0	8	50.3	0	1006	671	503	335	252	201	168	144	126
WD 8.5	8.5	56.7	0.445	1134	756	567	378	284	227	189	162	142
WD 9.0	9	63.6	0.499	1272	848	636	424	318	254	212	182	159
WD 9.5	9.5	70.9	0.556	1418	945	709	473	355	284	236	203	177
WD 10.0	10	78.5	0.617	1570	1047	785	523	393	314	262	224	196
WD 10.5	10.5	86.6	0.68	1732	1155	866	577	433	346	289	247	217
WD 11.0	11	95	0.746	1900	1267	950	633	475	380	317	271	238
WD 11.5	11.5	103.9	0.815	2078	1385	1039	693	520	416	346	297	260
WD 12.0	12	113.1	0.888	2262	1508	1131	754	566	452	377	323	283

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.)