Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01)

Reported by ACI Committee 349

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This standard covers the proper design and construction of concrete structures which form part of a nuclear power plant and which have nuclear safety related functions, but does not cover concrete reactor vessels and concrete containment structures (as defined by ACI-ASME Committee 359).

The structures covered by the Code include concrete structures inside and outside the containment system.

This Code may be referenced and applied subject to agreement between the Owner and the Regulatory Authority.

The format of this Code is based on the "Building Code Requirement for Structural Concrete (ACI 318-95)" and incorporates recent revisions of that standard, except for Chapter 12, which is based on ACI 318-99.

Keywords: admixtures; aggregates; anchorage (structural); beam-column frame; beams (supports); building codes; cements; cold weather construction; columns (supports); combined stress; composite construction (concrete and steel); composite construction (concrete to concrete); compressive strength; concrete construction; concretes; concrete slabs; construction joints; continuity (structural); cover; cracking (fracturing); creep properties; curing; deep beams; deflection; drawings (drafting); earthquake resistant structures; edge beams; embedded service ducts; flexural strength; floors; folded plates; footings; formwork (construction); frames; hot weather construction; inspection; joists; loads (forces); load tests (structural); mixing; mix proportioning; modules of elasticity; moments; nuclear power plants; nuclear reactor containments; nuclear reactors; nuclear reactor safety; pipe columns; pipes (tubes); placing; precast concrete; prestressed concrete; prestressing steels; quality control; reinforced concrete; reinforcing steels; roofs; safety; serviceability; shear strength; shearwalls; shells (structural forms); spans; specifications; splicing; strength; strength analysis; structural analysis; structural design; T-beams; temperature; torsion; walls; water; welded wire fabric.

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About the presentation: To aid the reader in distinguishing changes between the 1997 version of the ACI 349 Code and this 2001 edition, all new or revised sections are marked by a sidebar to the left of the column.

PART 1—GENERAL

CHAPTER 1—GENERAL REQUIREMENTS

1.1—Scope

This Code provides the minimum requirements for the design and construction of nuclear safety related concrete structures and structural elements for nuclear power generating stations. Safety related structures and structural elements subject to this standard are those concrete structures which support, house, or protect nuclear safety class systems or component parts of nuclear safety class systems.

Specifically excluded from this Code are those structures covered by "Code for Concrete Reactor Vessels and Containments," ASME Boiler and Pressure Vessel Code Section III, Division 2, and pertinent General Requirements (ACI Standard 359).

1.1.1 This Code includes design and loading conditions that are unique to nuclear facilities including shear design under biaxial tension conditions, consideration of thermal and seismic effects, and impact and impulsive loads.

1.1.2 This Code shall govern in all matters pertaining to design and construction of reinforced-concrete structures, as defined in 1.1.1, except where the Code is in conflict with the specific provisions of the regulatory or jurisdictional authorities.

1.1.3 This Code shall govern in all matters pertaining to design, construction, and material properties wherever this Code is in conflict with requirements contained in other standards referenced in this Code.

1.1.4 For special structures, such as arches, tanks, reservoirs, bins and silos, blast-resistant structures, and chimneys, provisions of this Code shall govern where applicable.

1.1.5 This Code does not govern design and installation of portions of concrete piles and drilled piers embedded in ground.

1.1.6 This Code does not govern design and construction of soil-supported slabs, unless the slab transmits vertical loads 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 this Code.

1.1.7.2 This Code 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 Code, where applicable.

1.1.8 Special provisions for earthquake resistance—Provisions of Chapter 21 shall be satisfied. See 21.2.1.

1.2—Drawings, specifications, and calculations

1.2.1 Copies of structural drawings, typical details, and specifications for all reinforced concrete construction shall be signed by a licensed engineer. These drawings (including supplementary drawings to generate the as-built condition), typical details, and specifications shall be retained by the Owner, or his designee, as a permanent record for the life of the structure. As a minimum, these drawings, details, and specifications together shall show:

(a) Name and date of issue of code and supplement to which the design conforms;

(b) Live load and other loads used in the 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 and reinforcement;

(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 welded splices and mechanical connections of reinforcement; and

(j) Details and locations of all construction or isolation joints.

1.2.2 Calculations pertinent to the design and the basis of design (including the results of model analysis, if any) shall be retained by the Owner or his or her designee, as a permanent record for the life of the structure. Accompanying these calculations shall be a statement of the applicable design and analysis methods. When computer programs are used, design assumptions and identified input and output data may be retained in lieu of calculations. Model analysis shall be permitted to supplement calculations.

1.3—Inspection and record keeping

1.3.1 The Owner is responsible for the inspection of concrete construction throughout all work stages. The Owner shall require compliance with design drawings and specifications. The Owner shall also keep records required for quality assurance and traceability of construction, fabrication, material procurement, manufacture, or installation.

1.3.2 The Owner shall be responsible for designating the records to be maintained and the duration of retention. Records pertinent to plant modifications or revisions, in-service inspections, and durability and performance of structures shall be maintained for the life of the plant. The Owner shall be responsible for continued maintenance of the records. The records shall be maintained at the power plant site, or at other locations as determined by the Owner. As a minimum, the following installation/construction records shall be considered for lifetime retention:

(a) Check-off sheets for tendon and reinforcing steel installation;

(b) Concrete cylinder test reports and charts;

(c) Concrete design mix reports;

(d) Concrete placement records;

(e) Sequence of erection and connection of precast members;

(f) Reports for construction and removal of forms and reshoring;

(g) Material property reports on reinforcing steel;

(h) Material property reports on reinforcing steel mechanical connection material;

(i) Material property reports on steel embedments in concrete;

(j) Material property reports on tendon and anchorage fabrication material and corrosion inhibitors;

(k) Reports for periodic tendon inspection;

(l) Tensioning of prestressing tendons; and

(m) Quality and proportions of concrete materials.

1.4—Approval of special systems of design or construction

Sponsors of any system of design or construction within the scope of this Code, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by this Code, shall have the right to present the data on which their design is based to the Regulatory Authority for review and approval. The Regulatory Authority may investigate the data so submitted, and may require tests and formulate rules governing the design and construction of such systems to meet the intent of this Code.

1.5—Quality assurance program

A quality assurance program covering nuclear safety related structures shall be developed prior to starting any work. The general requirements and guidelines for establishing and executing the quality assurance program during the design and construction phases of nuclear power generating stations are established by Title 10 of the Code of Federal Regulations, Part 50 (10CFR50), Appendix B.

CHAPTER 2—DEFINITIONS

2.1 The following terms are defined for general use in this Code. 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.

Anchorage—In post-tensioning, a device used to anchor tendon to concrete member; in pretensioning, a device used to anchor tendon during hardening of concrete.

Bonded tendon—Prestressing tendon that is bonded to concrete either directly or through grouting.

Cementitious materials—Materials as specified in Chapter 3 that 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 of 3 or greater used primarily to support axial compressive load.

Composite concrete flexural members—Concrete flexural members of precast and/or cast-in-place concrete elements 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.

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 in accordance with provisions of Chapter 5, expressed in pounds per square inch (psi). Whenever the quantity f_c' is under a radical sign, square root of numerical value only is intended, and the result has units of psi.

Contraction joint—Formed, sawed, or tooled groove in a concrete structure used to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure.

Creep—Stress-induced, time-temperature dependent strain.

Curvature friction—Friction resulting from bends or curves in the specified prestressing tendon profile.

Deformed reinforcement—Deformed reinforcing bars, bar and rod mats, deformed wire, welded smooth 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.

Effective depth of section (d)—Distance measured from extreme compression fiber to centroid of tension reinforcement.

Effective prestress—Stress remaining in prestressing tendons after all losses have occurred excluding effects of dead load and superimposed load.

Embedment—A steel component embedded in the concrete to transmit applied loads to the concrete structure. The embedment can be fabricated of plates, shapes, fasteners, reinforcing bars, shear connectors, inserts, or any combination thereof.

Embedment length—Length of embedded reinforcement provided beyond a critical section.

Engineer—The licensed professional engineer, employed by the Owner-contracted design authority or other agency, responsible for issuing design drawings, specifications, or other documents.

Evaluation—An engineering review of an existing safety related concrete structure with the purpose of determining physical condition and functionality. This review may include analysis, condition surveys, maintenance, testing, and repair.

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 so as to interfere least with the performance of the structure, yet 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 tendons.

Load, dead—Dead weight supported by a member (without load factors).

Load, factored—Load, multiplied by appropriate load factors, used to proportion members by the strength design method of this code. See 8.1 and 9.2.

Load, live—Live load specified by the engineer (without load factors).

Load, sustained—Dead load and the portions of other normal loads in 9.1.1 which are expected to act for a sufficient period of time to cause time-dependent effects.

Massive concrete—Mass of concrete of sufficient dimensions to produce excessive temperatures due to heat of hydration unless special precautions are taken regarding concrete placement temperatures, placing rate, or heat removal. Portions of the structure to be treated as massive concrete shall be so identified on the design drawings or specifications.

Modulus of elasticity—Ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material. See 8.5.

Net tensile strain—The tensile strain at nominal strength exclusive of strains due to effective prestress, creep, shrinkage, and temperature.

Operating basis earthquake—The operating basis earthquake (OBE) for a reactor site is that which produces the vibratory ground motion for which those features of the nuclear plant necessary for continued operation without undue risk to the health and safety of the public are designed to remain func-

tional. The OBE is only associated with plant shutdown and inspection unless selected by the Owner as a design input. See Appendix S of 10CFR50 of the Federal Regulation.

Operating basis wind—Wind velocities and forces required for the design of a structure in accordance with ASCE 7-95 for a 100 year recurrence interval.

Owner—The organization responsible for the operation, maintenance, safety, and power generation of the nuclear power plant.

Pedestal—Upright compression member with a ratio of unsupported height to average least lateral dimension of less than 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 tendons are 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.

Pretensioning—Method of prestressing in which tendons are tensioned before concrete is placed.

Regulatory Authority—The governmental agency or agencies having legal jurisdiction over the design, construction, and operation of nuclear power generating stations to assure public health and safety.

Reinforced concrete—Concrete containing adequate reinforcement, prestressed or nonprestressed, and designed on the assumption that the two materials act together in resisting forces.

Reinforcement—Material that conforms to 3.5, excluding prestressing tendons unless specifically included.

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.

Safe shutdown earthquake—The safe shutdown earthquake ground motion (SSE) is the vibratory ground motion for which certain structures, systems, and components (SSCs) in nuclear power plants must be designed to remain functional. For the definition of these SSCs, see Appendix S of 10CFR50 of the Federal Regulation.

Shores—Vertical or inclined support members designed to carry the weight of the formwork, concrete, and construction loads above.

Shrinkage—Time-temperature-humidity dependent volume reduction of concrete as a result of hydration, moisture migration, and drying process.

Span length—See 8.7.

Spiral reinforcement—Continuously wound reinforcement in the form of a cylindrical helix.

Stirrup—Reinforcement used to resist shear and torsion stresses in a structural member; typically bars, wires, or welded wire fabric (plain or deformed) 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 **f**. 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 this code 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 this code. See 9.1.1.

Stress—Intensity of force per unit area.

Stress relaxation—A phenomenon in which loss of stress occurs when a constant strain is maintained at a constant temperature.

Tendon—Steel element such as wire, cable, bar, rod, or strand, or a bundle of such elements, used to impart prestress to concrete.

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 reentrant corners is acceptable. See also stirrup.

Transfer—Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member.

Unbonded tendons—Tendons in which the prestressing steel is permanently free to move relative to the surrounding concrete to which they are applying their prestressing forces.

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 in pounds per square inch. Yield strength or yield point is determined in tension according to applicable ASTM specifications as modified by 3.5 of this Code.

PART 2—STANDARDS FOR TESTS AND MATERIALS

CHAPTER 3—MATERIALS

3.0—Notation
f_y = specified yield strength of nonprestressed reinforcement, psi

3.1—Tests of materials
3.1.1 The Owner 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 available for inspection as required by 1.3.2.

3.2—Cements
3.2.1 Cement shall conform to one of the following specifications for portland cement:

(a) "Specification for Portland Cement" (ASTM C 150); or

(b) "Specification for Blended Hydraulic Cements" (ASTM C 595), excluding Types S and SA which are not intended as principal cementing constituents of structural concrete; or

(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.

3.2.3 Every shipment of cement shall be accompanied by a certified mill test report stating the results of tests representing the cement in shipment and the ASTM specification limits for each item of required chemical, physical, and optional characteristics. No cement shall be used in any structural concrete prior to receipt of 7 day mill test strengths.

3.3—Aggregates
3.3.1 Concrete aggregates shall conform to one of the following specifications:

(a) "Specification for Concrete Aggregates" (ASTM C 33); or

(b) "Specification for Aggregates for Radiation-Shielding Concrete" (ASTM C 637).

Exception: Aggregates failing to meet ASTM C 33 but which have been shown by special test or actual service to produce concrete of adequate strength and durability shall be permitted to be used for normal-weight concrete where authorized by the engineer.

3.3.2 Nominal maximum size of coarse aggregate shall not be 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, or prestressing tendons or ducts.

These limitations may be waived if, in the judgment of the engineer, workability, and methods of consolidation are such that concrete can be placed without honeycomb or voids.

3.3.3—Testing requirements
3.3.3.1 Tests for full conformance with the appropriate specification, including tests for potential reactivity, shall be performed prior to usage in construction unless such tests are specifically exempted by the specifications as not being applicable.

3.3.3.2 A daily inspection control program shall be carried out during concrete production to determine and control consistency in potentially variable characteristics such as water content, gradation, and material finer than No. 200 sieve.

3.3.3.3 Tests for conformance with ASTM C 131, ASTM C 289, and ASTM C 88 shall be repeated whenever there is reason to suspect a change in the basic geology or mineralogy of the aggregates.

3.4—Water
3.4.1 Water used in mixing concrete shall be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances that may be deleterious to concrete or reinforcement.

3.4.2 Mixing water for prestressed 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 4.3.1.

3.4.3 Nonpotable water shall not be used in concrete unless the following are satisfied:

(a) Selection of concrete proportions shall be based on concrete mixes using water from the same source.

(b) Mortar test cubes made with nonpotable mixing water shall have 7-day and 28-day strengths equal to at least 90% 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 "Method of Test for Compressive Strength of Hydraulic Cement Mortars (Using 2-inch or 50-mm Cube Specimens)" (ASTM C 109).

3.5—Steel reinforcement
3.5.1 Reinforcement shall be deformed reinforcement, except that plain reinforcement may be used for spirals or tendons; and reinforcement consisting of structural steel, steel pipe, or steel tubing shall be permitted as specified in this code.

3.5.2 Welding of reinforcing bars shall conform to "Structural Welding Code—Reinforcing Steel," ANSI/AWS D1.4 of the American Welding Society. Type and location of welded splices and other required welding of reinforcing bars shall be indicated on the design drawings or in the project specifications. ASTM reinforcing bar specifications,

except for ASTM A 706, shall be supplemented to require a report of material properties necessary to conform to the requirements in ANSI/AWS D1.4.

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 615).

(b) "Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement" (ASTM A 706).

3.5.3.1.1 A minimum of one tensile test shall be required for each 50 tons of each bar size produced from each heat of steel.

3.5.3.2 Specified yield strength f_y for deformed reinforcing bars shall not exceed 60,000 psi.

3.5.3.3 Bar mats for concrete reinforcement shall conform to "Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement" (ASTM A 184). Reinforcement used in bar mats shall conform to one of the specifications listed in 3.5.3.1.

3.5.3.4 Deformed wire for concrete reinforcement shall conform to "Specification for Deformed Steel Wire for Concrete Reinforcement" (ASTM A 496), except that wire shall not be smaller than size D4.

3.5.3.5 Welded plain wire fabric for concrete reinforcement shall conform to "Specification for Welded Steel Wire Fabric for Concrete Reinforcement" (ASTM A 185). Welded intersections shall not be spaced farther apart than 12 in. in direction of calculated stress, except for wire fabric used as stirrups in accordance with 12.13.2.

3.5.3.6 Welded deformed wire fabric for concrete reinforcement shall conform to "Specification for Welded Deformed Steel Wire Fabric for Concrete Reinforcement" (ASTM A 497). Welded intersections shall not be spaced farther apart than 16 in. in direction of calculated stress, except for wire fabric used as stirrups in accordance with 12.13.2.

3.5.3.7 (This section not used to maintain section number correspondence with ACI 318-95).

3.5.3.8 Epoxy-coated reinforcing bars shall comply with "Specification for Epoxy Coated Reinforcing Steel Bars" (ASTM A 775) or with "Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars" (ASTM A 934). The engineer shall evaluate the suitability of coated reinforcing steel for the expected service environment in each application. Epoxy-coated reinforcing steel shall also conform to one of the specifications listed in 3.5.3.1.

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) including additional requirements of 3.5.3.1.1.

3.5.4.2 Smooth wire for spiral reinforcement shall conform to "Specification for Cold-Drawn Steel Wire for Concrete Reinforcement" (ASTM A 82).

3.5.5—Prestressing tendons

3.5.5.1 Tendons for prestressed reinforcement shall conform to one of the following specifications:

(a) Wire conforming to "Specification for Uncoated Stress-Relieved 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 Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete" (ASTM A 416).

(d) Bars conforming to "Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete" (ASTM A 722).

3.5.5.2 Wire, strands, and bars not specifically listed in ASTM A 421, A 416, or A 722 are permitted 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 416, 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 10.14.7 or 10.14.8 shall conform to one of the following specifications:

(a) "Specification for Structural Steel" (ASTM A 36).

(b) "Specification for High-Strength Low-Alloy Structural Steel" (ASTM A 242).

(c) "Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality" (ASTM A 572).

(d) "Specification for High-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick" (ASTM A 588).

3.5.6.2 Steel pipe or tubing for composite compression members composed of a steel encased concrete core meeting requirements of 10.14.6 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).

3.6—Admixtures

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 metal forms. See 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 Pozzolans for Use 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 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 "Specification for Silica Fume for Use in Hydraulic-Cement Concrete and Mortar" (ASTM C 1240).

3.6.10—Testing

3.6.10.1 Tests for compliance with the specification for each admixture shall be required prior to initial shipment and acceptance on site for usage in construction.

3.6.10.2 An infrared spectrum trace of the conformance test sample of air-entraining and water-reducing admixture shall be furnished with the conformance test results.

3.7—Storage and identification of materials

3.7.1 Measures shall be established to provide for storage of all materials so as to prevent damage or deterioration. When necessary for particular products, special protective environments such as inert gas atmosphere, specific moisture content levels, and control temperatures shall be provided.

All stored materials shall be properly tagged or labeled to permit identification.

3.7.2 Cementitious materials and aggregate shall be stored in such a manner as to prevent deterioration or intrusion of foreign matter. Any material that has deteriorated or has been contaminated shall not be used for concrete.

3.7.3 Reinforcing material shall be stored in such a manner as to permit inventory control and to preclude damage or degradation of properties to less than ASTM Reinforcement requirements.

Reinforcing steel, by groups of bars or shipments, shall be identifiable by documentation, tags, or other means of control, to a specific heat number or heat code until review of the Certified Material Test Report has been performed.

3.7.4 Prestressing system materials shall be stored in such a manner as to ensure traceability to the Certified Material Test Report during production and while in transit and storage.

3.8—Standards cited in this Code

3.8.1 Standards of the American Society for Testing and Materials referred to in this Code are listed below with their serial designations, including year of adoption or revision, and are declared to be part of this Code as if fully set forth herein.

A 36-94	Standard Specification for Structural Steel
A 53-93a	Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless
A 82-94	Standard Specification for Cold-Drawn Steel Wire for Concrete Reinforcement
A 108-99	Standard Specification for Steel Bars, Carbon, Cold-Finished, Standard Quality
A 184-90	Standard Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement
A 185-94	Standard Specification for Welded Steel Wire Fabric for Concrete Reinforcement
A242-93a	Standard Specification for High-Strength Low-Alloy Structural Steel
A 416-94	Standard Specification for Uncoated Seven-Wire Stress-Relieved Steel Strand for Prestressed Concrete
A 421-91	Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
A 496-94	Standard Specification for Deformed Steel Wire for Concrete Reinforcement
A 497-94a	Standard Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement
A 500-93	Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
A 501-93	Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
A 572-94b	Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
A 588-94	Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in. (100 mm) Thick
A 615-94	Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A 706-92b	Standard Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement
A 722-90	Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete
A 775-94d	Standard Specification for Epoxy-Coated Reinforcing Steel Bars
A 884-94a	Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement
A 934-95	Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars
C 31-91	Standard Method of Making and Curing Concrete Test Specimens in the Field
C 33-93	Standard Specification for Concrete Aggregates
C 39-93a	Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens
C 42-90	Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 88-76	Standard Method of Test for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate
C 94-94	Standard Specification for Ready-Mixed Concrete

C 109-93   Standard Method of Test for Compressive Strength of Hydraulic Cement Mortars (Using 2-inch or 50-mm Cube Specimens)

C 131-81   Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine

C 144-93   Standard Specification for Aggregate for Masonry Mortar

C 150-94   Standard Specification for Portland Cement

C 172-90   Standard Method of Sampling Fresh Concrete

C 192-90a Standard Method of Making and Curing Concrete Test Specimens in the Laboratory

C 260-94   Standard Specification for Air-Entraining Admixtures for Concrete

C 289-81   Standard Method of Test for Potential Reactivity of Aggregates (Chemical Method)

C 494-92   Standard Specification for Chemical Admixtures for Concrete

C 595-94a Standard Specification for Blended Hydraulic Cements

C 597-83   Standard Test Method for Pulse Velocity through (1991)   Concrete

C 618-94a Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete

C 637-73   Standard Specification for Aggregates for Radiation-Shielding Concrete

C 685-94   Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing

C 845-90   Standard Specification for Expansive Hydraulic Cement

C 989-93   Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars

C 1017-92 Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete

C 1218-92 Standard Test Method for Water Soluble Chloride E1   in Mortar and Concrete

C 1240-93 Standard Specification for Silica Fume for Use in Hydraulic-Cement Concrete and Mortar

3.8.2 Requirements of the American Welding Society referred to in this Code are listed below. Where applicable, they shall be considered a part of this Code the same as if fully set forth elsewhere herein.

"Structural Welding Code—Steel" (AWS D1.1:2000) of the American Welding Society.

"Structural Welding Code—Reinforcing Steel" (ANSI/AWS D1.4-98) of the American Welding Society.

3.8.3 Requirements of the United States Nuclear Regulatory Commission referred to in this Code are listed below. Where applicable, they shall be considered a part of this Code the same as if fully set forth elsewhere herein.

Code of Federal Regulations (Published by Office of the Federal Register)

10 CFR50   Domestic Licensing of Production and Utilization Facilities, Appendix B—Quality Assurance Requirements for Nuclear Power Plants and Fuel Reprocessing Plants

10 CFR100 Reactor Site Criteria, Appendix A—Seismic and Geologic Siting Criteria

3.8.4 "Specification for Unbonded Single Strand Tendons," July 1993, of the Post-Tensioning Institute is declared to be part of this Code as if fully set forth herein.

3.8.5 ASCE 7-95, "Minimum Design Loads for Buildings and Other Structures" is declared to be part of this Code as if fully set forth herein.

PART 3—CONSTRUCTION REQUIREMENTS

CHAPTER 4—DURABILITY REQUIREMENTS

4.0—Notation
f'_c = specified compressive strength of concrete, psi

4.1—Water-cementitious materials ratio
4.1.1 The water-cementitious materials ratios specified in Tables 4.2.2 and 4.3.1 shall be calculated using the weight of cement meeting ASTM C 150, C 595, or C 845 plus the weight of fly ash and other pozzolans meeting ASTM C 618, except as noted in 5.4.2 and silica fume meeting ASTM C 1240, except as limited by 4.2.3.

4.2—Freezing and thawing exposures
4.2.1 Normal weight concrete exposed to freezing and thawing or deicing chemicals shall be air-entrained with air content indicated in Table 4.2.1. Tolerance on air content as delivered shall be ±1.5%. For specified compressive strength f'_c greater than 5000 psi, air content indicated in Table 4.2.1 may be reduced 1%.

Table 4.2.1—Total air content for frost-resistant concrete

Nominal maximum aggregate size, in.*	Air content,%	
	Severe exposure	Moderate exposure
$^3/_8$	$7^1/_2$	6
$^1/_2$	7	$5^1/_2$
$^3/_4$	6	5
1	6	$4^1/_2$
$1^1/_2$	$5^1/_2$	$4^1/_2$
$2^\dagger$	5	4
$3^\dagger$	$4^1/_2$	$3^1/_2$

* See ASTM C 33 for tolerance on oversize for various nominal maximum size designations.

† These air contents apply to total mix, as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than $1^1/_2$ in. is removed by handpicking or sieving and air content is determined on the minus $1^1/_2$ in. fraction of mix (tolerance on air content as delivered applies to this value). Air content of total mix is computed from value determined on the minus $1^1/_2$ in. fraction.

4.2.2 Concrete that will be subject to the exposures given in Table 4.2.2 shall conform to the corresponding maximum water-cementitious materials ratios and minimum specified concrete compressive strength requirements of that table. In addition, concrete that will be exposed to deicing chemicals shall conform to the limitations of 4.2.3.

4.2.3 For concrete exposed to deicing chemicals, the maximum weight of fly ash, other pozzolans, silica fume, or slag that is included in the concrete shall not exceed the

Table 4.2.2—Requirements for Special Exposure Conditions

Exposure Condition	Maximum water-cementitious materials ratio, by weight, normal weight aggregate concrete	Minimum f'_c normal weight aggregate concrete, psi
Concrete intended to have low permeability when exposed to water	0.50	4000
Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals	0.45	4500
For corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources.	0.40	5000

percentages of the total weight of cementitious materials given in Table 4.2.3.

Table 4.2.3—Requirements for concrete exposed to deicing chemicals

Cementitious materials	Maximum % of total cementitious materials by weight*
Fly ash or other pozzolans conforming to ASTM C 618	25
Slag conforming to ASTM C 989	50
Silica fume conforming to ASTM C 1240	10
Total of fly ash or other pozzolans, slag, and silica fume	$50^\dagger$
Total of fly ash or other pozzolans and silica fume	$35^\dagger$

*The total cementitious material also includes ASTM C 150, C 595, and C 845 cement.
The maximum percentages above shall include:
(a) Fly ash or other pozzolans present in Type IP or I(PM) blended cement, ASTM C 595;
(b) Slag used in the manufacture of a IS or I(SM) blended cement, ASTM C 595; and
(c) Silica fume, ASTM C 1240, present in a blended cement.
†Fly ash or other pozzolans and silica fume shall constitute no more than 25 and 10%, respectively, of the total weight of the cementitious materials.

4.3—Sulfate exposures
4.3.1 Concrete to be exposed to sulfate-containing solutions or soils shall conform to requirements of Table 4.3.1 or shall be concrete made with a cement that provides sulfate resistance and that has a maximum water-cementitious materials ratio and minimum compressive strength from Table 4.3.1.

4.3.2 Calcium chloride as an admixture shall not be used in concrete to be exposed to severe or very severe sulfate-containing solutions, as defined in Table 4.3.1.

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 ingredients including water, aggregates, cementi-

tious materials, and admixtures shall not exceed the limits of Table 4.4.1. When testing is performed to determine 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, % by weight of cement
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15

4.4.2 When reinforced concrete will be exposed to deicing chemicals, salts, brackish water, seawater, or spray from these sources, requirements of Table 4.2.2 for water-cementitious materials ratio and concrete strength, and the minimum concrete cover requirements of 7.7 shall be satisfied. See 18.14 for unbonded prestressing tendons.

CHAPTER 5—CONCRETE QUALITY, MIXING, AND PLACING

5.0—Notation
f'_c = specified compressive strength of concrete, psi

f'_{cr} = required average compressive strength of concrete used as the basis for selection of concrete proportions, psi

s = standard deviation, psi

5.1—General
5.1.1 Concrete shall be proportioned to provide an average compressive strength as prescribed in 5.3.2 as well as satisfy the durability criteria of Chapter 4. Concrete shall be produced to minimize frequency of strengths below f'_c as prescribed in 5.6.2.3.

5.1.2 Requirements for f'_c shall be based on tests of cylinders made and tested as prescribed in 5.6.2.

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 Splitting tensile strength tests shall not be used as a basis for field acceptance of concrete.

5.1.5 Design drawings shall show specified compressive strength of concrete f'_c for which each part of the structure is designed.

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; and

(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, including water-cementitious materials ratio, shall be established on the basis of field experience and/or trial mixtures with materials to be employed (Section 5.3), except as permitted in 5.4 or required by Chapter 4.

5.3—Proportioning on the basis of field experience and/or trial mixtures
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 1000 psi of that specified for proposed work;

Table 4.3.1—Requirements for concrete exposed to sulfate-containing solutions

Sulfate exposure	Water soluble sulfate (SO₄) in soil,% by weight	Sulfate (SO₄) in water, ppm	Cement type	Maximum water-cementitious materials ratio, by weight, normal weight aggregate concrete*	Minimum f_c', normal weight aggregate concrete, psi*
Negligible	0.00-0.10	0-150	—	—	—
Moderate†	0.10-0.20	150-1500	II, IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS)	0.50	4000
Severe	0.20-2.00	1500-10,000	V	0.45	4500
Very severe	Over 2.00	Over 10,000	V plus pozzolan‡	0.45	4500

* A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing (Table 4.2.2).

† Seawater.

‡ Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

(c) Shall consist of at least 30 consecutive tests or two groups of consecutive tests totaling at least 30 tests as defined in 5.6.1.4, except as provided in 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 5.3.1.2. To be acceptable, test record shall meet requirements (a) and (b) of 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 the larger of Eq. (5-1) or (5-2) using a standard deviation calculated in accordance with 5.3.1.1 or 5.3.1.2.

$$f_{cr}' = f_c' + 1.34s \qquad (5\text{-}1)$$

or

$$f_{cr}' = f_c' + 2.33s - 500 \qquad (5\text{-}2)$$

5.3.2.2 When a concrete production facility does not have field strength test records for calculation of standard deviation meeting requirements of 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', psi	Required average compressive strength, f_{cr}', psi
Less than 3000 psi	$f_c' + 1000$
3000 to 5000	$f_c' + 1200$
Over 5000	$f_c' + 1400$

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 (Section 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}' (Section 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 may 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 ± 0.75 in. of maximum permitted, and for air-entrained concrete, within ± 0.5% 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; and

(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.

5.4—Proportioning by water-cementitious materials ratio

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 engineer. The required average compressive strength f_c' of concrete produced with materials similar to those proposed for use shall be at least 1200 psi greater than the specified compressive

strength f_c'. This alternative shall not be used for specified compressive strength greater than 4000 psi.

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.

5.5—Average strength reduction

As data become available during construction, it shall be permitted to reduce the amount by which f_c' must exceed specified value of f_c' provided:

(a) 30 or more test results are available and average of test results exceeds that required by 5.3.2.1, using a standard deviation calculated in accordance with 5.3.1.1; or

(b) 15 to 29 test results are available and average of test results exceeds that required by 5.3.2.1 using a standard deviation calculated in accordance with 5.3.1.2; and

(c) special exposure requirements of Chapter 4 are met.

5.6—Evaluation and acceptance of concrete

5.6.1—Frequency of testing

5.6.1.1 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 150 yd^3 of concrete, nor less than once for each 5000 ft^2 of surface area for slabs or walls.

5.6.1.2 On a given project, if total volume of concrete is such that frequency of testing required by 5.6.1.1 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.

5.6.1.3 When total quantity of a given class of concrete is less than 50 yd^3, strength tests may be waived by the engineer if the engineer has been provided adequate evidence of satisfactory strength.

5.6.1.4 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'.

5.6.2—Laboratory-cured specimens

5.6.2.1 Samples for strength tests shall be taken in accordance with "Method of Sampling Freshly Mixed Concrete" (ASTM C 172).

5.6.2.2 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).

5.6.2.3 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'; and

(b) No individual strength test (average of two cylinders) falls below f_c' by more than 500 psi.

5.6.2.4 If either of the requirements of 5.6.2.3 are not met, steps shall be taken to increase the average of subsequent strength test results. Requirements of 5.6.4 shall be observed if requirement of 5.6.2.3(b) is not met.

5.6.3—Field-cured specimens

5.6.3.1 The engineer may require strength tests of cylinders cured under field conditions to check the adequacy of curing and protection of concrete in the structure. The engineer may use non-destructive testing to confirm the accuracy of strength testing completed on field-cured specimens.

5.6.3.2 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).

5.6.3.3 Field-cured test cylinders shall be molded at the same time and from the same samples as laboratory-cured test cylinders.

5.6.3.4 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% of that of companion laboratory-cured cylinders. The 85% limitation shall not apply if field-cured strength exceeds f_c' by more than 500 psi.

5.6.4—Investigation of low-strength test results

5.6.4.1 If any strength test (Section 5.6.1.4) of laboratory-cured cylinders falls below specified value of f_c' by more than 500 psi (Section 5.6.2.3(b)] or if tests of field-cured cylinders indicate deficiencies in protection and curing (Section 5.6.3.4), steps shall be taken to assure that load-carrying capacity of the structure is not jeopardized.

5.6.4.2 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 42) shall be permitted. In such cases, three cores shall be taken for each strength test more than 500 psi below the specified value of f_c'.

5.6.4.3 If concrete in the structure will be dry under service conditions, cores shall be air dried (temperature 60 to 80 F, relative humidity less than 60%) for 7 days before test and shall be tested dry. If concrete in the structure will be more than superficially wet under service conditions, cores shall be immersed in water for at least 40 hr and be tested wet.

5.6.4.4 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% of f_c' and if no single core is less than 75% of f_c'. Additional testing of cores extracted from locations represented by erratic core strength results shall be permitted within limits established by the engineer.

5.6.4.5 If the criteria of 5.6.4.4 are met, and if structural adequacy remains in doubt, the engineer may order load tests as outlined in Chapter 20 to further assess adequacy or may take other appropriate action.

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 and ice shall be removed from spaces to be occupied by concrete;

(c) Forms shall be properly coated;

(d) Masonry filler units that will be in contact with concrete shall be well drenched;

(e) Reinforcement shall be thoroughly cleaned of ice or other deleterious coatings;

(f) Water shall be removed from place of deposit before concrete is placed unless a tremie is to be used or it shall be displaced by methods that shall exclude incorporation of additional water in the concrete during placement and consolidation; and

(g) Laitance and other unsound material shall be removed before additional concrete is placed against hardened concrete. The method for cleaning joints shall be stated in the specification.

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 type approved by the engineer;

(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); and

(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; and

(4) time and date of mixing and placing.

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.

5.9.3 Aluminum pipe shall not be used to convey concrete.

5.10—Depositing

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 Retempered concrete shall not be used.

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.

5.10.9 Where conditions make consolidation difficult, or where reinforcement is congested, batches may be reproportioned to exclude the larger of the coarse aggregate gradations. Where the coarse aggregate is furnished in only one gradation, batches of mortar containing approximately the same proportions of cement, sand, and water may be used. Such substitutions shall be limited to only those made in limited areas of specific difficulty and subject to the approval of the engineer as to location, mix proportioning, or alteration of this mix.

5.11—Curing

5.11.1 Concrete (other than high-early-strength) shall be maintained above 50 F 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 High-early-strength concrete shall be maintained above 50 F and in a moist condition for at least the first 3 days, except when cured in accordance with 5.11.3.

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, supplementary strength tests in accordance with 5.6.3 shall be performed to assure that curing is satisfactory.

5.11.5 Where a liquid membrane curing compound is used, particular attention shall be given to its compatibility with any protective coatings that are to be applied following curing efforts.

5.11.6 The method of curing shall be stated in the construction specifications.

5.12—Cold weather requirements

5.12.1 Adequate equipment shall be provided for heating concrete materials and protecting concrete during freezing or near-freezing weather.

5.12.2 All concrete materials and all reinforcement, forms, fillers, and ground with which concrete is to come in contact shall be free from frost.

5.12.3 Frozen materials or materials containing ice shall not be used.

5.13—Hot weather requirements

5.13.1 During hot weather, proper attention shall be given to ingredients, production methods, handling, placing, protection, and curing to prevent excessive concrete temperatures or water evaporation that could impair required strength or serviceability of the member or structure.

5.13.2 The method of controlling concrete temperatures shall be specified in the construction specification.

CHAPTER 6—FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION JOINTS

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; and

(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.

6.1.7 When using steel liners as formwork, special attention shall be given:

6.1.7.1 To liner supports to provide the required tolerances for penetrations.

6.1.7.2 To the depth of placement in order to limit the deformation of the liner.

6.1.8 Where coating systems are to be applied to the concrete, formwork shall be compatible with the coating system.

6.2—Removal of forms and shores

6.2.1 Forms shall be removed in such a manner as not to impair safety and serviceability of the structure. Concrete to be exposed by form removal shall have sufficient strength not to be damaged by removal operation.

6.2.2 The provisions of 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 engineer 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 engineer, 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.

6.2.3 Where coating systems are to be applied to the concrete, only those hardeners, additives, and form release agents that are compatible with the coating system shall be used.

6.3—Conduits, pipes, and sleeves 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 defined in 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% 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 the following:

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 be permitted to be considered as replacing structurally in compression the displaced concrete provided:

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 2 in. 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, temperature to which they will be subjected.

6.3.8 All piping and fittings except as provided in 6.3.8.1 shall be tested as a unit for leaks before concrete placement. Pressure tests shall be in accordance with the applicable piping code or standard. Where pressure testing requirements are not specified in a code or standard, pressure testing shall meet the following requirements: (1) The testing pressure above atmospheric pressure shall be 50% in excess of pressure to which piping and fittings may be subjected, but minimum testing pressure shall not be less than 150 psi above atmospheric pressure. (2) The test pressure shall be held for 4 hours with no drop in pressure allowed, except that which may be caused by a drop in air temperature.

6.3.8.1 Drain pipes and other piping designed for pressures of not more than 1 psi above atmospheric pressure need not be tested as required in 6.3.8.

6.3.8.2 Pipes carrying liquid, gas, or vapor that is explosive or injurious to health shall again be tested as specified in 6.3.8 after the concrete has reached its required 28-day strength.

6.3.9 No liquid, gas, or vapor, except water not exceeding 90 F nor 50 psi pressure, shall be placed in the pipes until the concrete has attained its design strength, unless otherwise approved by the engineer.

6.3.10 In solid slabs the piping, unless it is for radiant heating or snow melting, shall be placed between top and bottom reinforcement.

6.3.11 Concrete cover for pipes, conduits, and fittings shall not be less than 1-1/2 in. for concrete exposed to earth or weather, nor 3/4 in. for concrete not exposed to weather or in contact with ground.

6.3.12 Reinforcement with an area not less than 0.002 times the area of concrete section shall be provided normal to piping.

6.3.13 Piping and fittings shall be assembled according to the construction specifications. Screw connections shall be prohibited.

6.3.14 Piping and conduit shall be so fabricated and installed that cutting, bending, or displacement of reinforcement from its specified location, beyond the limitations of 7.5.2.3, will not be required.

6.3.15 All piping containing liquid, gas, or vapor pressure in excess of 200 psi above atmospheric pressure or temperature in excess of 150 F shall be sleeved, insulated, or otherwise separated from the concrete and/or cooled to limit concrete stresses to allowable design values and to limit concrete temperatures to the following:

(a) For normal operation or any other long-term period, the temperatures shall not exceed 150 F, except for local areas which are allowed to have increased temperatures not to exceed 200 F.

(b) For accident or any other short-term period, the temperatures shall not exceed 350 F for the interior surface. However, local areas are allowed to reach 650 F from fluid jets in the event of a pipe failure.

(c) Higher temperatures than given in Items (a) and (b) may be allowed in the concrete if tests are provided to evaluate the reduction in strength and this reduction is applied to the design allowables. Evidence shall also be provided which verifies that the increased temperatures do not cause deterioration of the concrete either with or without load.

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. All construction joints shall be indicated on the design drawings or shall be approved by the engineer. Provision shall be made for transfer of shear and other forces through construction joints. See 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

7.0—Notation

A = effective tensile area of concrete surrounding the reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars, sq in. When the reinforcement consists of several bar sizes, the number of bars shall be computed as the total steel area divided by the area of the largest bar used

$A_{s\,min}$ = minimum reinforcement for massive concrete elements (See 7.12.2)

d = distance from extreme compression fiber to centroid of tension reinforcement, in.

d_b = nominal diameter of bar, wire, or prestressing strand, in.

f_s = stress in reinforcing steel, psi

f'_t = specified tensile strength of concrete, psi.

f_y = specified yield strength of nonprestressed reinforcement, psi

ℓ_d = development length, in. (See Chapter 12)

7.1—Standard hooks

The term "standard hook" as used in this code shall mean one of the following:

7.1.1 180-degree bend plus $4d_b$ extension, but not less than 2-1/2 in. at free end of bar.

7.1.2 90-degree bend plus $12d_b$ extension at free end of bar.

7.1.3 For stirrup and tie hooks*

(a) No. 5 bar and smaller, 90-degree bend plus $6d_b$ extension at free end of bar; or

(b) No. 6, 7, and 8 bar, 90-degree bend plus $12d_b$ extension at free end of bar; or

(c) No. 8 bar and smaller, 135-degree bend plus $6d_b$ extension at free end of bar.

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 No. 3 through No. 5, shall not be less than the values in Table 7.2.

Table 7.2—Minimum diameters of bend

Bar size	Minimum diameter
No. 3 through No. 8	$6d_b$
No. 9, No. 10, and No. 11	$8d_b$
No. 14 and No. 18	$10d_b$

7.2.2 Inside diameter of bends for stirrups and ties shall not be less than $4d_b$ for No. 5 bar and smaller. For bars larger than No. 5, diameter of bend shall be in accordance with Table 7.2.

7.2.3 Inside diameter of bends in welded wire fabric (smooth or deformed) for stirrups and ties shall not be less than $4d_b$ for deformed wire larger than D6 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.

7.3—Bending

7.3.1 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.

7.4—Surface conditions of reinforcement

7.4.1 At time concrete is placed, reinforcement shall be free from mud, oil, or other nonmetallic coatings that de-

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* For closed ties and continuously wound ties defined as hoops in Chapter 21, a 135-degree bend plus an extension of at least $6d_b$ but not less than 75 mm. (See definition of "hoop" in 21.1.)

crease bond. Epoxy coatings of bars, in accordance with standards in this code, shall be permitted if the coating is qualified for service conditions (i.e., temperature and radiation), as well as fabrication conditions (i.e., damaged epoxy coatings shall be repaired).

7.4.2 Reinforcement, except prestressing tendons, 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 are not less than applicable ASTM specification requirements.

7.4.3 Prestressing tendons shall be clean and free of oil, dirt, scale, pitting, and excessive rust. A light oxide shall be permitted.

7.5—Placing reinforcement

7.5.1 Reinforcement, prestressing tendons, and ducts shall be accurately placed and adequately supported before concrete is placed, and shall be secured against displacement within tolerances permitted in 7.5.2.

7.5.2 Unless otherwise specified by the engineer, reinforcement, prestressing tendons, and prestressing ducts shall be placed within the following tolerances:

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
$d \le 8$ in.	$\pm\,^3/_8$ in.	$-\,^3/_8$ in.
$d \le 24$ in.	$\pm\,^1/_2$ in.	$-\,^1/_2$ in.
$d > 24$ in.	$\pm\,1$ in.	$-\,^1/_2$ in.

Except that tolerance for the clear distance to formed soffits shall be minus 1/4 in. and tolerance for cover shall not exceed minus 1/3 the minimum concrete cover required in the design drawings or in the specifications.

7.5.2.2 Tolerance for longitudinal location of bends and ends of reinforcement shall be $\pm\,2$ in. except at discontinuous ends of members where tolerance shall be $\pm\,1/2$ in.

7.5.3 Welded wire fabric (with wire size not greater than W5 or D5) used in slabs not exceeding 10 ft 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 midspan, 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.

7.5.5 Bars may be moved as necessary to avoid interference with other reinforcing steel, conduits, or embedded items subject to the approval of the engineer. If bars are moved more than one bar diameter, or enough to exceed the above tolerances, the resulting arrangement of bars shall be subject to approval by the engineer.

7.6—Spacing limits for reinforcement

7.6.1 The minimum clear spacing between parallel bars in a layer shall not be less than d_b nor 1 in. See also 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 1 in.

7.6.3 In spirally reinforced or tied reinforced compression members, clear distance between longitudinal bars shall not be less than $1.5d_b$ nor 1-1/2 in. See also 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 three times the wall or slab thickness, nor 18 in.

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 No. 11 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—Prestressing tendons and ducts

7.6.7.1 Clear distance between pretensioning tendons at each end of a member shall not be less than $4d_b$ for wire, nor $3d_b$ for strands. See also 3.3.2. Closer vertical spacing and bundling of strands 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 tendons, when tensioned, from breaking through the duct.

7.7—Concrete protection for reinforcement

7.7.1—Cast-in-place concrete (nonprestressed)

The following minimum concrete cover shall be provided for reinforcement:

Minimum cover, in.

(a) Concrete cast against and
   permanently exposed to earth 3

(b) Concrete exposed to earth or weather:

   No. 6 through No. 18 bar 2
   No. 5 bar, W31 or D31 wire, and
      smaller 1-1/2

(c) Concrete not exposed to weather or
   in contact with ground:

   Slabs, walls, joists:
   No. 14 and No. 18 bars 1-1/2
   No. 11 bar and smaller 3/4

Beams, columns:
Primary reinforcement, ties, stirrups, spirals 1-1/2

Shells, folded plate members:
   No. 6 bar and larger 3/4
   No. 5 bar, W31 or D31 wire, and smaller 1/2

7.7.2—Precast concrete (manufactured under plant control conditions)

The following minimum concrete cover shall be provided for reinforcement:

Minimum cover, in.

(a) Concrete exposed to earth or weather:

   Wall panels:
   No. 14 and No. 18 bars 1-1/2
   No. 11 bar and smaller 3/4

   Other members:
   No. 14 and No. 18 bars 2
   No. 6 through No. 11 bars 1-1/2
   No. 5 bar, W31 or D31 wire, and smaller 1-1/4

(b) Concrete not exposed to weather or
   in contact with ground:

   Slabs, walls, joists:
   No. 14 and No. 18 bars 1-1/4
   No. 11 bar and smaller 5/8

   Beams, columns:
   Primary reinforcement .... d_b
                                   but not less than 5/8
                           and need not exceed 1-1/2

   Ties, stirrups, spirals 3/8
   Shells, folded plate members:
   No. 6 bar and larger 5/8
   No. 5 bar, W31 or D31 wire, and smaller 3/8

7.7.3—Prestressed concrete

7.7.3.1 The following minimum concrete cover shall be provided for prestressed and nonprestressed reinforcement, ducts, and end fittings, except as provided in 7.7.3.2 and 7.7.3.3:

Minimum cover, in.

(a) Concrete cast against and
   permanently exposed to earth 3

(b) Concrete exposed to earth or weather:

   Wall panels, slabs, joists 1
   Other members 1-1/2

(c) Concrete not exposed to weather or
   in contact with ground:

   Slabs, walls, joists 3/4
   Beams, columns:
   Primary reinforcement 1-1/2
   Ties, stirrups, spirals 1

   Shells, folded plate members:
   No. 5 bar, W31 or D31 wire, and smaller 3/8

Other reinforcement ....d_b

but not less than 3/4

7.7.3.2 For prestressed concrete members exposed to earth, weather, or corrosive environments, and in which permissible tensile stress of 18.4.2(b) is exceeded, minimum cover shall be increased 50%.

7.7.3.3 For prestressed concrete members manufactured under plant control conditions, minimum concrete cover for nonprestressed reinforcement shall be as required in 7.7.2.

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 2 in.; except for concrete cast against and permanently exposed to earth, minimum cover shall be 3 in.

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.

7.7.6—Future extensions

Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

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 1-1/2 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 6 in. from points of bend.

7.8.1.4 Offset bars shall be bent before placement in the forms. See 7.3.

7.8.1.5 Where a column face is offset 3 in. 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% 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 may 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.

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 end anchorage of reinforcement terminating in such connections.

7.9.2 Enclosure at connections may consist of external concrete or internal closed ties, spirals, or stirrups.

7.10—Lateral reinforcement for compression members

7.10.1 Lateral reinforcement for compression members shall conform to the provisions of 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 prestressing tendons shall conform to 18.11.

7.10.3 It shall be permitted to waive the lateral reinforcement requirements of 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 3/8 in. diameter.

7.10.4.3 Clear spacing between spirals shall not exceed 3 in., nor be less than 1 in. See also 3.3.2.

7.10.4.4 Anchorage of spiral reinforcement shall be provided by 1-1/2 extra turns of spiral bar or wire at each end of a spiral unit.

7.10.4.5 Splices in spiral reinforcement shall be lap splices of 48 d_b but not less than 12 in., or welded.

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 No. 3 in size for longitudinal bars No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, 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 degree and no bar shall be farther than 6 in. 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 into all vertical faces of a column and if at least three quarters of each face is covered by the framing member, ties shall be permitted not more than 3 in. below lowest reinforcement in shallowest of such beams or brackets.

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 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 may 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 ℓ_d), or anchored in accordance with 12.13.

7.12—Minimum reinforcement

7.12.1 All exposed concrete surfaces shall be reinforced with reinforcement placed in two approximately perpendicular directions. For the purpose of the requirements of 7.12, concrete surfaces shall be considered to be exposed if they are not cast against existing concrete or against rock. The reinforcement shall be developed for its specified yield strength in conformance with Chapter 12. The minimum area of such reinforcement shall be in accordance with 7.12.2, 7.12.3 or 7.12.4., 7.12.5, or 7.12.6. This requirement may be met in total or in part by rein-

forcement otherwise required to resist design loads. Reinforcement shall be spaced not farther apart than 18 in.

7.12.2 For concrete sections less than 48 in. thick such reinforcement shall provide at least a ratio of area of reinforcement to gross concrete area of 0.0012 in each direction at each face.

7.12.3 For concrete sections having a thickness of 48 in. or more, such reinforcement shall provide an area A'_s in each direction at each face given by

$$A_{s\ min} = \frac{f'_t A}{f_s} \quad \text{but need not exceed } A/100$$

The minimum reinforcement size shall be No. 6 bars. In lieu of computation, f_s may be taken as 60% of the specified yield strength f_y.

7.12.4 For concrete sections having a thickness of 72 in. or more, no minimum reinforcement is required for members constructed by the principles and practice recommended by ACI Committee 207 for nonreinforced massive concrete structures.

7.12.5 On a tension face of a structural slab, wall, or shell, where a calculated reinforcement requirement exists, the ratio of reinforcement area provided at the tension face to gross concrete area shall not be less than 0.0018 unless the area of reinforcement provided at the tension face is at least one-third greater than that required by analysis. All other exposed faces of the structural slab, wall, or shell shall be reinforced to meet the minimum requirements of 7.12.1, 7.12.2 and 7.12.3.

7.12.6 Prestressing tendons conforming to 3.5.5 used for minimum reinforcement shall be provided in accordance with the following:

7.12.6.1 Tendons shall be proportioned to provide a minimum average compressive stress of 100 psi on gross concrete area using effective prestress, after losses, in accordance with 18.6.

7.12.6.2 Spacing of tendons shall not exceed 6 ft.

7.12.6.3 When spacing of tendons exceeds 54 in., additional bonded minimum reinforcement confining to 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.

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 over the support with a Class A tension splice and at noncontinuous supports be terminated with a standard hook.

7.13.2.2 Beams at the perimeter of the structure shall have at least one-sixth of the tension reinforcement required for negative moment at the support and one-quarter of the positive moment reinforcement required at midspan made continuous around the perimeter and tied with closed stirrups, or stirrups anchored around the negative moment

reinforcement with a hook having a bend of at least 135 degrees. Stirrups need not be extended through any joints. When splices are needed, the required continuity shall be provided with top reinforcement spliced at midspan and bottom reinforcement spliced at or near the support with Class A tension splices.

7.13.2.3 In other than perimeter beams, when closed stirrups are not provided, at least one-quarter of the positive moment reinforcement required at midspan shall be continuous or shall be spliced over the support with Class A tension splice and at noncontinuous supports be terminated with a standard hook.

7.13.2.4 For two-way slab construction, see 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.

7.1.3.4 For lift-slab construction, see 13.3.8.6 and 18.12.6.

PART 4—GENERAL REQUIREMENTS

CHAPTER 8—ANALYSIS AND DESIGN: GENERAL CONSIDERATIONS

8.0—Notation

A_s = area of nonprestressed tension reinforcement, in.^{2}

A'_s = area of compression reinforcement, in.^{2}

b = width of compression face of member, in.

d = distance from extreme compression fiber to centroid of tension reinforcement, in.

E_c = modulus of elasticity of concrete, psi. See 8.5.1

E_s = modulus of elasticity of reinforcement, psi. See 8.5.2 and 8.5.3

f'_c = specified compressive strength of concrete, psi

f_y = specified yield strength of nonprestressed reinforcement, psi

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_u = factored load per unit length of beam or per unit area of slab

w_c = unit weight of concrete, lb per ft^3

$ß_1$ = factor defined in 10.2.7.3

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

8.1—Design methods

8.1.1 In design of structural concrete, members shall be proportioned for adequate strength in accordance with provisions of this code, using load factors and strength reduction factors **f** specified in Chapter 9.

8.1.2 Anchors for attaching to concrete shall be designed using Appendix B, Anchoring to Concrete.

8.2—Loading

Design provisions of this Code are based on the assumption that structures shall be designed to resist all applicable loads. The loads shall be in accordance with the general requirements of 9.1.

8.3—Methods 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, and Appendices A, B, and C. It shall be permitted to simplify design by using the assumptions specified in 8.6 through 8.9.

8.3.2 Except for prestressed concrete, approximate methods of frame analysis are permitted for buildings of usual types of construction, spans, and story heights.

8.3.3 As an alternative 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%;

(c) Loads are uniformly distributed;

(d) Unit live load does not exceed 3 times unit dead load; and

(e) Members are prismatic.

Positive moment	
End spans	
Discontinuous end unrestrained........	$w_u \ell_n{}^2 / 11$
Discontinuous end integral with support...................................	$w_u \ell_n{}^2 / 14$
Interior spans.......................................	$w_u \ell_n{}^2 / 16$
Negative moment at exterior face of 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 10 ft; 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 a 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..	$1.15\, w_u \ell_n{}^2 / 2$
Shear at face of all other supports.............	$w_u \ell_n{}^2 / 2$

8.4—Redistribution of negative moments in continuous nonprestressed 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*

$$20\left(1 - \frac{\rho - \rho'}{\rho_b} \right)\ \%$$

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* For criteria on moment redistribution for prestressed concrete members, see 18.10.4.

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 the section, at which moment is reduced, is so designed that $\mathbf{r}$ or $\mathbf{r} - \mathbf{r}'$ is not greater than 0.50 $\mathbf{r}_b$, where

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{87,000}{87,000 + f_y}\right) \qquad (8\text{-}1)$$

8.5—Modulus of elasticity
8.5.1 Modulus of elasticity E_c for concrete shall be permitted to be taken as $w_c^{1.5} 33\sqrt{f_c'}$ (in psi) for values of w_c not exceeding 155 lb per cu ft. For normal weight concrete, E_c shall be permitted to be taken as $57,000\sqrt{f_c'}$.

8.5.2 Modulus of elasticity E_s for non-prestressed reinforcement shall be permitted to be taken as 29,000,000 psi.

8.5.3 Modulus of elasticity E_s for prestressing tendons shall be determined by tests or supplied by the manufacturer.

8.6—Stiffness
8.6.1 Use of any set of reasonable assumptions shall be permitted for computing the 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.

8.7—Span length
8.7.1 Span length of members not built integrally with supports shall be considered the clear span plus depth of member but need not exceed the 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 Solid or ribbed slabs built integrally with supports, with clear spans not more than 10 ft. It shall be permitted to analyze as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and width of beams otherwise neglected.

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 as fixed far ends of columns built integrally with the structure.

8.8.4 Resistance to moments at any floor or roof level shall be provided by distributing the moment between col-

umns immediately above and below the given floor in proportion to the relative column stiffnesses and conditions of restraint.

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; and
(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; and
(b) Factored dead load on all spans with full factored live load on alternate spans.

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 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; nor
(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; nor
(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 4 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 overhanging slab width need be considered.

    8.10.5.2 Transverse reinforcement shall not be spaced farther apart than five times the slab thickness, nor 18 in.

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 not be less than 4 in. in width; and shall have a depth of not more than 3-1/2 times the minimum width of rib.

8.11.3 Clear spacing between ribs shall not exceed 30 in.

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 Removable forms shall be used and slab thickness shall not be less than 1/12 the clear distance between ribs, nor less than 2 in.

8.11.6 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 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 1 in. 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% 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.

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 non-structural considerations.

CHAPTER 9—STRENGTH AND SERVICEABILITY REQUIREMENTS

9.0—Notation

A_g = gross area of section, in.^{2}

A_s = area of nonprestressed tension reinforcement, in.^{2}

A_s' = area of compression reinforcement, in.^{2}

d' = distance from extreme compression fiber to centroid of compression reinforcement, in.

d_s = distance from extreme tension fiber to centroid of tension reinforcement, in.

D = dead loads, or related internal moments and forces, including piping and equipment dead loads

E_c = modulus of elasticity of concrete, psi. See 8.5.1

E_o = load effects of operating basis earthquake (OBE), or related internal moments and forces, including OBE-induced piping and equipment reactions

E_{ss} = load effects of safe shutdown earthquake (SSE), or related internal moments and forces, including SSE-induced piping and equipment reactions

f_c' = specified compressive strength of concrete, psi

$\sqrt{f_c'}$ = square root of specified compressive strength of concrete, psi

f_r = modulus of rupture of concrete, psi

f_y = specified yield strength of nonprestressed reinforcement, psi

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, in.

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

I_e = effective moment of inertia for computation of deflection

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement

ℓ = span length of beam or one-way slab, as defined in 8.7; clear projection of cantilever, in.

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

L = live loads, or related internal moments and forces

M_a = maximum moment in member at stage deflection is computed

M_{cr} = cracking moment. See 9.5.2.3

P_a = differential pressure load, or related internal moments and forces, generated by a postulated pipe break

P_b = nominal axial load strength at balanced strain conditions. See 10.3.2

P_n = nominal axial load strength at given eccentricity

P_u = factored axial load at given eccentricity $\leq \phi P_n$

R_a = piping and equipment reactions, or related internal moments and forces, under thermal conditions generated by a postulated pipe break and including R_o.

R_o = piping and equipment reactions, or related internal moments and forces, which occur under normal operating and shutdown conditions, excluding dead load and earthquake reactions

T_a = internal moments and forces caused by temperature distributions within the concrete structure occurring as a result of accident conditions generated by a postulated pipe break and including T_o

T_o = internal moments and forces caused by temperature distributions within the concrete structure occurring as a result of normal operating or shutdown conditions

U = required strength to resist factored loads or related internal moments and forces

w_c = unit weight of concrete, lb per ft^3

W = operating basis wind load (OBW), or related internal moments and forces

W_t = loads generated by the design basis tornado (DBT), or related internal moments and forces. These include loads due to tornado wind pressure, tornado created differential pressures, and tornado generated missiles

Y_j = jet impingement load, or related internal moments and forces, on the structure generated by a postulated pipe break

Y_m = missile impact load, or related internal moments and forces, on the structure generated by a postulated pipe break, such as pipe whip

Y_r = loads, or related internal moments and forces, on the structure generated by the reaction of the bro-

ken pipe during a postulated break

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension

a = ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by center line of adjacent panel (if any) on each side of beam. See Chapter 13

a_m = average value of α for all beams on edges of a panel

ß = ratio of clear spans in long to short direction of two-way slabs

$ß_s$ = ratio of length of continuous edges to total perimeter of a slab panel

g = ratio of the bending moments of factored loads to unfactored loads

f = strength reduction factor. See 9.3

l = multiplier for additional long-term deflection as defined in 9.5.2.5

e = time-dependent factor for sustained load. See 9.5.2.5

r' = reinforcement ratio for nonprestressed compression reinforcement, A'_s / bd

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 stipulated for the following loads combined in accordance with the provisions specified in 9.2.

9.1.1.1—Normal loads

Those loads which are encountered during normal plant operation and shutdown including D, L, F, H, T_o, and R_o.

9.1.1.2—Severe environmental loads

Those loads that could infrequently be encountered during the plant life including E_o and W.

9.1.1.3—Extreme environmental loads

Those loads which are credible but are highly improbable including E_{ss} and W_t.

9.1.1.4—Abnormal loads

Those loads generated by a postulated high-energy pipe break accident including P_a, T_a, R_a, Y_r, Y_j and Y_m.

9.1.2 Members also shall meet all other requirements of this Code to ensure adequate performance at normal load levels.

9.1.3 In the design for normal loads, consideration shall be given to the forces due to such effects as prestressing, crane loads, vibration, impact, shrinkage, creep, unequal settlement of supports, construction, and testing.

9.1.4 In the determination of earthquake loads, consideration shall be given to the dynamic response characteristics of the concrete structure and its foundation and surrounding soil.

9.1.5 The determination of impulsive and impactive loads, such as the loads associated with missile impact, whipping pipes, jet impingement, and compartment pressurization, shall be consistent with the provisions of Appendix C.

9.2—Required strength

9.2.1 The required strength U shall be at least equal to the greatest of the following:

1. $U = 1.4D + 1.4F + 1.7L + 1.7H + 1.7R_o$

2. $U = 1.4D + 1.4F + 1.7L + 1.7H + 1.7E_o + 1.7R_o$

3. $U = 1.4D + 1.4F + 1.7L + 1.7H + 1.7W + 1.7R_o$

4. $U = D + F + L + H + T_o + R_o + E_{ss}$

5. $U = D + F + L + H + T_o + R_o + W_t$

6. $U = D + F + L + H + T_a + R_a + 1.25P_a$

7. $U = D + F + L + H + T_a + R_a + 1.15P_a$
    $+ 1.0(Y_r + Y_j + Y_m) + 1.15E_o$

8. $U = D + F + L + H + T_a + R_a + 1.0P_a$
    $+ 1.0(Y_r + Y_j + Y_m) + 1.0E_{ss}$

9. $U = 1.05D + 1.05F + 1.3L + 1.3H + 1.05T_o + 1.3R_o$

10. $U = 1.05D + 1.05F + 1.3L + 1.3H + 1.3E_o + 1.05T_o + 1.3R_o$

11. $U = 1.05D + 1.05F + 1.3L + 1.3H + 1.3W + 1.05T_o + 1.3R_o$

9.2.2 Where the structural effects of differential settlement, creep, shrinkage, or expansion of shrinkage-compensating concrete are significant, they shall be included with the dead load D in Load Combinations 4 through 11. Estimation of these effects shall be based on a realistic assessment of such effects occurring in service.

9.2.3 For the Load Combinations in 9.2.1, where any load reduces the effects of other loads, the corresponding factor for that load shall be taken as 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with the other loads. Otherwise, the factor for that load shall be taken as zero.

9.2.4 Where applicable, impact effects of moving loads shall be included with the live load L.

9.2.5 In Load Combinations 6, 7, and 8, the maximum values of P_a, T_a, R_a, Y_j, Y_r, and Y_m, including an appropriate dynamic load factor, shall be used unless an appropriate time-history analysis is performed to justify otherwise.

9.2.6 Load combinations 5, 7, and 8 shall be satisfied first without the tornado missile load in 5, and without Y_r, Y_j, and Y_m in 7 and 8. When considering these concentrated loads, local sections strengths and stresses may be exceeded provided there will be no loss of intended function of any safety related systems. For additional requirements related to impulsive and impactive effects, refer to Appendix C.

9.2.7 If resistance to other extreme environmental loads such as extreme floods is specified for the plant, then an additional load combination shall be included with the additional extreme environmental load substituted for W_t in Load Combination 5 of 9.2.1.

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 this Code, multiplied by a strength reduction factor ϕ in 9.3.2.

9.3.2 Strength reduction factor ϕ shall be as follows:

   9.3.2.1 Flexure, without axial load 0.90

   9.3.2.2 Axial load, and axial load with flexure. (For axial load with flexure, both axial load and moment nominal strength shall be multiplied by appropriate single value of ϕ)

(a) Axial tension, and axial tension with flexure.........0.90

(b) Axial compression, and axial compression with flexure:

   Members with spiral reinforcement conforming to

   10.9.3 ....0.75

   Other reinforced members 0.70

except that for low values of axial compression ϕ shall be permitted to be increased in accordance with the following:

For members in which f_y does not exceed 60,000 psi, with symmetric reinforcement, and with $(h - d\prime - d_s)/h$ not less than 0.70, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ to zero.

For other reinforced members, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ or ϕP_b, whichever is smaller, to zero.

   9.3.2.3 Shear and torsion 0.85

   9.3.2.4 Bearing on concrete

      (See also 18.13) 0.70

   9.3.2.5 Flexure compression, shear, and bearing for structural plain concrete 0.65

9.3.3 Development lengths specified in Chapter 12 do not require a ϕ-factor.

9.3.4 For determining the strength of joints, the shear strength reduction factor shall be 0.6 for any structural member 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. Shear strength reduction factor for joints shall be 0.85.

9.4—Design strength for reinforcement

Designs shall not be based on a yield strength of reinforcement f_y in excess of 60,000 psi, except for prestressing tendons.

9.5—Control of deflections

9.5.1—General

9.5.1.1—Deflection limits

Reinforced concrete members subject to flexure shall be designed to have adequate stiffness to limit deflections or any deformations which may adversely affect the strength and serviceability of structural and nonstructural elements.

One-way construction, two-way construction, and shored composite construction shall satisfy the minimum thickness requirements specified in this chapter. Prestressed concrete and unshored composite construction shall satisfy the deflection limits indicated in Table 9.5(a). Lesser thicknesses may be used if it is determined by computation that the resulting deflections will not adversely affect strength and serviceability.

Table 9.5(a)—Maximum deflections for unfactored loads

Type	Loading equation number (Section 9.2.1)	Beams	Slabs*
1	Eq. (1), (2), and (3)	$\ell/400$	$\ell/320$
2	Eq. (4) and (5)	$\ell/250$	$\ell/200$

* For two-way construction l shall be replaced by l_s.

When deflection limits more stringent than those specified in Table 9.5(a) are required to ensure the proper functioning of certain nonstructural systems, the minimum thicknesses specified in Tables 9.5(b) and 9.5(c) shall not apply and the members shall be sized such that the calculated deflections are within the required limits.

Table 9.5(b)—Minimum thickness of beams or one-way construction unless deflections are computed

Member	Simply supported	One end continuous	Both ends continuous	Cantilever
Solid one-way construction	$\ell/12$	$\ell/15$	$\ell/19$	$\ell/5$
Beams or ribbed one-way slabs	$\ell/10$	$\ell/13$	$\ell/16$	$\ell/4$

The values given shall be used directly for nonprestressed reinforced concrete members made with normal weight concrete ($w = 145$ pcf) and Grade 60 reinforcement.
For nonprestressed reinforcement having yield strengths less than 60,000 psi, the values in this table shall be multiplied by $(0.4 + f_{yt}/100,000)$.
The thickness of any one-way construction shall not be less than 6 in.

Table 9.5(c)—Minimum thickness of two-way construction unless deflections are computed

Support condition	Edge continuity	Minimum thickness, h	
		Ratio of clear span in long-to-short direction	
		$\beta = 1.0$	$\beta = 2.0$
$\alpha_m \geq 2.0$	$\beta_s = 0$	$\ell_n/22$	$\ell_n/25$
	$\beta_s = 1$	$\ell_n/25$	$\ell_n/30$
$\alpha_m \leq 1.0$	$\beta_s = 0$	$\ell_n/19$	$\ell_n/21$
	$\beta_s = 1$	$\ell_n/22$	$\ell_n/25$

The values given in this table shall be used directly for nonprestressed reinforced concrete members made with normal weight concrete. ($w = 145$ pcf) and Grade 60 reinforcement. For nonprestressed reinforcement having yield strengths less than 60,000 psi, the values n this table shall be multiplied by $(800 + 0.005 f_y)/1100$.
For other values of a_m, b_s, and b, the minimum thickness may be linearly interpolated.
The thickness of any two-way construction shall not be less than 6 in.

9.5.1.2—Loading conditions

When deflection computations are performed, these computations shall be based on the loading condition critical for flexure.

9.5.1.3—Factored load computations

The deflection limits specified in this chapter are for unfactored loads. Deflections may be computed by factored load analysis and divided by a factor **g** to obtain the deflections corresponding to unfactored loads. Unless otherwise determined by computation, the factor **g** shall be as follows:

(a) For load combinations 1 through 3, **g** = 1.5;

(b) For load combinations 4 through 8, **g** = 1.0; and

(c) For load combinations 9 through 11, **g** = 1.2.

9.5.1.4—Deflections to be considered

When minimum thickness requirements are satisfied, a deflection equal to the limits given in Table 9.5(a) may be considered for the design of nonstructural elements.

When calculations are performed, the sum of the long-time deflection due to all appropriate sustained loads, and the immediate elastic deflection due to all appropriate non-sustained loads shall be considered. Due consideration shall be given to the effective moment of inertia at each of these stages.

The long-time deflection shall be determined in accordance with 9.5.2.3, 9.5.3.5, or 9.5.4.2, but may be reduced to the amount of long-time deflection that occurs after the attachment of the nonstructural elements or the leveling of equipment. This amount of long-time deflection shall be determined on the basis of accepted engineering data relating to the time deflection characteristics of members similar to those being considered.

9.5.2—One-way construction (nonprestressed)

9.5.2.1 Minimum thickness stipulated in Table 9.5(b) shall apply for one-way construction unless computation of deflection indicates a lesser thickness may be used without adverse effects.

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 and with the effective moment of inertia as follows, but not greater than I_g.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (9\text{-}7)$$

where

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad (9\text{-}8)$$

and

$$f_r = 7.5 \sqrt{f_c'} \qquad (9\text{-}9)$$

When the values of M_a are obtained from factored load analysis, these values shall be divided by the factor **g** as specified in 9.5.1.3

Where the computation of deflection is to be based on I_e, the deflection calculated by an analysis using I_g may be used, if the deflection thus calculated is increased by a factor of I_g / I_e.

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. (9-7) 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. (9-7) 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 shall be determined by multiplying the immediate deflection caused by the sustained load considered, by the factor

$$\lambda = \frac{\xi}{1 + 50\rho'} \qquad (9\text{-}10)$$

where **r′** shall be the value at midspan for simple and continuous spans and at support for cantilevers. It is permitted to assume the time-dependent factor ξ for sustained loads 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 the design specification.

9.5.3—Two-way construction (nonprestressed)

9.5.3.1 For two-way construction, the minimum thickness stipulated in Table 9.5(c) shall apply unless the computation of deflection indicates that lesser thickness shall be permitted to be used without adverse effects.

9.5.3.2 For slabs without beams, but with drop panels extending in each direction from center line of support a distance not less than one-sixth the span length in that direction measured center-to-center of supports, and a projection below the slab at least one-fourth the slab thickness beyond the drop, thickness required by Table 9.5(c) shall be permitted to be reduced by 10%.

9.5.3.3 At discontinuous edges, an edge beam shall be provided with a stiffness ratio **a** not less than 0.80; or the minimum thickness required by Table 9.5(c) or 9.5.3.2, shall be increased by at least 10% in the panel with a discontinuous edge.

9.5.3.4—Computation of immediate deflection

Where deflections are to be computed, those which occur immediately on application of load shall be computed by the usual methods or formulas for elastic deflections and as specified in this chapter. These computations shall also take into account the size and shape of the panel,

the conditions of the support, and the nature of restraints at the panel edges. For such computations, the modulus of elasticity, E_c, of the concrete shall be as specified in 8.5.1. The effective moment of inertia shall satisfy the provisions of Section 9.5.2.3; other values may be used if they result in predictions of deflection in reasonable agreement with the results of comprehensive tests.

9.5.3.5—Computation of long-time deflections

Unless values are obtained by a more comprehensive analysis or test, the additional long-time deflection for normal weight two-way construction shall be computed in accordance with 9.5.2.3.

9.5.3.6—Allowable deflection

The deflection computed in accordance with 9.5.3.4 and 9.5.3.5 shall not exceed the limits stipulated in the design specification.

9.5.4—Prestressed concrete construction

9.5.4.1 For flexural members designed in accordance with provisions of Chapter 18, immediate camber and 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 uncracked sections. When members are cracked, a bilinear moment-curvature method shall be used. I_e as provided in Eq. (9-7) shall be permitted to be used for this purpose.

9.5.4.2 Additional long-time camber and deflection of prestressed concrete members shall be computed taking into account stresses and strain in concrete and steel under sustained load and including effects of creep and shrinkage of concrete and relaxation of steel.

9.5.4.3 Deflection computed in accordance with 9.5.4.1 and 9.5.4.2 shall not exceed limits stipulated in Table 9.5(a).

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 considered equivalent to a monolithically cast member, the values given in Table 9.5(b), or Table 9.5(c) as appropriate 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(b) or Table 9.5(c), as appropriate, deflection need not be computed. If the thickness of a nonprestressed composite member meets the requirements of Table 9.5(b) or Table 9.5(c), as appropriate, 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 9.5.5.1 and 9.5.5.2 shall not exceed limits stipulated in Table 9.5(a).

9.5.6—Walls

Walls subjected to transverse loads shall also satisfy the requirements as specified in this chapter for nonprestressed one-way or nonprestressed two-way, prestressed construction, or composite construction, as appropriate.

CHAPTER 10—FLEXURE AND AXIAL LOADS

10.0—Notation

a = depth of equivalent rectangular stress block as defined in 10.2.7.1

A = effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires, in.^{2} When the flexural reinforcement consists of different bar or wire sizes the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used

A_c = area of core of spirally reinforced compression member measured to outside diameter of spiral, in.^{2}

A_g = gross area of section, in.^{2}

A_s = area of nonprestressed tension reinforcement, in.^{2}

A_{sk} = area of skin reinforcement per unit height in one side face, in.^{2}/ft. See 10.6.7.

$A_{s,min}$ = minimum amount of flexural reinforcement, in.^{2} See 10.5

A_{st} = total area of longitudinal reinforcement, (bars or steel shapes), in.^{2}

A_t = area of structural steel shape, pipe, or tubing in a composite section, in.^{2}

A_1 = loaded area

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

b = width of compression face of member, in.

b_w = web width, in.

c = distance from extreme compression fiber to neutral axis, in.

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, in.

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto, in.

d_t = distance from extreme compression fiber to extreme tension steel, in.

E_c = modulus of elasticity of concrete, psi. See 8.5.1

E_s = modulus of elasticity of reinforcement, psi. See 8.5.2 or 8.5.3

EI = flexural stiffness of compression member. See Eq. (10-12) and Eq. (10-13)

f'_c = specified compressive strength of concrete, psi

f_s = calculated stress in reinforcement at service loads, ksi

f_y = specified yield strength of nonprestressed reinforcement, psi

h = overall thickness of member, in.

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement

I_{se} = moment of inertia of reinforcement about centroidal axis of member cross section

I_t = moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross section

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

ℓ_u = unsupported length of compression member

M_c = factored moment to be used for design of compression member

M_s = moment due to loads causing appreciable sway

M_u = factored moment at section

M_1 = smaller factored end moment on a compression member, positive if member is bent in single curvature, negative if bent in double curvature

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

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

M_2 = larger factored end moment on compression member, always positive

$M_{2,min}$ = minimum value of M_2

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

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

P_b = nominal axial load strength at balanced strain conditions. See 10.3.2

P_c = critical load. See Eq. (10- 11)

P_n = nominal axial load strength at given eccentricity

P_o = nominal axial load strength at zero eccentricity

P_u = factored axial load at given eccentricity $\leq \mathbf{f} P_n$

Q = stability index for a story. See 10.11.4

r = radius of gyration of cross section of a compression member

V_u = factored horizontal shear in a story

z = quantity limiting distribution of flexural reinforcement. See 10.6

β_1 = factor defined in 10.2.7.3

β_d = (a) for non-sway frames, $\mathbf{b}_d$ is the ratio of the maximum factored axial dead load to the total factored axial load

(b) for sway frames, except as required in (c), $\mathbf{b}_d$ is the ratio of the maximum factored sustained shear within a story to the total factored shear in that story

(c) for stability checks of sway frames carried out in accordance with 10.13.6, $\mathbf{b}_d$ is the ratio of the maximum factored sustained axial load to the total factored axial load

δ_{ns} = moment magnification factor for frames braced against sidesway, to reflect effects of member curvature between ends of compression member

$\mathbf{d}_s$ = moment magnification factor for frames not braced against sidesway, to reflect lateral drift resulting from lateral and gravity loads

$\mathbf{D}_o$ = 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

$\mathbf{r}$ = ratio of nonprestressed tension reinforcement
= A_s / bd

$\mathbf{r}_b$ = reinforcement ratio producing balanced strain conditions. See 10.3.2

$\mathbf{r}_s$ = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member

$\mathbf{f}$ = strength reduction factor. See 9.3

$\mathbf{f}_K$ = stiffness reduction factor. See R10.12.3

10.1—Scope

Provisions of Chapter 10 shall apply for design of members subject to flexure or axial loads or to combined flexure and axial loads.

10.2—Design assumptions

10.2.1 Strength design of members for flexure and axial loads shall be based on assumptions given in 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 for deep flexural members with overall depth to clear span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a nonlinear distribution of strain shall be considered. See 10.7.

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 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 10.2.6 are satisfied by an equivalent rectangular concrete stress distribution defined by the following:

10.2.7.1 Concrete stress of $0.85f'_c$ 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 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 4000 psi. For strengths above 4000 psi, β_1 shall be reduced continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi, but β_1 shall not be taken less than 0.65.

10.3—General principles and requirements

10.3.1 Design of cross section 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 For flexural members, and for members subject to combined flexure and compressive axial load when the design axial load strength ϕP_n is less than the smaller of $0.10f'_c A_g$ or ϕP_b, the ratio of reinforcement ρ provided shall not exceed 0.75 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 0.75 factor.

10.3.4 Use of compression reinforcement shall be permitted in conjunction with additional tension reinforcement to increase the strength of flexural members.

10.3.5 Design axial load strength ϕP_n of compression members shall not be taken greater than the following:

10.3.5.1 For nonprestressed members with spiral reinforcement conforming to 7.10.4 or composite members conforming to 10.16:

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

10.3.5.2 For nonprestressed members with tie reinforcement conforming to 7.10.5:

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

10.3.5.3 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.

10.3.6 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 10.3.5. The maximum factored moment M_u shall be magnified for slenderness effects in accordance with 10.10.

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.

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 10.5.2, 10.5.3, and 10.5.4, the area A_s provided shall not be less than that given by

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y}b_w d \quad (10\text{-}3)$$

and not less than $200\,b_w d/f_y$.

10.5.2 For a statically determinate T-section with flange in tension, the area $A_{s,min}$ shall be equal to or greater than the smaller value given either by

$$A_{s,min} = \frac{6\sqrt{f'_c}}{f_y}b_w d \quad (10\text{-}4)$$

or Eq. (10-3) with b_w set equal to the width of the flange.

10.5.3 The requirements of 10.5.1 and 10.5.2 need not be applied if at every section the area of tensile reinforcement provided is at least one-third greater than that required by analysis

10.5.4 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 the lesser of three times the thickness and 18 in.

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 When design yield strength f_y for tension reinforcement exceeds 40,000 psi, cross sections of maximum posi-

tive and negative moment shall be so proportioned that the quantity z given by

$$z = f_s \sqrt[3]{d_c A} \qquad (10\text{-}5)$$

does not exceed 175 kips per in. for interior exposure and 145 kips per in. for exterior exposure. Calculated stress in reinforcement at sustained loads f_s (kips /in.^{2}) shall be computed as the moment divided by the product of steel area and the internal moment arm. Alternatively, it shall be permitted to take f_s as 40% of specified yield strength f_y. The sustained loads shall include those loads identified in Load Combination 9, 9.2.1, with the load factors taken as unity.

10.6.5 Provisions of 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 8.10, or a width equal to 1/10 the span, whichever is smaller. If the effective flange width exceeds 1/10 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 36 in., 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 area of skin reinforcement A_{sk} per foot of height on each side face shall be $\geq 0.012\,(d-30)$. The maximum spacing of the skin reinforcement shall not exceed the lesser of $d/6$ and 12 in. 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.

10.7—Deep flexural members

10.7.1 Flexural members with overall depth to clear span ratios greater than 2/5 for continuous spans, or 4/5 for simple spans shall be designed as deep flexural members taking into account nonlinear distribution of strain and lateral buckling. (See also 12.10.6)

10.7.2 Shear strength of deep flexural members 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 flexural members shall be the greater of the requirements of 11.8.8, 11.8.9 and 11.8.10 or 14.3.2 and 14.3.3.

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 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 1-1/2 in. 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, octogonal, 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.

10.9—Limits for reinforcement of compression members

10.9.1 Area of longitudinal reinforcement for noncomposite compression members shall not be 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 10.9.3.

10.9.3 Ratio of spiral reinforcement ρ_s shall not be less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_y} \qquad (10\text{-}6)$$

where f_y is the specified yield strength of spiral reinforcement but not more than 60,000 psi.

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% 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 10.10.1, it shall be permitted to base the design of compression members, restraining beams, and other supporting mem-

bers on axial forces and moments from the analyses described in 10.11.

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 reflections $\mathbf{D}_o$ 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 $(1 + \mathbf{b}_d)$

(a) When sustained lateral loads act; or

(b) For stability checks made in accordance with 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 non-sway or sway columns or stories. The design of columns in non-sway frames or stories shall be based on 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 non-sway if the increase in column end moments due to second-order effects does not exceed 5% of the first-order end moments.

10.11.4.2 It also shall be permitted to assume a story within a structure is non-sway if:

$$Q = \frac{\Sigma P_u \Delta_o}{V_u \ell_c} \qquad (10\text{-}7)$$

is less than or equal to 0.05, where $\mathbf{S}P_u$ and V_u are the total vertical load and the story shear, respectively, in the story in question and $\mathbf{D}_o$ 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, 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.

10.12—Magnified moments: Non-sway frames

10.12.1 For compression members in non-sway 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 10.11.1.

10.12.2 In non-sway frames it shall be permitted to ignore slenderness effects for compression members which satisfy:

$$\frac{k\ell_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where M_1/M_2 is not taken less than –0.5. The term M_1/M_2 is positive if the column is bent in single 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:

$$M_c = \delta_{ns} M_2 \qquad (10\text{-}9)$$

where

$$\delta_{ns} = \frac{C_m}{1 - \dfrac{P_u}{0.75 P_c}} \ge 1.0 \qquad (10\text{-}10)$$

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \qquad (10\text{-}11)$$

EI shall be taken as

$$EI = \frac{(0.2 E_c I_g + E_s I_{se})}{1 + \beta_d} \qquad (10\text{-}12)$$

or

$$EI = \frac{0.4 E_c I_g}{1 + \beta_d} \qquad (10\text{-}13)$$

10.12.3.1 For members without transverse loads between supports, C_m shall be taken as

$$C_m = 0.6 + 0.4 \frac{M_1}{M_2} \ge 0.4 \qquad (10\text{-}14)$$

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. (10-9) shall not be taken less than

$$M_{2,\,min} = P_u(0.6 + 0.03h) \qquad (10\text{-}15)$$

about each axis separately, where 0.6 and h are in inches. For members for which $M_{2,min}$ exceeds M_2, the value of C_m in Eq. (10-14) 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.

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 10.11.1 and shall be greater than 1.0.

   10.13.2 For compression members not braced against sidesway, effects of slenderness may be neglected when $k\ell_u/r$ is less than 22.

   10.13.3 The moments M_1 and M_2 at the ends of an individual compression member shall be taken as

$$M_1 = M_{1ns} + \delta_s M_{1s} \qquad (10\text{-}16)$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \qquad (10\text{-}17)$$

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 10.11.1.

   10.13.4.2 Alternatively it shall be permitted to calculate $\delta_s M_s$ as

$$\delta_s M_s = \frac{M_s}{1 - Q} \geq M_s \qquad (10\text{-}18)$$

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

$$\delta_s M_s = \frac{M_s}{1 - \dfrac{\Sigma P_u}{0.75\Sigma P_c}} \geq M_s \qquad (10\text{-}19)$$

where ΣP_u is the summation for all the vertical loads in a story and ΣP_c is the summation for all sway resisting columns in a story. P_c is calculated using Eq. (10-11) using k from 10.13.1 and EI from Eq. (10-12) or Eq. (10-13).

   10.13.5 If an individual compression member has

$$\frac{\ell_u}{r} > \frac{35}{\sqrt{\dfrac{P_u}{f_c' A_g}}} \qquad (10\text{-}20)$$

it shall be designed for the factored axial load P_u and the moment M_c calculated using 10.12.3 in which M_1 and M_2 are computed in accordance with 10.13.3, β_d as defined for the load combination under consideration, and k as defined in 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 ΣP_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 ΣP_u and ΣP_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 total 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.

10.14—Axially loaded members supporting slab system

Axially loaded members supporting a slab system included within the scope of 13.1 shall be designed as provided in Chapter 10 and in accordance with the additional requirements of Chapter 13.

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 one of the following.

   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 2 ft into the slab from face of column. Column concrete shall be well integrated with floor concrete, and shall be placed in accordance with 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, strength of the column may be based on an assumed concrete strength in the column joint equal to 75% of column concrete strength plus 35% of floor concrete strength.

10.16—Composite compression members

10.16.1 Composite compression members shall include all such members reinforced longitudinally with structural steel shapes, pipe, or tubing with or without longitudinal bars.

10.16.2 Strength of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

10.16.3 Any axial load strength assigned to concrete of a composite member shall be transferred to the concrete by members or brackets in direct bearing on the composite member concrete.

10.16.4 All axial load strength not assigned to concrete of a composite member shall be developed by direct connection to the structural steel shape, pipe, or tube.

10.16.5 For evaluation of slenderness effects, radius of gyration of a composite section shall be not greater than the value given by

$$r = \sqrt{\frac{(E_c I_g / 5) + E_s I_t}{(E_c A_g / 5) + E_s A_t}} \qquad (10\text{-}21)$$

and, as an alternative to a more accurate equation, *EI* in Eq. (10-11) shall be taken either as Eq. (10-12) or

$$EI = \frac{(E_c I_g / 5)}{1 + \beta_d} + E_s I_t \qquad (10\text{-}22)$$

10.16.6—Structural steel encased concrete core

10.16.6.1 For a composite member with concrete core encased by structural steel, thickness of the steel encasement shall not be less than

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

nor

$$h \sqrt{\frac{f_y}{8E_s}} \quad \text{for circular sections of diameter } h$$

10.16.6.2 Longitudinal bars located within the encased concrete core shall be permitted to be used in computing A_t and I_t.

10.16.7—Spiral reinforcement around structural steel core

A composite member with spirally reinforced concrete around a structural steel core shall conform to the following.

10.16.7.1 Specified compressive strength of concrete f_c' shall be not less than 25000 psi.

10.16.7.2 Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 50,000 psi.

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 the following.

10.16.8.1 Specified compressive strength of concrete f_c' shall be not less than 2500 psi.

10.16.8.2 Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 50,000 psi.

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 1/50 times the greatest side dimension of composite member, except that ties shall not be smaller than No. 3 and are not required to be larger than No. 5. 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 1/2 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 items 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 further 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.

10.17—Bearing strength

10.17.1 Design bearing strength on concrete shall not exceed $\phi(0.85 f_c' A_1)$, except when the supporting surface is wider on all sides than the loaded area, design bearing strength on 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

11.0—Notation

a = shear span, distance between concentrated load and face of support

A_c = area of concrete section resisting shear transfer, in.^{2}

A_{cp} = area enclosed by outside perimeter of concrete cross section, in.^{2} See 11.6.1

A_f = area of reinforcement in bracket or corbel resisting factored moment, $[V_u a + N_{uc}(h - d)]$, in.^{2}

A_g = gross area of section, in.^{2}

A_h = area of shear reinforcement parallel to flexural tension reinforcement, in.^{2}

A_ℓ = total area of longitudinal reinforcement to resist torsion, in.^{2}

A_n = area of reinforcement in bracket or corbel resisting tensile force N_{uc}, in.^{2}

A_o = gross area enclosed by shear flow path, in.^{2}

A_{oh} = area enclosed by centerline of the outermost closed

transverse torsional reinforcement, in.^{2}

A_{ps} = area of prestressed reinforcement in tension zone, in.^{2}

A_s = area of nonprestressed tension reinforcement, in.^{2}

A_t = area of one leg of a closed stirrup resisting torsion within a distance s, in.^{2}

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, in.^{2}

A_{vf} = area of shear-friction reinforcement, in.^{2}

A_{vh} = area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2, in.^{2}

b = width of compression face of member, in.

b_o = perimeter of critical section for slabs and footings, in.

b_t = width of that part of cross section containing the closed stirrups resisting torsion

b_w = web width, or diameter of circular section, in.

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, in.

b_2 = width of the critical section defined in 11.12.1.2 measured in the direction perpendicular to b_1, in.

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, in.

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, in.

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, but need not be less than $0.80h$ for prestressed members, in. (For circular sections, d need not be less than the distance from extreme compression fiber to centroid of tension reinforcement in opposite half of member.)

f'_c = specified compressive strength of concrete, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, psi

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, psi. (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)

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, psi

f_{pu} = specified tensile strength of prestressing tendons, psi

f_y = specified yield strength of nonprestressed reinforcement, psi

f_{yv} = yield strength of closed transverse torsional reinforcement, psi

$f_{y\ell}$ = yield strength of longitudinal torsional reinforcement, psi

h = overall thickness of member, in.

h_v = total depth of shearhead cross section, in.

h_w = total height of wall from base to top, in.

I = moment of inertia of section resisting externally applied factored loads

ℓ_n = clear span measured face-to-face of supports

ℓ_v = length of shearhead arm from centroid of concentrated load or reaction, in.

ℓ_w = horizontal length of wall, in.

M_{cr} = moment causing flexural cracking at section due to externally applied loads. See 11.4.2.1

M_m = modified moment

M_{max} = maximum factored moment at section due to externally applied loads

M_p = required plastic moment strength of shearhead cross section

M_u = factored moment at section

M_v = moment resistance contributed by shearhead reinforcement

N_u = factored axial load normal to cross section occurring simultaneously with V_u; to be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage

N_{uc} = factored tensile force applied at top of bracket or corbel acting simultaneously with V_u, to be taken as positive for tension

p_{cp} = outside perimeter of the concrete cross section, in. See 11.6.1

p_h = perimeter of centerline of outermost closed transverse torsional reinforcement, in.

s = spacing of shear or torsion reinforcement in direction parallel to longitudinal reinforcement, in.

s_1 = spacing of vertical reinforcement in wall, in.

s_2 = spacing of shear or torsion reinforcement in direction perpendicular to longitudinal reinforcement or spacing of horizontal reinforcement in wall, in.

t = thickness of a wall of a hollow section, in.

T_n = nominal torsional moment strength

T_u = factored torsional moment at section

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

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web

V_d = shear force at section due to unfactored dead load

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}

V_n = nominal shear strength

V_p = vertical component of effective prestress force at section

V_s = nominal shear strength provided by shear reinforcement

V_u = factored shear force at section

v_n = nominal shear stress, psi. See 11.12.6.2

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension

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 stiffness of shearhead arm to 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.

11.1—Shear strength

11.1.1 Design of cross sections subject to shear shall be based on

$$\phi V_n \geq V_u \qquad (11\text{-}1)$$

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

$$V_n = V_c + V_s \qquad (11\text{-}2)$$

where V_c is nominal shear strength provided by concrete in accordance with 11.3 or 11.4, and V_s is nominal shear strength provided by shear reinforcement in accordance with 11.5.6.

11.1.1.1 In determining shear strength V_n, 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 100 psi except as allowed in 11.1.2.1.

11.1.2.1 Values of $\sqrt{f_c'}$ greater than 100 psi 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 equal to f_c' /5000 times, but not more than three times the amounts required by 11.5.5.3, 11.5.5.4, or 11.5.5.5.

11.1.3 Computation of maximum factored shear force V_u at supports shall be permitted in accordance with 11.1.3.1 or 11.1.3.2 when both of the following conditions are satisfied:

(a) support reaction, in direction of applied shear, introduces compression into the end regions of member, and

(b) no concentrated load occurs between face of support and location of critical section defined in 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 flexural members, brackets and corbels, walls, and slabs and footings, the special provisions of 11.8 through 11.12 shall apply.

11.2—Lightweight concrete

Rules for lightweight aggregate concrete are not provided herein.

11.3—Shear strength provided by concrete for nonprestressed members

11.3.1 Shear strength V_c shall be computed by provisions of 11.3.1.1 through 11.3.1.4, unless a more detailed calculation is made in accordance with 11.3.2.

11.3.1.1 For members subject to shear and flexure only,

$$V_c = 2\sqrt{f_c'}\, b_w d \qquad (11\text{-}3)$$

11.3.1.2 For members subject to axial compression,

$$V_c = 2\left(1 + \frac{N_u}{2000 A_g}\right)\sqrt{f_c'}\, b_w d \qquad (11\text{-}4)$$

Quantity N_u / A_g shall be expressed in psi.

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 11.3.2.3.

11.3.2 Shear strength V_c may be computed by the more detailed calculation of 11.3.2.1 through 11.3.2.3.

11.3.2.1 For members subject to shear and flexure only,

$$V_c = \left(1.9\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d \qquad (11\text{-}5)$$

but not greater than $3.5\sqrt{f_c'}\, b_w d$. Quantity $V_u d / M_u$ shall not be taken greater than 1.0 in computing V_c by Eq. (11-6), 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

$$M_m = M_u - N_u \frac{(4h - d)}{8} \tag{11-6}$$

However, V_c shall not be taken greater than

$$V_c = 3.5 \sqrt{f_c'} b_w d \sqrt{1 + \frac{N_u}{500 A_g}} \tag{11-7}$$

Quantity N_u / A_g shall be expressed in psi. 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,

$$V_c = 2\left(1 + \frac{N_u}{500 A_g}\right)\sqrt{f_c'} b_w d \tag{11-8}$$

but not less than 0 where N_u is negative for tension. Quantity N_u / A_g shall be expressed in psi.

11.4—Shear strength provided by concrete for prestressed members

11.4.1 For members with effective prestress force not less than 40% of the tensile strength of flexural reinforcement, unless a more detailed calculation is made in accordance with 11.4.2,

$$V_c = \left(0.6\sqrt{f_c'} + 700\frac{V_u d}{M_u}\right) b_w d \tag{11-9}$$

but V_c need not be taken less than $2\sqrt{f_c'}b_w d$ nor shall V_c be taken greater than $5\sqrt{f_c'}b_w d$ nor the value given in 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 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 may be computed in accordance with 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

$$V_{ci} = 0.6\sqrt{f_c'} b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \tag{11-10}$$

but V_{ci} need not be taken less than $1.7 f_c' \, b_w \, d$, where

$$M_{cr} = (I/y_t)(6\sqrt{f_c'} + f_{pe} - f_d) \tag{11-11}$$

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

$$V_{cw} = (3.5\sqrt{f_c'} + 0.3 f_{pc})b_w d + V_p \tag{11-12}$$

Alternatively, V_{cw} may be computed as the shear force corresponding to dead load plus live load that results in a principal tensile stress of $4\sqrt{f_c'}$ at the centroidal axis of member, or at intersection of flange and web when centroidal axis is in the flange. In composite members, 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 end of member than the transfer length of the prestressing tendons, 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 prestress force shall be assumed to vary linearly from zero at end of tendon to a maximum at a distance from end of tendon 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 the member, a reduced prestress shall be considered when computing V_c in accordance with 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 the 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.

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 may be permitted:

(a) Stirrups perpendicular to axis of member; and

(b) Welded wire fabric with wires located perpendicular to axis of member.

11.5.1.2 For nonprestressed members, shear reinforcement shall be permitted to also consist of:

(a) Stirrups making an angle of 45 degrees or more with longitudinal tension reinforcement;

(b) Longitudinal reinforcement with bent portion making an angle of 30 degrees or more with the longitudinal tension reinforcement;

(c) Combinations of stirrups and bent longitudinal reinforcement; and

(d) Spirals.

11.5.2 Design yield strength of shear reinforcement shall not exceed 60,000 psi except that the design yield strength of welded deformed wire fabric shall not exceed 80,000 psi.

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 and $(3/4)h$ in prestressed members, nor 24 in.

11.5.4.2 Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45 degree 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 $4\sqrt{f_c'}b_w d$, maximum spacings given in 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; and

(c) Beams with total depth not greater than 10 in., 2-1/2 times thickness of flange, or one-half the width of web, whichever is greatest.

11.5.5.2 Minimum shear reinforcement requirements of 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 11.5.5.1 or for strength and where 11.6.1 allows for torsion to be neglected, the minimum area of shear reinforcement for prestressed (except as provided in *11.5.5.4) an*d nonprestressed members shall be computed by

$$A_v = 50\frac{b_w s}{f_y} \qquad (11\text{-}13)$$

where b_w and s are in inches.

11.5.5.4 For prestressed members with an effective prestress force not less than 40% of the tensile strength of flexural reinforcement, the area of shear reinforcement shall not be less than the smaller A_v from Eq. (11-13) and (11-14).

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

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 11.5.6.8.

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 inclined stirrups are used as shear reinforcement,

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

11.5.6.4 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 $3\sqrt{f_c'}b_w d$.

11.5.6.5 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).

11.5.6.6 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

11.5.6.7 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.

11.5.6.8 Shear strength V_s shall not be taken greater than $8\sqrt{f_c'}b_w d$.

11.6—Design for torsion

11.6.1 It shall be permitted to neglect torsion effects when the factored torsional moment T_u is less than:

(a) for nonprestressed members:

$$\phi\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$$

(b) for prestressed members:

$$\phi \sqrt{f_c'} \left(\frac{A_{cp}^2}{p_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{4\sqrt{f_c'}}}$$

For members cast monolithically with a slab, the over-hanging flange width used in computing A_{cp} and p_{cp} shall conform to 13.2.4.

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

(a) for nonprestressed members, at the sections described in 11.6.2.4:

$$\phi 4 \sqrt{f_c'} \left(\frac{A_{cp}^2}{p_{cp}} \right)$$

(b) for prestressed members, at the sections described in 11.6.2.5:

$$\phi 4 \sqrt{f_c'} \left(\frac{A_{cp}^2}{p_{cp}} \right) \sqrt{1 + \frac{f_{pc}}{4\sqrt{f_c'}}}$$

In such a case, the correspondingly redistributed bending moments and shears in the adjoining members shall be used in the design of these members.

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:

$$\sqrt{\left(\frac{V_u}{b_w d} \right)^2 + \left(\frac{T_u p_h}{1.7 A_{oh}^2} \right)^2} \leq \phi \left(\frac{V_c}{b_w d} + 8\sqrt{f_c'} \right) \qquad (11\text{-}18)$$

(b) for hollow sections:

$$\left(\frac{V_u}{b_w d} \right) + \left(\frac{T_u p_h}{1.7 A_{oh}^2} \right) \leq \phi \left(\frac{V_c}{b_w d} + 8\sqrt{f_c'} \right) \qquad (11\text{-}19)$$

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 terms in Eq. (11-19) shall be taken as:

$$\left(\frac{T_u}{1.7 A_{oh} t} \right)$$

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 60,000 psi.

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:

$$T_n = \frac{2 A_o A_t f_{yv}}{s} \cot \theta \qquad (11\text{-}21)$$

where A_o shall be determined by analysis except that it shall be permitted to take A_o equal to $0.85 A_{oh}$; θ shall not be taken smaller than 30 degrees nor larger than 60 degrees. It shall be permitted to take θ equal to:

(a) 45 degrees for nonprestressed members or members with less prestress than in (b); or

(b) 37.5 degrees for prestressed members with an effective prestress force not less than 40% 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 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 must be met.

11.6.3.9 It shall be permitted to reduce the area of longitudinal torsion reinforcement in the flexural compression zone by an amount equal to $M_u/(0.9 d f_{y\ell})$, where M_u is the factored moment acting at the section in combination with T_u, except that the reinforcement provided shall not be less than that required by 11.6.5.3 or 11.6.6.2.

11.6.3.10 In prestressed beams:

(a) the total longitudinal reinforcement including tendons 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; and

(b) the spacing of the longitudinal reinforcement including tendons shall satisfy the requirements in 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 11.6.3.10 in accordance with 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; or

(b) a closed cage of welded wire fabric with transverse wires perpendicular to the axis of the member; or

(c) in nonprestressed beams, spiral reinforcement.

11.6.4.2 Transverse torsional reinforcement shall be anchored by one of the following:

(a) a 135 degree standard hook around a longitudinal bar; or

(b) according to 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 close stirrups shall be computed by:

$$(A_v + 2A_t) = \frac{50b_w s}{f_{yv}} \qquad (11\text{-}23)$$

11.6.5.3 Where torsional reinforcement is required by 11.6.5.1, the minimum total area of longitudinal torsional reinforcement shall by computed by:

$$A_{\ell, min} = \frac{5\sqrt{f_c'}\,A_{cp}}{f_{y\ell}} - \left(\frac{A_t}{s}\right)p_h\frac{f_{yv}}{f_{y\ell}} \qquad (11\text{-}24)$$

where A_t/s shall not be taken less than $25b_w/f_{yv}$.

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 12 in.

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 12 in. 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 1/24 of the stirrup spacing, but not less than a No. 3 bar.

11.6.6.3 Torsion reinforcement shall be provided for a distance of at least $(b_t + d)$ beyond the point theoretically required.

11.7—Shear-friction

11.7.1 Provisions of 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 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 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 shear plane, shear strength V_n shall be computed by

$$V_n = A_{vf}f_y\mu \qquad (11\text{-}25)$$

where μ is coefficient of friction in accordance with 11.7.4.3.

11.7.4.2 When shear-friction reinforcement is inclined to shear plane, such that the shear force produces tension in shear-friction reinforcement, shear strength V_n shall be computed by

$$V_n = A_{vf}f_y(\mu\sin\alpha_f + \cos\alpha_f) \qquad (11\text{-}26)$$

where α_f is the angle between shear-friction reinforcement and shear plane.

11.7.4.3 Coefficient of friction μ in Eq. (11-25) and Eq. (11-26) shall be

Concrete placed monolithically 1.4

Concrete placed against hardened concrete with surface intentionally roughened as specified in 11.7.9 ....1.0

Concrete placed against hardened concrete not intentionally roughened 0.6

Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see 11.7.10)...0.7

11.7.5 Shear strength V_n shall not be taken greater than $0.2f_c' A_c$ nor $800 A_c$ in pounds, where A_c is area of concrete section resisting shear transfer.

11.7.6 Design yield strength of shear-friction reinforcement shall not exceed 60,000 psi.

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 1.0, interface shall be roughened to a full amplitude of approximately 1/4 in.

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.

11.8—Special provisions for deep flexural members

11.8.1 The provisions of 11.8 shall apply to members with ℓ_n/d less than 5 that are loaded on one face and supported on the opposite face so that compression struts can develop between the loads and the supports. See also 12.10.6.

11.8.2 The design of simply supported deep flexural members for shear shall be based on Eq. (11-1) and (11-2), where the shear strength V_c shall be in accordance with 11.8.6 or 11.8.7 and the shear strength V_s shall be in accordance with 11.8.8.

11.8.3 The design of continuous deep flexural members for shear shall be based on 11.1 through 11.5 with 11.8.5 substituted for 11.1.3, or on methods satisfying equilibrium and strength requirements. In either case the design shall also satisfy 11.8.4, 11.8.9, and 11.8.10.

11.8.4 Shear strength V_n for deep flexural members shall not be taken greater than $8 \sqrt{f_c'} b_w d$ when ℓ_n/d is less than 2. When ℓ_n/d is between 2 and 5,

$$V_n = \frac{2}{3}\left(10 + \frac{\ell_n}{d}\right)\sqrt{f_c'} b_w d \qquad (11\text{-}27)$$

11.8.5 Critical section for shear measured from face of support shall be taken at a distance $0.15\ell_n$ for uniformly loaded beams and $0.50a$ for beams with concentrated loads, but not greater than d.

11.8.6 Unless a more detailed calculation is made in accordance with 11.8.7,

$$V_c = 2 \sqrt{f_c'} b_w d \qquad (11\text{-}28)$$

11.8.7 Shear strength V_c may be computed by

$$V_c = \left(3.5 - 2.5\frac{M_u}{V_u d}\right)\left(1.9\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d \quad (11\text{-}29)$$

except that the term

$$\left(3.5 - 2.5\frac{M_u}{V_u d}\right)$$

shall not exceed 2.5, and V_c shall not be taken greater than $6\sqrt{f_c'} b_w d$. M_u is factored moment occurring simultaneously with V_u at the critical section defined in 11.8.5.

11.8.8 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 by

$$V_s = \left[\frac{A_v}{s}\left(\frac{1 + \frac{\ell_n}{d}}{12}\right) + \frac{A_{vh}}{s_2}\left(\frac{11 - \frac{\ell_n}{d}}{12}\right)\right] f_y d \qquad (11\text{-}30)$$

where A_v is area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s, and A_{vh} is area of shear reinforcement parallel to flexural reinforcement within a distance s_2.

11.8.9 Area of shear reinforcement A_v shall not be less than $0.0015 b_w s$, and s shall not exceed $d/5$, nor 18 in.

11.8.10 The area of horizontal shear reinforcement A_{vh} shall not be less than $0.0025 b_w s_2$, and s_2 shall not exceed $d/3$, nor 18 in.

11.8.11 Shear reinforcement required at the critical section defined in 11.8.5 shall be used throughout the span.

11.9—Special provisions for brackets and corbels

11.9.1 Provisions of 11.9 shall apply to brackets and corbels with a shear span-to-depth ratio a/d not greater than unity, and subject to a horizontal tensile force N_{uc} not larger than V_u. Distance d shall be measured at face of 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 $[V_u a + N_{uc} (h - d)]$, and a horizontal tensile force N_{uc}.

11.9.3.1 In all design calculations in accordance with 11.9, strength reduction factor $\mathbf{f}$ shall be taken equal to 0.85.

11.9.3.2 Design of shear-friction reinforcement A_{vf} to resist shear V_u shall be in accordance with 11.7.

11.9.3.2.1 Shear strength V_n shall not be taken greater than $0.2f_c' b_w d$ nor $800 b_w d$ in pounds.

11.9.3.3 Reinforcement A_f to resist moment $[V_u a + N_{uc}(h - d)]$ 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 $0.2 V_u$ 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_h not less than $0.4(A_s - A_n)$, shall be uniformly distributed within two-thirds of the effective depth adjacent to A_s.

11.9.5 Ratio $\mathbf{r} = A_s / bd$ 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).

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 shear forces in plane of wall shall be in accordance with 11.10.2 through 11.10.8.

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 $10\sqrt{f_c'}hd$.

11.10.4 For design for horizontal shear forces in plane of wall, d shall be taken equal to $0.8\ell_w$. A 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 $2\sqrt{f_c'}hd$ 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 computed by Eq. (11-31) and (11-32), where V_c shall be the lesser of Eq. (11-31) or (11-32).

$$V_c = 3.3\sqrt{f_c'}hd + \frac{N_u d}{4\ell_w} \qquad (11\text{-}31)$$

or

$$V_c = \left[0.6\sqrt{f_c'} + \frac{\ell_w\left(1.25\sqrt{f_c'} + 0.2\dfrac{N_u}{\ell_w h}\right)}{\dfrac{M_u}{V_u} - \dfrac{\ell_w}{2}} \right]hd \qquad (11\text{-}32)$$

where N_u is negative for tension. When $(M_u / V_u - \ell_w/2)$ is negative, Eq. (11-32) 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_c 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 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

$$V_s = \frac{A_v f_y d}{s_2} \qquad (11\text{-}33)$$

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 18 in.

11.10.9.4 Ratio ρ_n of vertical shear reinforcement area to gross concrete area of horizontal section shall not be less than

$$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_h - 0.0025) \qquad (11\text{-}34)$$

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 18 in.

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. (11-14) within the column for a depth not less than that of the deepest connection of framing elements to the columns. See also 7.9.

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_0 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 11.12.2.1, 11.12.2.2, or 11.12.3.1. V_s shall be computed in accordance with 11.12.3. For slabs with shearheads, V_n shall be in accordance with 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)
$$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f_c'}\, b_0 d \qquad (11\text{-}35)$$

where b_c is the ratio of long side to short side of the column, concentrated load or reaction area;

(b)
$$V_c = \left(\frac{\alpha_s d}{b_0} + 2\right)\sqrt{f_c'}\, b_0 d \qquad (11\text{-}36)$$

where a_s is 40 for interior columns, 30 for edge columns, 20 for corner columns; and

(c)
$$V_c = 4\sqrt{f_c'}\, b_0 d \qquad (11\text{-}37)$$

11.12.2.2 At columns of two-way prestressed slabs and footings that meet the requirements of 18.9.3

$$V_c = (\beta_p\sqrt{f_c'} + 0.3 f_{pc}) b_0 d + V_p \qquad (11\text{-}38)$$

where b_p is the smaller of 3.5 or ($a_s d/b_0 + 1.5$), a_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns, b_0 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. (11-38) if the following are satisfied; otherwise, 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. (11-38) shall not be taken greater than 5000 psi; and

(c) f_{pc} in each direction shall not be less than 125 psi, nor be taken greater than 500 psi.

11.12.2.3 For slabs or footings in which the shear force V_u occurs with membrane stresses f_{m1} or f_{m2},

$$V_c = V_{c1} + V_{c2} \qquad (11\text{-}38a)$$

where V_{c1} and V_{c2} are computed in accordance with 11.12.2.3.1 and 11.12.2.3.2.

11.12.2.3.1 For f_{m1} tensile and not exceeding $0.9\, \rho_1' f_y$

$$V_{c1} = (2 + 4/\beta_c)\sqrt{f_c'}\, b_1' h(1 + 0.25 f_{m1}/\rho_1' f_y) \quad (11\text{-}38b)$$

except the factor $(2 + 4/b_c)$ shall not be taken greater than 4. For f_{m1} tensile and exceeding $0.9\, \rho_1' f_y$

$$V_{c1} = 0.5\sqrt{f_c'}\, b_1' h \qquad (11\text{-}38c)$$

For f_{m1} compressive and not less than 125 psi, V_{c1} shall be taken as V_c computed in accordance with 11.12.2.2 except that in Eq. (11-38) f_{m1} and b_1' shall be used in place of f_{pc} and b_0, respectively. For f_{m1} compressive and less than 125 psi, V_{c1} shall be taken as V_c computed in accordance with 11.12.2.1 except that b_1' shall be used in place of b_0.

11.12.2.3.2 For f_{m2} tensile and not exceeding $0.9\, \rho_2' f_y$

$$V_{c2} = (2 + 4/\beta_c)\sqrt{f_c'}\, b_2' h(1 + 0.25 f_{m2}/\rho_2' f_y) \quad (11\text{-}38d)$$

except the factor $(2 + 4/b_c)$ shall not be taken greater than 4. For f_{m2} tensile and exceeding $0.9\, \rho_2' f_y$

$$V_{c2} = 0.5\sqrt{f_c'}\, b_2' h \qquad (11\text{-}38e)$$

For f_{m2} compressive and not less than 125 psi, V_{c2} shall be taken as V_c computed in accordance with 11.12.2.2 except that in Eq. (11-38) f_{m2} and b_2' shall be used in place of f_{pc} and b_0, respectively. For f_{m2} compressive and less than 125 psi, V_{c2} shall be taken as V_c computed in accordance with 11.12.2.1 except b_2' shall be used in place of b_0.

11.12.2.3.3 When V_u does not exceed ϕV_c and f_{m1} is a tensile stress that exceeds $0.6\, r_1' f_y$, then r_1' provided shall not be less than

$$\rho_1' = f_{m1}/(0.9 f_y) + V_u/(0.85 f_y b_1' h) \qquad (11\text{-}38f)$$

In addition, r_1' shall be increased if required for any in-plane shear force that exists, computed in accordance with 11.10.

11.12.2.3.4 When V_u does not exceed ϕV_c and f_{m2} is a tensile stress that exceeds $0.6\, \rho_2' f_y$, then ρ_2' provided shall not be less than

$$\rho_2' = f_{m2}/(0.9f_y) + V_u/(0.85f_yb_2'h) \qquad (11\text{-}38g)$$

In addition, $\mathbf{r_2'}$ shall be increased if required for any in-plane shear force that exists, computed in accordance with 11.10.

11.12.3 Shear reinforcement consisting of bars or wires shall be permitted in slabs and footings in accordance with the following:

11.12.3.1 V_n shall be computed by Eq. (11-2), where V_c shall not be taken greater than $2\sqrt{f_c'}b_0d$, and the required area of shear reinforcement A_v and V_s shall be calculated in accordance with 11.5 and anchored in accordance with 12.13.

11.12.3.2 V_n shall not be taken greater than $6\sqrt{f_c'}b_0d$.

11.12.4 Shear reinforcement consisting of steel I- or channel-shaped sections (shearheads) shall be permitted in slabs. The provisions of 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, 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 degrees 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 $\mathbf{a}_v$ between the 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

$$\phi M_p = \frac{V_u}{2\eta}\left[h_v + \alpha_v\left(\ell_v - \frac{c_1}{2}\right)\right] \qquad (11\text{-}39)$$

where $\mathbf{f}$ is the strength reduction factor for flexure, $\mathbf{h}$ is the number of arms, and ℓ_v is the minimum length of each shearhead arm required to comply with requirements of 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_0 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 $4\sqrt{f_c'}b_0d$, on the critical section defined in 11.12.4.7. When shearhead reinforcement is provided, V_n shall not be taken greater than $7\sqrt{f_c'}b_0d$ on the critical section defined in 11.12.1.2(a).

11.12.4.9 The moment resistance M_v contributed to each slab column strip by a shearhead shall not be taken greater than

$$M_v = \frac{\phi\alpha_v V_u}{2\eta}\left(\ell_v - \frac{c_1}{2}\right) \qquad (11\text{-}40)$$

where $\mathbf{f}$ is the strength reduction factor for flexure, 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% of the total factored moment required for each slab column strip;

(b) the change in column strip moment over the length ℓ_v; and

(c) the value of M_p computed by Eq. (11-39)

11.12.4.10 When unbalanced moments are considered, the shearhead must have adequate anchorage to transit M_p to 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 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 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 $\mathbf{g}\cdot M_u$ of the unbalanced moment shall be transferred by flexure in accordance with 13.5.3. The remainder of the unbalanced moment given by $\mathbf{g}\cdot M_u$ shall be considered to be transferred by eccentricity of shear about the centroid of the critical section defined in 11.12.1.2 where

$$\gamma_v = 1 - \gamma_f \qquad (11\text{-}41)$$

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 11.12.1.2. The maximum shear stress due to the factored shear force and moment shall not exceed $\mathbf{f}\,v_n$:

For members without shear reinforcement

$$\phi v_n = \phi V_c/(b_od) \qquad (11\text{-}42)$$

where V_c is as defined in 11.12.2.1 or 11.12.2.2.

For members with shear reinforcement other than shearheads:

$$\phi v_n = \phi(V_c + V_s)/(b_od) \qquad (11\text{-}43)$$

where V_c and V_s are defined in 11.12.3. If shear reinforcement is provided, the design shall take into account the variation of shear stress around the column.

11.12.6.3 When shear reinforcement consisting of 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 11.12.1.2 shall not exceed $\phi 4\sqrt{f_c'}$.

CHAPTER 12—DEVELOPMENT AND SPLICES OF REINFORCEMENT

12.0—Notation

a = depth of equivalent rectangular stress block as defined in 10.2.7.1

A_b = area of an individual bar, in.^{2}

A_s = area of nonprestressed tension reinforcement, in.^{2}

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, in.^{2}

A_v = area of shear reinforcement within a distance s, in.^{2}

A_w = area of an individual wire to be developed or spliced, in.^{2}

b_w = web width, or diameter of circular section, in.

c = spacing or cover dimension, in. See 12.2.4

d = distance from extreme compression fiber to centroid of tension reinforcement, in.

d_b = nominal diameter of bar, wire, or prestressing strand, in.

f_c' = specified compressive strength of concrete, psi

$\sqrt{f_c'}$ = square root of specified compressive strength of concrete, psi

f_{ps} = stress in prestressed reinforcement at nominal strength, ksi

f_{se} = effective stress in prestressed reinforcement (after allowance for all prestress losses), ksi

f_y = specified yield strength of nonprestressed reinforcement, psi

f_{yt} = specified yield strength of transverse reinforcement, psi

h = overall thickness of member, in.

K_{tr} = transverse reinforcement index

= $\dfrac{A_{tr}f_{yt}}{1500sn}$ (constant 1500 carries the unit lb/in.^{2})

ℓ_a = additional embedment length at support or at point of inflection, in.

ℓ_d = development length, in.

= $\ell_{db} \times$ applicable modification factors

ℓ_{db} = basic development length, in.

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), in.

= $\ell_{hb} \times$ applicable modification factors

ℓ_{hb} = basic development length of standard hook in tension, in.

M_n = nominal moment strength at section, in.-lb

= $A_s f_y(d - a/2)$

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

s = maximum spacing of transverse reinforcement within ℓ_d center-to-center, in.

s_w = spacing of wire to be developed or spliced, in.

V_u = factored shear force at section

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.

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 100 psi.

12.2—Development of deformed bars and deformed wire in tension

12.2.1 Development length ℓ_d, in terms of diameter d_b for deformed bars and deformed wire in tension shall be determined from either 12.2.2 or 12.2.3, but ℓ_d shall not be less than 12 in.

12.2.2 For deformed bars or deformed wire, ℓ_d/d_b shall be as follows:

	No. 6 and smaller bars and deformed wires	No. 7 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	$\dfrac{\ell_d}{d_b} = \dfrac{f_y\alpha\beta}{25\sqrt{f_c'}}$	$\dfrac{\ell_d}{d_b} = \dfrac{f_y\alpha\beta}{20\sqrt{f_c'}}$
Other cases	$\dfrac{\ell_d}{d_b} = \dfrac{3f_y\alpha\beta}{50\sqrt{f_c'}}$	$\dfrac{\ell_d}{d_b} = \dfrac{3f_y\alpha\beta}{40\sqrt{f_c'}}$

12.2.3 For deformed bars or deformed wire, ℓ_d/d_b shall be:

$$\frac{\ell_d}{d_b} = \frac{3}{40} \frac{f_y}{\sqrt{f_c'}} \frac{\alpha\beta\gamma}{\left(\dfrac{c + K_{tr}}{d_b}\right)} \qquad (12\text{-}1)$$

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:

a = reinforcement location factor

Horizontal reinforcement so placed that more than 12 in. of fresh concrete is cast in the member below the development length or splice 1.3

Other reinforcement 1.0

b = coating factor

Epoxy-coated bars or wires with cover less than $3d_b$, or clear spacing less than $6d_b$ 1.5

All other epoxy-coated bars or wires 1.2

Uncoated reinforcement 1.0

However, the product of **ab** need not be taken greater than 1.7.

γ = reinforcement size factor

No. 6 and smaller bars and deformed wires 0.8

No. 7 and larger bars 1.0

c = spacing or cover dimension, in.

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

= $\dfrac{A_{tr}f_{yt}}{1500sn}$

where

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, in.^{2}

f_{yt} = specified yield strength of transverse reinforcement, psi

s = maximum spacing of transverse reinforcement within ℓ_d, center-to-center, in.

n = number of bars or wires being developed along the plane of splitting

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 de-

velopment for f_y is specifically required or the reinforcement is designed under provisions of 21.2.1.4 .... $(A_s$ required$)/(A_s$ provided$)$

12.3—Development of deformed bars in compression

12.3.1 Development length ℓ_d, in inches, for deformed bars in compression shall be computed as the product of the basic development length ℓ_{db} of 12.3.2 and applicable modification factors of 12.3.3, but ℓ_d shall be not less than 8 in.

12.3.2—Basic development length

ℓ_{db} shall be .... $0.02\, d_b f_y/\sqrt{f_c'}$

but not less than .... $0.0003 d_b f_y$*

12.3.3 Basic development length ℓ_{db} shall be permitted to be multiplied by applicable factors for:

12.3.3.1—Excess reinforcement

Reinforcement in excess of that required by analysis .... $(A_s$ required$)/(A_s$ provided$)$

12.3.3.2—Spirals and ties

Reinforcement enclosed within spiral reinforcement not less than 1/4 in. diameter and not more than 4 in. pitch or within No. 4 ties in conformance with 7.10.5 and spaced at not more than 4 in. on center 0.75

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% for three-bar bundle, and 33% for four-bar bundle.

12.4.2 For determining the appropriate factors in 12.2, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

12.5—Development of standard hooks in tension

12.5.1 Development length ℓ_{dh}, in inches, for deformed bars in tension terminating in a standard hook (see 7.1) shall be computed as the product of the basic development length ℓ_{hb} of 12.5.2 and the applicable modification factor or factors of 12.5.3, but ℓ_{dh} shall not be less than $8d_b$ nor less than 6 in.

12.5.2—Basic development length

ℓ_{hb} for a hooked bar with f_y equal to 60,000 psi shall be .... $1200 d_b/\sqrt{f_c'}$ *

12.5.3 Basic development length ℓ_{hb} shall be multiplied by applicable factor or factors for:

12.5.3.1—Bar yield strength

Bars with f_y other than 60,000 psi .... $f_y/60{,}000$

12.5.3.2—Concrete cover

For No. 11 bar and smaller, side cover (normal to plane of hook) not less than 2-1/2 in., and for 90 degree hook, cover on bar extension beyond hook not less than 2 in. 0.7

12.5.3.3—Ties or stirrups

For No. 11 bar and smaller, hook enclosed vertically or horizontally within ties or stirrup-ties spaced along the

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* The constant carries the unit of in.^{2}/lb
* Constant carries unit of lb/in.^{2}

full development length ℓ_{dh} not greater than $3d_b$, where d_b is diameter of hooked bar 0.8

12.5.3.4—Excess reinforcement

Where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis .... $(A_s$ required$)/(A_s$ provided$)$

12.5.3.5—Epoxy-coated reinforcement

Hooked bars with epoxy coating 1.2

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 2-1/2 in., hooked bar shall be enclosed within ties or stirrup-ties spaced along the full development length ℓ_{dh} not greater than $3d_b$, where d_b is diameter of hooked bar. For this case, factor of 12.5.3.3 shall not apply.

12.5.5 Hooks shall not be considered effective in developing bars in compression.

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 Mechanical anchorages shall be designed in accordance with Appendix B—Steel Embedments.

12.7—Development of welded deformed wire fabric in tension

12.7.1 Development length ℓ_d, in inches, 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 12.7.2 or 12.7.3. It shall be permitted to reduce the development length in accordance with 12.2.5 when applicable, but ℓ_d shall not be less than 8 in. except in computation of lap splices by 12.18. When using the wire fabric factor from 12.7.2, it shall be permitted to use an epoxy-coating factor **b** of 1.0 for epoxy-coated welded wire fabric in 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 2 in. from the point of the critical section, the wire fabric factor shall be the greater of:

$$\left(\frac{f_y - 35{,}000}{f_y} \right)$$

or

$$\left(\frac{5d_b}{s_w} \right)$$

but need not be taken 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 2 in. 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.

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 2 in. from the point of the critical section. However, the development length ℓ_d, in inches, measured from the point of the critical section to the outermost cross wire shall not be less than

$$0.27 \frac{A_w}{s_w} \left(\frac{f_y}{\sqrt{f_c'}} \right)$$

except that when reinforcement provided is in excess of that required, this length may be reduced in accordance with 12.2.5. ℓ_d shall not be less than 6 in. except in computation of lap splices by 12.19.

12.9—Development of prestressing strand

12.9.1 Three- or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length, in inches, not less than

$$\left(f_{ps} - \frac{2}{3} f_{se} \right) d_b \quad {}^†$$

where d_b is strand diameter in inches, and f_{ps} and f_{se} are expressed in kips/in.^{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.

12.9.3 Where bonding of a strand does not extend to end of member, and design includes tension at service load in precompressed tensile zone as permitted by 18.4.2, development length specified in 12.9.1 shall be doubled.

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 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 be-

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† Expression in parenthesis used as a constant without units

yond 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 one of the following conditions is satisfied:

12.10.5.1 Shear at the cutoff point does not exceed two-thirds that permitted, including shear strength of shear reinforcement provided.

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 $60b_w s/f_y$. Spacing s shall not exceed $d/8\boldsymbol{b}_b$ where $\boldsymbol{b}_b$ is the ratio of area of reinforcement cut off to total area of tension reinforcement at the section.

12.10.5.3 For No. 11 bar and smaller, continuing reinforcement provides double the area required for flexure at the cutoff point and shear does not exceed three-fourths that permitted.

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 12.11.4 and 12.12.4 for deep flexural members.

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 6 in.

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 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. (12-2); except, Eq. (12-2) 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.

$$l_d \leq \frac{M_n}{V_u} + l_a \qquad (12\text{-}2)$$

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; and

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% 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 flexural members, positive moment tension reinforcement shall be anchored to develop the specified yield strength f_y in tension at the face of support. At interior supports of deep flexural members, positive moment tension reinforcement shall be continuous or be spliced with that of the adjacent spans.

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 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.

12.13—Development of web reinforcement

12.13.1 Web reinforcement shall be carried as close to compression and tension surfaces of member as cover requirements and proximity of other reinforcement will permit.

12.13.2 Ends of single leg, simple U-, or multiple U-stirrups shall be anchored by one of the following means:

12.13.2.1 For No. 5 bar and D31 wire, and smaller, and for No. 6, No. 7, and No. 8 bars with f_y of 40,000 psi or less, a standard hook around longitudinal reinforcement.

12.13.2.2 For No. 6, No. 7, and No. 8 stirrups with f_y greater than 40,000 psi, 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 $0.014d_b f_y/\sqrt{f_c'}$.

12.13.2.3 For each leg of welded plain wire fabric forming simple U-stirrups, either:

(a) Two longitudinal wires spaced at a 2 in. spacing along the member at the top of the U.

(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 2 in. 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 2 in. and with the inner wire at least the greater of $d/4$ or 2 in. from middepth of member $d/2$. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

12.13.2.5 In joist construction as defined in 8.11, for No. 4 bar and D20 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 middepth $d/2$ as specified for development length in 12.2 for that part of f_y required to satisfy Eq. (11-19).

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 18 in. deep, such splices with A_bf_y not more than 9000 lb per leg shall be considered adequate if stirrup legs extend the full available depth of member.

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 No. 11 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 6 in.

12.14.3—*Welded splices and mechanical connections*

12.14.3.1 Welded splices and other mechanical connections are allowed.

12.14.3.2 Except as provided in this code, all welding shall conform to "Structural Welding Code—Reinforcing Steel" (ANSI/AWS D1.4).

12.14.3.3 A full welded splice shall develop at least 125% of the specified yield strength f_y of the bar.

12.14.3.4 A full mechanical connection shall develop in tension or compression, as required, at least 125% of specified yield strength f_y of the bar.

12.14.3.4.1 Mechanical connections shall be qualified for use in the construction on the basis of the following performance tests:

(a) *Static Tensile Strength Tests*—A minimum of six static tensile strength tests shall be conducted considering the range of variabilities in splicing material, in material of reinforcing bars and in the anticipated environmental conditions. All test samples shall meet the requirement of 12.14.3.4.

(b) *Cyclic Tests*—Three specimens of the bar-to-bar connection for each reinforcing bar size and grade shall be subjected to 100 cycles of tensile stress variations from 5 to 90% of the specified minimum yield strength of the reinforcing bar. The specimens shall withstand the cyclic test without loss of static tensile strength capacity when compared with like specimen in (a) and tested statically to failure following cyclic tests.

12.14.3.5 Welded splices and mechanical connections not meeting requirements of 12.14.3.3 or 12.14.3.4 are allowed only for No. 5 bars and smaller and in accordance with 12.15.4.

12.14.3.6 All welded splices and mechanical connections shall be visually examined by a qualified and experienced inspector to assure that they are properly installed at the place of construction. Where it is deemed necessary, the engineer shall be permitted to require the destructive tests of production splices to assure the compliance with 12.14.3.3 and 12.14.3.4.

12.14.3.7 Mechanical connections shall be staggered if the strain measured over the full length of connection (at 0.9 yield) exceeds that of a bar that is not mechanically connected by more than 50% and if the maximum computed factored load stress in the bar equals or exceeds $0.5f_y$. If staggered mechanical connections are required, no more than one-half of the bars shall be connected in one plane normal to the bars and the mechanical connections shall be staggered at least 36 in.

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 12 in., 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 Welded splices or mechanical connections used where area of reinforcement provided is less than twice that required by analysis shall meet requirements of 12.14.3.3 or 12.14.3.4.

12.15.4 Welded splices and mechanical connections not meeting the requirements of 12.14.3.3 or 12.14.3.4 are allowed for No. 5 bars and smaller when the area of reinforcement provided is at least twice that required by analysis, and the following requirements are met:

12.15.4.1 Splices shall be staggered at least 24 in. and in such manner as to develop at every section at least twice the calculated tensile force at that section but not less than 20,000 psi for total area of reinforcement provided.

12.15.4.2 In computing tensile forces developed at each section, rate the spliced reinforcement at the specified splice strength. Unspliced reinforcement shall be rated at that fraction of f_y defined by the ratio of the shorter actual development length to ℓ_d required to develop the specified yield strength f_y.

12.15.5 Splices in "tension tie members" shall be made with a full welded splice or full mechanical connection in accordance with 12.14.3.3 or 12.14.3.4 and splices in adjacent bars shall be staggered at least 30 in.

12.15.6 Welded splices or mechanical connections shall be used for connecting tension resisting, but not crack-controlling, reinforcing bars located in a region with membrane tension normal to the welded splice or mechanical connection. The average strength of these welded splices or mechanical connections shall be equal to the minimum ultimate strength of the bar.

12.16—Splices of deformed bars in compression

12.16.1 Compression lap splice length shall be $0.0005f_y d_b$, for f_y of 60,000 psi or less, or $(0.0009f_y – 24)d_b$ for f_y greater than 60,000 psi, but not less than 12 in. For $f_c{'}$ less than 3000 psi, length of lap shall be increased by one-third.

12.16.2 When bars of different size are lap spliced in compression, splice length shall be the larger of development length of larger bar, or splice length of smaller bar. Lap splices of No. 14 and No. 18 bars to No. 11 and smaller bars shall be permitted.

12.16.3 Welded splices or mechanical connections used in compression shall meet requirements of 12.14.3.3 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 1-1/2 degrees of a right angle to the axis of the bars and shall be fitted within 3 degrees 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.

12.17—Special splice requirements for columns

12.17.1 Lap splices, butt welded splices, mechanical connections, or end-bearing splices shall be used with the limitations of 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 12.16.1, 12.16.2, and, where applicable, to 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 $0.5f_y$ 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 $0.5f_y$ 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 $0.0015hs$, lap splice length shall be permitted to be multiplied by 0.83, but lap length shall not be less than 12 in. 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 12 in.

12.17.3—Welded splices or mechanical connectors in columns

Welded splices or mechanical connectors in columns shall meet the requirements of 12.14.3.3 or 12.14.3.4.

12.17.4—End-bearing splices in columns

End-bearing splices complying with 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 $0.25f_y$ times the area of the vertical reinforcement in that face.

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 8 in., and the overlap measured between outermost cross wires of each fabric sheet shall be not less than 2 in. 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.

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 the following.

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 2 in., nor less than $1.5\ell_d$, nor 6 in. ℓ_d shall be the development length for the specified yield strength f_y in accordance with 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 be not less than $1.5\ell_d$, nor 2 in. ℓ_d shall be the development length for the specified yield strength f_y in accordance with 12.8.

PART 5—STRUCTURAL SYSTEMS OR ELEMENTS

CHAPTER 13—TWO-WAY SLAB SYSTEMS

13.0—Notation

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, in.

b_2 = width of the critical section defined in 11.12.1.2 measured in the direction perpendicular to b_1, in.

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, in.

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, in.

C = cross-sectional constant to define torsional properties

$$= \mathbf{S}\left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3}$$

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

E_{cb} = modulus of elasticity of beam concrete

E_{cs} = modulus of elasticity of slab concrete

h = overall thickness of member, in.

I_b = moment of inertia about centroidal axis of gross section of beam as defined in 13.2.4.

I_s = moment of inertia about centroidal axis of gross section of slab

= $h^3/12$ times width of slab defined in notations α and $\mathsf{ß}_t$

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

ℓ_1 = length of span in direction that moments are being determined, measured center-to-center of supports

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.

M_o = total factored static moment

M_u = factored moment at section

V_c = nominal shear strength provided by concrete. See 11.12.2.1

V_u = factored shear force at section

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 of cross section

y = longer overall dimension of rectangular part of cross section

$\mathbf{a}$ = 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

$$= \frac{E_{cb}I_b}{E_{cs}I_s}$$

$\mathbf{a}_1$ = $\mathbf{a}$ in direction of ℓ_1

$\mathbf{a}_2$ = $\mathbf{a}$ in direction of ℓ_2

$\mathsf{ß}_t$ = 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

$$= \frac{E_{cb}C}{2E_{cs}I_s}$$

$\mathbf{g}_f$ = fraction of unbalanced moment transferred by flexure at slab-column connections. See 13.5.3.2.

$\mathbf{g}_v$ = fraction of unbalanced moment transferred by eccentricity of shear at slab-column connections

= $1 - \mathbf{g}_f$

$\mathbf{r}$ = ratio of nonprestressed tension reinforcement

$\mathbf{r}_b$ = reinforcement ratio producing balanced strain conditions

$\mathbf{f}$ = strength reduction factor

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 A slab system may be supported on columns or walls. If supported by columns, 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 capital or bracket and are oriented no greater than 45 degrees 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 9.5.3.

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 4 times the slab thickness.

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 6 in. 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, to 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 **a** greater than 1.0, special top and bottom slab reinforcement shall be provided at exterior corners in accordance with the following:

13.3.6.1 The special reinforcement in both top and bottom of slab shall be sufficient to resist a moment equal to the maximum positive moment (per foot of width) 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 corner 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 following:

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, thickness of 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 45 degrees or less.

13.3.8.4 For slabs in frames not braced against sidesway, and for slabs 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 splices 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 supports.

13.3.8.6 In slabs with shearheads and in lift-slab construction, 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.

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 considering 9.2 and 9.3, and that all serviceability conditions, including the specified 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 the following:

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.

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 11.12.5 shall be satisfied.

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 considering 9.2 and 9.3, and that all serviceability conditions, including specified 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 13.6 or the Equivalent Frame Method of 13.7 shall be permitted.

STRIP	TYPE OF BARS	LOCATION	MINIMUM PERCENT-A_s AT SECTION	WITHOUT DROP PANELS	WITH DROP PANELS
COLUMN STRIP	STRAIGHT BARS	TOP	50 / Remainder	d / b	e / b
COLUMN STRIP	STRAIGHT BARS	BOTTOM	50 / Remainder	3" Max. / Max. 0.125ℓ / 6"	Max. 0.125ℓ / 24 bar dia. or 12" Min. all bars / 6" / Edge of drop
COLUMN STRIP	BENT BARS *	TOP	50 / Remainder	d / b	b / e / e / b
COLUMN STRIP	BENT BARS *	BOTTOM	50 / Remainder	g / 6" / 3" Max.	g / Max. 0.125ℓ / 24 bar dia. or 12" Min. / 6" / Edge of drop
MIDDLE STRIP	STRAIGHT BARS	TOP	100	c	c
MIDDLE STRIP	STRAIGHT BARS	BOTTOM	50 / Remainder	3" Max. / 6" / Max. 0.15ℓ	3" Max. / Max. 0.15ℓ / 6"
MIDDLE STRIP	BENT BARS *	TOP	50 / Remainder	c (all bars) / a	c / a / c / c (all bars)
MIDDLE STRIP	BENT BARS *	BOTTOM	50 / Remainder	f / 6" / 3" Max.	f / 3" Max. / 6"

Clear span – $ℓ_n$
Face of support
Center to center span – $ℓ$
℄ Exterior support (No slab continuity) — ℄ Interior support (Continuity provided) — Exterior support ℄ (No slab continuity)

* Bent bars at exterior supports may be used if a general analysis is made

	BAR LENGTH FROM FACE OF SUPPORT						
	MINIMUM LENGTH					MAXIMUM LENGTH	
MARK	a	b	c	d	e	f	g
LENGTH	$0.14ℓ_n$	$0.20ℓ_n$	$0.22ℓ_n$	$0.30ℓ_n$	$0.33ℓ_n$	$0.20ℓ_n$	$0.24ℓ_n$

Fig. 13.4.8—Minimum bend point locations and extensions for reinforcement in slabs without beams (See 12.11.1 for reinforcement extension into supports).

13.5.1.2 For lateral loads, analysis of unbraced 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.

13.5.2 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 13.5.3.2 and 13.5.3.3.

13.5.3.1 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 $0.75\mathbf{f}V_c$ or at a corner support does not exceed $0.5\mathbf{f}V_c$. 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% provided that V_u at the support does not exceed $0.4\mathbf{f}V_c$. The reinforcement ratio $\mathbf{r}$, within the effective slab width defined in 13.5.3.2, shall not exceed $0.375\mathbf{r}_b$. 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 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.

13.6—Direct design method
13.6.1—Limitations

Design of slab systems within the following limitations 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% 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

$$\frac{\alpha_1 \ell_2^2}{\alpha_2 \ell_1^2} \qquad (13\text{-}2)$$

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 span

13.6.2.1 Total factored static moment for a span shall be determined in a strip bounded laterally by center line 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

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} \qquad (13\text{-}3)$$

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_n 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)
			Slab without beams between interior supports		
	Exterior edge unre-strained	Slab with beams between all supports	Without edge beam	With edge beam	Exterior edge fully restrained
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 $0.3M_o$.

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% 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 0%.

13.6.5.3 In addition to moments calculated for uniform loads according to 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 an edge supported by a wall 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% 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 bounded by 45 degree 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 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 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 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.

$$M = 0.07[(w_d + 0.5w_\ell)\ell_2\ell_n^2 - w'_d\ell'_2(\ell'_n)^2] \qquad (13\text{-}4)$$

where w_d, ℓ'_2 and ℓ'_n refer to shorter span.

13.7—Equivalent frame method

13.7.1 Design of slab systems by the Equivalent Frame Method shall be based on assumptions given in 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 Neglecting the change in length of columns and slabs due to direct stress, and deflections due to shear, shall be permitted.

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 (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 infinite.

13.7.5—Torsional members

13.7.5.1 Torsional members (13.7.2.3) shall be assumed to have a constant cross section throughout their length consisting of the largest of

(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; and

(c) The transverse beam as defined in 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 moment of inertia of slab with such beam to moment of inertia of slab without such beam.

13.7.6—Arrangement of live load

13.7.6.1 When 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 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 13.7.6.2, it shall be permitted to assume that maximum positive factored moment near midspan of a panel occurs with three-quarters 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 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, critical section for negative factored moment (in both column and middle strips) shall be taken at face of rectilinear supports, but not greater than $0.175\ell_1$ from center of a column.

13.7.7.2 At exterior supports provided with brackets or capitals, 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 13.6.4, 13.6.5, and 13.6.6 shall be permitted if the requirement of 13.6.1.6 is satisfied.

CHAPTER 14—WALLS

14.0—Notation

A_g = gross area of section, in.^{2}

f'_c = specified compressive strength of concrete, psi

h = overall thickness of member, in.

k = effective length factor

ℓ_c = vertical distance between supports, in.

P_{nw} = nominal axial load strength of wall designed by 14.4

ϕ = strength reduction factor. See 9.3.

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 in accordance with the flexural design provisions of Chapter 10. The minimum horizontal reinforcement shall be in accordance with 14.3.3 but shall not be less than that required by 7.12.

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 14.2, 14.3, and either 14.4 or 14.5.

14.2.3 Design for shear shall be in accordance with 11.10.

14.2.4 Unless demonstrated by a detailed analysis, horizontal length of wall to be considered as effective for each concentrated load shall not exceed center-to-center distance between loads, nor width of bearing 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, roofs, or to columns, pilasters, buttresses, and intersecting walls, and 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.

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 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 No. 5 with specified yield strength not less than 60,000 psi; or

(b) 0.0015 for other deformed bars; or

(c) 0.0012 for welded wire fabric (plain or deformed) not larger than W31 or D31.

14.3.3 Minimum ratio of horizontal reinforcement area to gross concrete area shall be:

(a) 0.0020 for deformed bars not larger than No. 5 with a specified yield strength not less than 60,000 psi; or

(b) 0.0025 for other deformed bars; or

(c) 0.0020 for welded wire fabric (plain or deformed) not larger than W31 or D31.

14.3.4 Walls more than 10 in. 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 2 in. nor more than one-third the thickness of wall from exterior surface; and

(b) The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than 3/4 in. nor more than one-third the thickness of wall from interior surface.

14.3.5 Vertical and horizontal reinforcement shall not be spaced farther apart than three times the wall thickness, nor 18 in.

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 14.3.1, not less than two No. 5 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 24 in.

14.4—Walls designed as compression members

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 10.2, 10.3, 10.10, 10.11, 10.12, 10.13, 10.14, 10.15, 10.17, and 14.2 and 14.3.

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 14.5 if resultant of all factored loads is located within the middle-third of the overall thickness of 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 top and bottom against lateral translation and

    (a) restrained against rotation at one or both ends (top and/or bottom) 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 unsupported height or length, whichever is shorter, nor less than 4 in.

    14.5.3.2 Thickness of exterior basement walls and foundation walls shall not be less than 7-1/2 in.

14.6—Nonbearing walls

14.6.1 Thickness of non-bearing walls shall not be less than 4 in., nor less than 1/30 the least distance between members that provide lateral support.

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 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.

CHAPTER 15—FOOTINGS

15.0—Notation

A_g = gross area of section, in.^{2}
d_p = diameter of pile at footing base
ß = ratio of long side to short side of footing

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.

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 this code 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 selected through principles of soil mechanics.

15.2.3 For footings on piles, computations for moments and shears may be based on the assumption that the reaction from any pile is concentrated at pile center.

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.

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 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; and

(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 as follows:

(a) Reinforcement in long direction shall be distributed uniformly across entire width of footing; and

(b) 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 center line 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.

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

15.5—Shear in footings

15.5.1 Computation of shear in footings 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 Computation of shear on any section through a footing supported on piles shall be in accordance with the following:

(a) 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;

(b) 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; and

(c) 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.

15.6—Development of reinforcement in footings

15.6.1 Computation of 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.

15.7—Minimum footing depth

Depth of footing above bottom reinforcement shall not be less than 6 in. for footings on soil, nor less than 12 in. for footings on piles.

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 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; and

(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, 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, No. 14 and 18 longitudinal bars, in compression only, may be lap spliced with dowels to provide reinforcement required to satisfy 15.8.1. Dowels shall not be larger than No. 11 bar and shall extend into supported member a distance not less than the development length of No. 14 or No. 18 bars 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.

15.8.3.1 Connection between precast columns or pedestals and supporting member shall meet the requirements of 16.5.1.3(a).

15.8.3.2 Connection between precast wall and supporting member shall meet the requirements of 16.5.1.3(b) and (c).

15.8.3.3 Anchor bolts and mechanical connectors shall be designed to reach their design strength prior to anchorage failure or failure of surrounding concrete.

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.

15.9.2 Sloped or stepped footings designed as a unit shall be constructed to assure action as a unit.

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 this 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

16.0—Notation
A_g = gross area of column, in.^{2}
ℓ = clear span, in.

16.1—Scope

16.1.1 All provisions of this Code, not specifically excluded and not in conflict with provisions of Chapter 16, shall apply

to structures incorporating precast concrete structural members.

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, the following 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; and

(b) Required concrete strength at stated ages or stages of construction.

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, the following 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.

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 12 ft, 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 shall each be not less than 0.001 times the gross cross-sectional area of the wall panel. Spacing of reinforcement shall not exceed 5 times the wall thickness or 30 in. for interior walls or 18 in. for exterior walls.

16.5—Structural integrity

16.5.1 Except where the provisions of 16.5.2 govern, the following minimum provisions for structural integrity shall apply to all precast concrete structures:

16.5.1.1 Longitudinal and transverse ties required by 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 300 lb per lin ft.

16.5.1.3 Vertical tension tie requirements of 7.13.3 shall apply to all vertical structure members, except cladding, and shall be achieved by providing connections at horizontal joints in accordance with the following:

(a) Precast columns shall have a nominal strength in tension not less than 200 A_g in pounds. 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 10,000 lb per tie; and

(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 following minimum provisions shall apply:

16.5.2.1 Longitudinal and transverse ties shall be provided in floor and roof systems to provide a nominal strength of 1500 lb per foot 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 2 ft 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 10 ft 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 4 ft of the edge, shall provide a nominal strength in tension not less than 16,000 lb.

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 3000 lb per horizontal foot of wall. Not less than two ties shall be provided for each precast panel.

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 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 the following:

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, the following minimum requirements 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 .... 2 in.
    For beams or stemmed members .... 3 in.

(b) Bearing pads at unarmored edges shall be set back a minimum of 1/2 in. from the face of the support, or at least the chamfer dimension at chamfered edges.

16.6.2.3 The requirements of 12.11.1 shall not apply to the positive bending moment reinforcement for statically determinate precast members, but at least one-third of such reinforcement shall extend to the center of the bearing length.

16.7—Items embedded after concrete placement

16.7.1 When approved by the engineer, 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:

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.

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.

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 Precast members and structures shall be adequately supported and braced during erection to ensure proper alignment and structural integrity until permanent connections ar completed.

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 the following:

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 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

17.0—Notation

A_c =   area of contact surface being investigated for horizontal shear, in.^{2}

A_v =   area of ties within a distance s, in.^{2}

b_v =   width of cross section at contact surface being investigated for horizontal shear

d =   distance from extreme compression fiber to centroid of tension reinforcement for entire composite section, in.

h =   overall thickness of composite member, in.

s =   spacing of ties measured along the longitudinal axis of the member, in.

V_{nh} =   nominal horizontal shear strength

V_u =   factored shear force at section

$\mathbf{r}_v$ =   ratio of tie reinforcement area to area of contact surface

    = $A_v / b_v s$

$\mathbf{f}$ =   strength reduction factor. See 9.3.

17.1—Scope

17.1.1 Provisions of Chapter 17 shall apply for design of composite concrete flexural members defined as precast and/or cast-in-place concrete elements constructed in separate placements but so interconnected that all elements respond to loads as a unit.

17.1.2 All provisions of this code shall apply to composite concrete flexural members, except as specifically modified in Chapter 17.

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 control 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 9.5.5.

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.

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 12.13.

17.4.3 Extended and anchored shear reinforcement shall be permitted as ties for horizontal shear.

17.5—Horizontal shear strength

17.5.1 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

17.5.2 Unless calculated in accordance with 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 section considered and V_{nh} is nominal horizontal shear strength in accordance with the following.

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 $80b_v d$ in pounds.

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 $80b_v d$ in pounds.

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 1/4 in., shear strength V_{nh} shall be taken equal to $(260 + 0.6\mathbf{r}_v f_y) b_v d$ in pounds, but not greater than $500b_v d$ in pounds.

17.5.2.4 When factored shear force V_u at section considered exceeds $\mathbf{f}(500\,b_v d)$, design for horizontal shear shall be in accordance with 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 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 $\mathbf{f}V_{nh}$ as given in 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 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.

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 4 times the least dimension of supported element, nor 24 in.

17.6.2 Ties for horizontal shear may 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

18.0—Notation

A = area of that part of cross section between flexural tension face and center of gravity of gross section, in.^{2}

A_{ps} = area of prestressed reinforcement in tension zone, in.^{2}

A_s = area of nonprestressed tension reinforcement, in.^{2}

A_s' = area of compression reinforcement, in.^{2}

b = width of compression face of member, in.

d = distance from extreme compression fiber to centroid of nonprestressed tension reinforcement, in.

d' = distance from extreme compression fiber to centroid of compression reinforcement, in.

d_p = distance from extreme compression fiber to centroid of prestressed reinforcement

D = dead loads, or related internal moments and forces

e = base of Napierian logarithms

f_c' = specified compressive strength of concrete, psi

$\sqrt{f_c'}$ = square root of specified compressive strength of concrete, psi

f_{ci}' = compressive strength of concrete at time of initial prestress, psi

$\sqrt{f_{ci}'}$ = square root of compressive strength of concrete at time of initial prestress, psi

f_{pc} = average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses), psi

f_{ps} = stress in prestressed reinforcement at nominal strength, psi

f_{pu} = specified tensile strength of prestressing tendons, psi

f_{py} = specified yield strength of prestressing tendons, psi

f_r = modulus of rupture of concrete, psi

f_{se} = effective stress in prestressed reinforcement, (after allowance for all prestress losses), psi

f_y = specified yield strength of nonprestressed reinforcement, psi

h = overall thickness of member, in.

K = wobble friction coefficient per foot of prestressing tendons

ℓ = length of span of two-way flat plates in direction parallel to that of the reinforcement being determined, in. See Eq. (18-8).

ℓ_x = length of prestressing tendon element from jacking end to any point x, ft. See Eq. (18-1) and (18-2)

L = live loads, or related internal moments and forces

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

P_s = prestressing tendon force at jacking end

P_x = prestressing tendon force at any point x

α = total angular change of prestressing tendon profile in radians from tendon jacking end to any point x

β_1 = factor defined in 10.2.7

g_p = factor for type of prestressing tendon
  = 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

m = curvature friction coefficient

r = ratio of nonprestressed tension reinforcement
  = A_s/bd

r' = ratio of compression reinforcement
  = A_s'/bd

r_p = ratio of prestressed reinforcement
  = A_{ps}/bd_p

f = strength reduction factor. See 9.3.

w = $\rho f_y/f_c'$

w' = $\rho' f_y/f_c'$

w_p = $\rho_p f_{ps}/f_c'$

w_w, w_{pw}, w_w' = reinforcement indices for flanged sections computed as for w, w_p, and w' except that b shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only

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 tendons in 3.5.5.

18.1.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of Chapter 18, shall apply to prestressed concrete.

18.1.3 The following provisions of this code shall not apply to prestressed concrete, except as specifically noted: Sections 7.6.5, 8.4, 8.10.2, 8.10.3, 8.10.4, 8.11, 10.3.2, 10.3.3, 10.5, 10.6, 10.9.1, 10.9.2; Chapter 13; and Sections 14.3, 14.5, and 14.6.

18.1.4 Service load conditions are defined to be Load Combinations 1, 2, 3, 9, 10, and 11 of 9.2.1 with the load factors taken as unity.

18.2—General

18.2.1 Prestressed members shall meet the strength requirements specified in this code.

18.2.2 Design of prestressed members shall be based on strength and on behavior at service conditions at all load stages that may 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 Possibility of buckling in a member between points where concrete and prestressing tendons are in contact and of buckling in thin webs and flanges shall be considered.

18.2.6 In computing section properties prior to bonding of prestressing tendons, effect of loss of area due to open ducts shall be considered.

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 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, straight-line theory may be used with the following assumptions.

18.3.2.1 Strains vary linearly with depth through entire load range.

18.3.2.2 At cracked sections, concrete resists no tension.

18.4—Permissible stresses in concrete: 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 ....0.60f_{ci}'

(b) Extreme fiber stress in tension
    except as permitted in (c).................................. $3\sqrt{f_{ci}'}$

(c) Extreme fiber stress in tension at ends of simply supported members .... $6\sqrt{f_{ci}'}$

Where computed tensile stresses exceed these values, bonded auxiliary 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 Stresses in concrete at service loads (after allowance for all prestress losses) shall not exceed the following:

(a) Extreme fiber stress in compression
    Load Combinations 1, 2, 3 ....0.45f_c'
    Load Combinations 9, 10, 11 ....0.60f_c'

(b) Extreme fiber stress in tension in
    precompressed tensile zone................................. $6\sqrt{f_c'}$

(c) Extreme fiber stress in tension in precompressed tensile zone of members (except two-way slab systems) where analysis based on transformed cracked sections and on bilinear moment-deflection relationships shows that immediate and long-time deflections comply with requirements of 9.5.4, and where cover requirements comply with 7.7.3.2 .... $12\sqrt{f_c'}$

18.4.3 Permissible stresses in concrete of 18.4.1 and 18.4.2 may be exceeded if shown by test or analysis that performance will not be impaired.

18.5—Permissible stresses in prestressing tendons

18.5.1 Tensile stress in prestressing tendons shall not exceed the following:

(a) Due to tendon jacking force, $0.94f_{py}$ but not greater than the lesser of $0.80f_{pu}$ and the maximum value recommended by manufacturer of prestressing tendons or anchorages;

(b) Immediately after prestress transfer, $0.82f_{py}$ but not greater than $0.74f_{pu}$; and

(c) Post-tensioning tendons, at anchorages and couplers, immediately after tendon anchorage, $0.70f_{pu}$.

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) Anchorage seating loss;

(b) Elastic shortening of concrete;

(c) Creep of concrete;

(d) Shrinkage of concrete;

(f) Relaxation of tendon stress; and

(g) 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 permitted to be computed by

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

When $(K\ell_x + \mu\alpha)$ is not greater than 0.3, effect of friction loss may be computed by

$$P_s = P_x(1 + K\ell_x + \mu\alpha) \qquad (18\text{-}2)$$

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 may occur due to connection of member to adjoining construction, such loss of prestress shall be allowed for in design.

18.7—Flexural strength

18.7.1 Design moment strength of flexural members shall be computed by the strength design methods of this code. For prestressing tendons, 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 used if f_{se} is not less than $0.5f_{pu}$

(a) For members with bonded prestressing tendons

$$f_{ps} = f_{pu}\left\{1 - \frac{\gamma_p}{\beta_1}\left[\rho_p\frac{f_{pu}}{f_c'} + \frac{d}{d_p}(\omega - \omega')\right]\right\} \qquad (18\text{-}3)$$

If any compression reinforcement is taken into account when calculating f_{ps} by Eq. (18-3), the term

$$\left[\rho_p\frac{f_{pu}}{f_c'} + \frac{d}{d_p}(\omega - \omega')\right] \qquad (18.7.2)$$

shall be taken not less than 0.17 and d' shall be no greater than $0.15d_p$;

(b) For members with unbonded prestressing tendons and with a span-to-depth ratio of 35 or less

$$f_{ps} = f_{se} + 10{,}000 + \frac{f_c'}{100\rho_p} \qquad (18\text{-}4)$$

but f_{ps} in Eq. (18-4) shall not be taken greater than f_{py} nor $(f_{se} + 60{,}000)$; and

(c) For members with unbonded prestressing tendons and with a span-to-depth ratio greater than 35

$$f_{ps} = f_{se} + 10{,}000 + \frac{f_c'}{300\rho_p} \qquad (18\text{-}5)$$

but f_{ps} in Eq. (18-5) shall not be taken greater than f_{py}, nor $(f_{se} + 30{,}000)$.

18.7.3 Nonprestressed reinforcement conforming to 3.5.3, if used with prestressing tendons, 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 made to determine stresses in such reinforcement.

18.8—Limits for reinforcement of flexural members

18.8.1 Ratio of prestressed and nonprestressed reinforcement used for computation of moment strength of a member, except as provided in 18.8.2, shall be such that ω_p, $[\omega_p + d/d_p(\omega - \omega')]$ or $[\omega_{pw} + d/d_p(\omega_w - \omega'_w)]$ is not greater than $0.36\beta_1$.

18.8.2 When a reinforcement ratio in excess of that specified in 18.8.1 is provided, design moment strength shall not exceed the moment strength based on the compression portion of the moment couple.

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_t specified in 9.5.2.3, except for flexural members with shear and flexural strength at least twice that required by 9.2.

18.9—Minimum bonded reinforcement

18.9.1 A minimum area of bonded reinforcement shall be provided in all flexural members with unbonded prestressing tendons as required by 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 precompressed 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 plates, defined as solid slabs of uniform thickness, minimum area and distribution of bonded reinforcement shall be as follows.

18.9.3.1 Bonded reinforcement shall not be required in positive moment areas where computed tensile stress in concrete at service load (after allowance for all prestress losses) does not exceed $2\sqrt{f_c'}$.

18.9.3.2 In positive moment areas where computed tensile stress in concrete at service load exceeds $2\sqrt{f_c'}$, minimum area of bonded reinforcement shall be computed by

$$A_s = \frac{N_c}{0.5f_y} \qquad (18\text{-}7)$$

where design yield strength f_y shall not exceed 60,000 psi. Bonded reinforcement shall be uniformly distributed over precompressed tensile zone as close as practicable to extreme tension fiber.

18.9.3.3 In negative moment areas at column supports, minimum area of bonded reinforcement in each direction shall be computed by

$$A_s = 0.00075h\ell \qquad (18\text{-}8)$$

where ℓ is length of span in direction parallel to that of the reinforcement being determined. Bonded reinforcement required by Eq. (18-8) shall be distributed within a slab width between lines that are $1.5h$ outside opposite faces of the column support. At least 4 bars or wires shall be provided in each direction. Spacing of bonded reinforcement shall not exceed 12 in.

18.9.4 Minimum length of bonded reinforcement required by 18.9.2 and 18.9.3 shall be as follows.

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 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.

18.10—Statically indeterminate structures

18.10.1 Statically indeterminate structures 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 produced by prestressing, creep, shrinkage, temperature change, axial deformation, restraint of attached structural elements, and foundation settlement.

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 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 18.9.2, negative moments calculated by elastic theory for any assumed loading arrangement, shall be permitted to be increased or decreased by not more than

$$20\left[1 - \frac{\omega_p + \dfrac{d}{d_p}(\omega - \omega')}{0.36\beta_1}\right] \quad \%$$

18.10.4.2 The modified negative moments shall be used for calculating moments at sections within spans for the same loading arrangement.

18.10.4.3 Redistribution of negative moments shall be made only when the section at which moment is reduced is so designed that $\mathbf{w}_p$, $[\mathbf{w}_p + d/d_p(\mathbf{w} - \mathbf{w}')]$, or $[\mathbf{w}_{pw} + d/d_p (\mathbf{w}_w - \mathbf{w}'_w)]$, whichever is applicable, is not greater than $0.24\beta_1$.

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 this code for members without prestressing. Effects of prestress, creep, shrinkage, and temperature change shall be included.

18.11.2—Limits for reinforcement of prestressed compression members

18.11.2.1 Members with average prestress f_{pc} less than 225 psi shall have minimum reinforcement in accordance with 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 225 psi shall have all prestressing tendons enclosed by spirals or lateral ties in accordance with the following:

(a) Spirals shall conform to 7.10.4;

(b) Lateral ties shall be at least No. 3 in size or welded wire fabric of equivalent area, and spaced vertically not to exceed 48 tie bar or wire diameters, or least dimension of 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 shall be spaced as provided herein to not more than half a tie spacing below lowest horizontal reinforcement in members supported above; and

(d) Where beams or brackets frame into all sides of a column, it shall be permitted to terminate ties not more than 3 in. below lowest reinforcement in such beams or brackets.

18.11.2.3 For walls with average prestress f_{pc} equal to or greater than 225 psi, minimum reinforcement required by 14.3 may be waived where structural analysis shows adequate strength and stability.

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 Moment strength of prestressed slabs at every section shall be at least equal to the required strength considering 9.2, 9.3, 18.10.3, and 18.10.4. Shear strength of prestressed slabs at columns shall be at least equal to the required strength considering 9.2, 9.3, 11.1, 11.11.2, and 11.12.2.4.

18.12.3 At service load conditions, all serviceability limitations, including specified 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 prestressing tendons or groups of tendons in one direction shall not exceed 8 times the slab thickness, nor 5 ft. Spacing of tendons also shall provide a minimum average prestress, after allowance for all prestress losses, of 125 psi 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 prestressing 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 13.3.8.6.

18.13—Tendon anchorage zones

18.13.1 Reinforcement shall be provided where required in tendon anchorage zones to resist bursting, splitting, and spalling forces induced by tendon anchorages. Regions of abrupt change in section shall be adequately reinforced.

18.13.2 End blocks shall be provided where required for support bearing or for distribution of concentrated prestressing forces.

18.13.3 Post-tensioning anchorages and supporting concrete shall be designed to resist maximum jacking force for strength of concrete at time of prestressing.

18.13.4 Post-tensioning anchorage zones shall be designed to develop the guaranteed ultimate tensile strength of prestressing tendons using a strength reduction factor ϕ of 0.90 for concrete.

18.14—Corrosion protection for unbonded prestressing tendons

18.14.1 Unbonded tendons shall be completely coated with suitable material to ensure corrosion protection.

18.14.2 Tendon wrapping shall be continuous over entire length to be unbonded, and shall prevent intrusion of cement paste or loss of coating materials during concrete placement.

18.14.3 Unbonded single strand tendons shall be protected against corrosion in accordance with "Specification for Unbonded Single Strand Tendons," revised July 1993, published by the Post-Tensioning Institute.

18.15—Post-tensioning ducts

18.15.1 Ducts for grouted or unbonded tendons shall be mortar-tight and nonreactive with concrete, tendons, or filler material.

18.15.2 Ducts for grouted single wire, strand, or bar tendons shall have an inside diameter at least 1/4 in. larger than tendon diameter.

18.15.3 Ducts for grouted multiple wire, strand, or bar tendons shall have an inside cross-sectional area at least 2 times area of tendons.

18.15.4 Ducts shall be maintained free of water if members to be grouted are exposed to temperatures below freezing prior to grouting.

18.16—Grout for bonded prestressing tendons

18.16.1 Grout shall consist of portland cement and water; or portland cement, sand, and water.

18.16.2 Materials for grout shall conform to the following:

18.16.2.1 Portland cement shall conform to 3.2.

18.16.2.2 Water shall conform to 3.4.

18.16.2.3 Sand, if used, shall conform to "Standard Specifications for Aggregate for Masonry Mortar" (ASTM C 144) except that gradation shall be permitted to be modified as necessary to obtain satisfactory workability.

18.16.2.4 Admixtures conforming to 3.6 and known to have no injurious effects on grout, steel, or concrete may be used. Calcium chloride shall not be used.

18.16.3—Selection of grout proportions

18.16.3.1 Proportions of materials for grout shall be based on either of the following:

(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.

18.16.3.2 Cement used in the work shall correspond to that on which selection of grout proportions was based.

18.16.3.3 Water content shall be minimum necessary for proper pumping of grout; however, water-cement ratio shall not exceed 0.45 by weight.

18.16.3.4 Water shall not be added to increase grout flowability that has been decreased by delayed use of grout.

18.16.4—Mixing and pumping grout

18.16.4.1 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 tendon ducts.

18.16.4.2 Temperature of members at time of grouting shall be above 35 F and shall be maintained above 35 F until field-cured 2-in. cubes of grout reach a minimum compressive strength of 800 psi.

18.16.4.3 Grout temperatures shall not be above 90 F during mixing and pumping.

18.17—Protection for prestressing tendons

Burning or welding operations in vicinity of prestressing tendons shall be carefully performed, so that tendons are not subject to excessive temperatures, welding sparks, or ground currents.

18.18—Application and measurement of prestressing force

18.18.1 Prestressing force shall be determined by both of the following methods:

(a) Measurement of tendon elongation. Required elongation shall be determined from average load-elongation curves for prestressing tendons used; and

(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% for pretensioned elements or 7% for post-tensioned construction shall be ascertained and corrected.

18.18.2 Where transfer of force from bulkheads of pretensioning bed to concrete is accomplished by flame cutting prestressing tendons, cutting points and cutting sequence shall be predetermined to avoid undesired temporary stresses.

18.18.3 Long lengths of exposed pretensioned strand shall be cut near the member to minimize shock to concrete.

18.18.4 Total loss of prestress due to unreplaced broken tendons shall not exceed 2% of total prestress.

18.19—Post-tensioning anchorages and couplers

18.19.1 Anchorages and couplers for bonded and unbonded prestressing tendons shall develop at least 95% of the specified breaking strength of the tendons, when tested in an unbonded condition, without exceeding anticipated set. For bonded tendons, anchorages and couplers shall be located so that 100% of the specified breaking strength of the tendons shall be developed at critical sections after tendons are bonded in the member.

18.19.2 Couplers shall be placed in areas approved by the engineer and enclosed in housing long enough to permit necessary movements.

18.19.3 In unbonded construction subject to repetitive loads, special attention shall be given to the possibility of fatigue in anchorages and couplers.

18.19.4 Anchorage and end fittings shall be permanently protected against corrosion.

CHAPTER 19—SHELLS

19.0—Notation
f'_c = specified compressive strength of concrete, psi

19.1—Scope and definitions

19.1.1 Provisions of Chapter 19 shall apply for design of shell concrete structures having thicknesses equal to or greater than 12 in.

19.1.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of Chapter 19 shall apply to shell structures.

19.1.3 Shells may be bounded by supporting members and edge members provided to stiffen the shell and distribute or carry load in composite action with that shell.

19.2—General

19.2.1 Methods of analysis that are based on accepted principles of engineering mechanics and applicable to the geometry of the structure shall be used.

19.2.2 Elastic behavior shall be an accepted basis for determining internal forces, displacements, and stability of shells. Equilibrium checks of internal forces and external loads shall be made to ensure consistency of results.

19.2.3 The redistribution of forces in a statically indeterminate structure shall be considered.

19.2.4 The stiffening effect of buttresses or other integral portions of the structure shall be considered.

19.2.5 Shell elements shall be proportioned for the required strength in accordance with provisions of Chapter 9 of this Code.

19.2.6 Investigation of thin shells for stability shall include consideration of possible reduction in buckling capacity caused by large deflections, creep effects, temperature, cracking, and deviation between actual and theoretical shell surface.

19.2.7 The effect of openings or penetrations on the strength and behavior of the overall structure shall be considered. The shell shall be permitted to be thickened around the openings or penetrations if necessary to satisfy strength requirements and facilitate concrete placement.

19.2.8 Nonlinear variations in circumferential and meridional stresses across the shell thickness shall be considered when appropriate.

19.2.9—Supporting members

19.2.9.1 Supporting members shall be designed in accordance with applicable provisions of this code.

19.2.9.2 A portion of the shell equal to the effective flange width as specified in 8.10 shall be permitted to act with supporting members.

19.2.9.3 Within the effective flange width of shell assumed to act with supporting members, reinforcement perpendicular to supporting member shall be at least equal to that required for a T-beam flange as specified in 8.10.5.

19.2.9.4 Compatibility shall be maintained at the junction of the shell and the supporting member and at all locations of discontinuities in geometric and material properties which affect the shell stiffness.

19.2.10—Model tests

19.2.10.1 Model tests shall be permitted in support of the design if they are planned and executed by individuals or laboratories with experience in physical testing.

19.2.10.2 When model tests are used, only those portions of the shell structure that significantly affect items under study need be simulated.

19.2.10.3 Every attempt shall be made to ensure that elastic model tests reveal quantitative behavior of prototype structure.

19.3—Design strength of materials

Specified compressive strength of concrete f'_c at 28 days shall not be less than 3000 psi.

19.4—Section design and reinforcement requirements

19.4.1 Tensile strength of the concrete shall not be relied upon to resist flexural and membrane action.

19.4.2 Reinforcement shall be provided in two or more directions and shall be proportioned such that its resistance in any direction exceeds the component of applied forces in that direction.

19.4.3 Shell reinforcement required for flexure shall be proportioned with due regard to axial forces.

19.4.4 Reinforcement shall meet the minimum requirements of 7.12.

19.4.5 Shell reinforcement at junction of shell and supporting members or edge members shall be anchored in or through supporting members by embedment length, hooks, or mechanical anchorage in accordance with Chapter 12.

19.4.6 All forces imposed by curved reinforcement shall be considered in the design of local areas, such as around penetrations.

19.5—Construction

The 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.

PART 6—SPECIAL CONSIDERATIONS

CHAPTER 20—STRENGTH EVALUATION OF EXISTING STRUCTURES

20.0—Notation
D = dead loads or related internal moments and forces

f'_c = specified compressive strength of concrete, psi

h = overall thickness of member, in.

L = live loads or related internal moments and forces

ℓ_t = span of member under load test, in. (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, in.

Δ_{max} = measured maximum deflection, in. See Eq. (20-1)

Δ_{rmax} = measured residual deflection, in. See Eq. (20-2) and (20-3)

Δ_{fmax} = maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test, in. See Eq. (20-3)

20.1—Strength evaluation: General
20.1.1 If doubt develops concerning the safety of a structure or member, and/or low-strength concrete is confirmed in accordance with 5.6.4.4 and computations indicate that load-carrying capacity has been significantly reduced, the engineer may order a strength evaluation. (For approval of special systems of design or construction, see 1.4 regarding use of tests.)

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, the structure or part of the structure shall be permitted to remain in service for a specified time period. If deemed necessary by the engineer, periodic reevaluations shall be conducted.

20.2—Analytical investigations: General
20.2.1 If strength evaluation is by analysis, a thorough field investigation shall be made of dimensions and details of members, properties of materials, and other pertinent conditions of the structure as actually built.

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 5.6.4.

20.2.4 If required, reinforcement or tendon 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 and subject to the special requirement of 1.4, it shall be permitted to increase the strength reduction factor in 9.3, but the strength reduction factor shall not be more than:

Flexure, without axial load...1.0

Axial tension, and axial tension with flexure 1.0

Axial compression and axial compression with flexure:

   Members with spiral reinforcement conforming to 10.9.3..0.9

   Other members 0.85

Shear and/or torsion 0.9

Bearing on concrete 0.85

20.3—Load tests: General
20.3.1 If strength evaluation is by load tests, a qualified engineer authorized by the Owner and engineer shall control such tests.

20.3.2 A load test shall not be made until that portion of the structure to be subject to load is at least 56 days old. If the Owner, engineer, and all other involved parties agree, it is permitted to make the test at an earlier age.

20.3.3 When only a portion of the structure is to be load tested, the questionable portion shall be load tested in such a manner as to adequately test the suspected source of weakness.

20.3.4 Forty-eight hours prior to application of test load, a load to simulate effect of that portion of the dead loads not already acting shall be applied and shall remain in place until all testing has been completed.

20.3.5 Load tests are not confined to the complete concrete structure; tests may be utilized to determine strength characteristics of specific elements such as anchorages and embedments. The engineer shall specify the appropriate testing parameters.

20.4—Load test procedure
20.4.1—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.

20.4.2—Load intensity

The test load shall be of a magnitude and in the direction of interest necessary to fully evaluate the structural behavior and response of the member or portion thereof. The total test load (including dead load already in place) shall not be less than 0.85 (1.4D + 1.7L).

20.5—Loading criteria

20.5.1 The initial value for all applicable response measurements (such as deflection, rotation, strain, slip, crack widths) shall be obtained not more than one 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.

20.5.2 Test load shall be applied in not less than four approximately equal increments.

20.5.3 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.

20.5.4 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.

20.5.5 Total test load shall be removed immediately after all response measurements defined in 20.5.4 are made.

20.5.6 A set of final response measurements shall be made 24 hours after the test load is removed.

20.6—Acceptance criteria

20.6.1 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.

20.6.2 Measured maximum deflections shall satisfy one of the following conditions:

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

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

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:

$$\Delta_{rmax} \le \frac{\Delta_{fmax}}{5} \qquad (20\text{-}3)$$

where $\mathbf{D}_{fmax}$ is the maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test.

20.6.3 Structural members tested shall not have cracks indicating the imminence of shear failure.

20.6.4 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.

20.6.5 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.

20.6.6 The engineer shall also consider the original design and functional requirements of the structure in question when establishing acceptance criteria for testing.

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. The load testing shall not interfere with the operating status of the nuclear plant, or violate any plant Technical Specifications.

20.7.2 No safety measures shall interfere with load test procedures or affect results.

CHAPTER 21—SPECIAL PROVISIONS FOR SEISMIC DESIGN

21.0—Notation

A_{ch} = cross-sectional area of a structural member measured out-to-out of transverse reinforcement, in.^{2}

A_{cp} = area of concrete section, resisting shear, of an individual pier or horizontal wall segment, in.^{2}

A_{cv} = net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, in.^{2}

A_g = gross area of section, in.^{2}

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. 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:
- beam width plus the joint depth
- 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

b = effective compressive flange width of a structural member, in.

b_w = web width, or diameter of circular section, in.

d = effective depth of section

d_b = bar diameter

f'_c = specified compressive strength of concrete, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi

f_y = specified yield strength of reinforcement, psi

f_{yh} = specified yield strength of transverse reinforcement, psi

h_c = cross-sectional dimension of column core measured center-to-center of confining reinforcement

h_w = height of entire wall (diaphragm) or of the segment of wall (diaphragm) considered

ℓ_d = development length for a straight bar

ℓ_{dh} = development length for a bar with a standard hook as defined in Eq. (21-5)

ℓ_o = minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, in.

ℓ_w = length of entire wall (diaphragm) or of segment of wall (diaphragm) considered in direction of shear force

M_{pr} = probable flexural moment 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 $1.25f_y$ and a strength reduction factor ϕ of 1.0

M_s = portion of slab moment balanced by support moment

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, in.

s_o = maximum spacing of transverse reinforcement, in.

V_c = nominal shear strength provided by concrete

V_e = design shear force determined from 21.3.4.1 or 21.4.5.1

V_n = nominal shear strength

V_u = factored shear force at section

α_c = coefficient defining the relative contribution of concrete strength to wall strength. See Eq. (21-7).

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 distributed shear reinforcement on a plane perpendicular to plane of A_{cv}

ρ_s = ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out)

ρ_v = A_{sv}/A_{cv}; where A_{sv} is the projection on A_{cv} of area of distributed shear reinforcement crossing the plane of A_{cv}

ϕ = strength reduction factor

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 wall and 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 if required by 21.6.6 or 21.6.7.

Collector elements—Elements that serve to transmit the inertial forces within the diaphragms to members of the lateral-force resisting systems.

Crosstie—A continuous reinforcing bar having a seismic hook at one end and a hook not less than 90 degrees with at least a six-diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90-degree hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.

Design load combinations—Combinations of factored loads and forces specified 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-degree hook.

Factored loads and forces—Specified loads and forces modified by the factors in 9.2.

Hoop—A hoop is 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.

Lateral-force resisting system—That portion of the structure composed of members proportioned to resist forces related to earthquake effects.

Seismic hook—A hook on a stirrup, hoop, or crosstie having a bend not less than 135 degrees with a six-diameter (but not less than 3 in.) extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

Shell concrete—Concrete outside the transverse reinforcement confining the concrete.

Specified lateral forces—Lateral forces corresponding to the appropriate distribution of the design base shear force prescribed by the governing code for earthquake-resistant design.

Structural diaphragms—Structural members, such as floor and roof slabs, which 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."

Strut—An element of a structural diaphragm used to provide continuity around an opening in the diaphragm.

Tie elements—Elements that serve to transmit inertial forces and prevent separation of such building components as footings and walls.

21.2—General requirements
21.2.1—Scope
21.2.1.1 The reinforcing bar detailing requirements of this chapter shall be the design practice for nuclear plants within the purview of this code.

21.2.1.2 The provisions of Chapters 1 through 18 shall apply except as modified by the provisions of this chapter.

21.2.1.3 A reinforced concrete structural system not satisfying the requirements of this chapter is 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.

21.2.1.4 All reinforced concrete structural members shall satisfy 21.2 through 21.7 of Chapter 21 in addition to the requirements of Chapters 1 through 17.

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 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 nonstructural members that are not a part of the lateral force resisting system shall also be considered.

21.2.2.3 Structural members below base of structure 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 21.7.

21.2.3—Strength reduction factors
Strength reduction factors shall be as given in 9.3.4.

21.2.4—Concrete in members resisting earthquake-induced forces
Compressive strength, f'_c of the concrete shall be not less than 3000 psi.

21.2.5—Reinforcement in members resisting earthquake-induced forces
Reinforcement resisting earthquake-induced flexural and axial forces in frame members and in wall boundary elements shall comply with ASTM A 706. ASTM A 615 Grades 40 and 60 reinforcement are permitted in these members if (a) the actual yield strength based on mill tests does not exceed the specified yield strength by more than 18,000 psi (retests shall not exceed this value by more than an additional 3000 psi) 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—Welded splices and mechanically connected reinforcement
21.2.6.1 Reinforcement resisting earthquake-induced flexural or axial forces in frame members or in wall boundary elements shall be permitted to be spliced using welded splices or mechanical connections conforming to 12.14.3.3 or 12.14.3.4 provided not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the center-to-center distance between splices of adjacent bars is 24 in. or more measured along the longitudinal axis of the member.

21.2.6.2 Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement required by design shall not be permitted.

21.2.7—Anchoring to concrete
21.2.7.1 Anchors resisting earthquake-induced forces shall conform to the requirements of Appendix B.

21.3—Flexural members of frames
21.3.1—Scope
Requirements of 21.3 apply to frame members (a) resisting earthquake-induced forces and (b) proportioned primarily to resist flexure. These frame members shall also satisfy the following conditions:

21.3.1.1 Factored axial compressive force on the member shall not exceed $(A_g f'_c / 10)$.

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 10 in. 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 $200 b_w d/f_y$, and the reinforcement ratio **r** shall not exceed 0.025. 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 4 in. 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 anal-

ysis indicates flexural yielding caused by inelastic lateral displacements of the frame.

21.3.2.4 Welded splices and mechanical connections shall conform to 21.2.6.1.

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; and

(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 2 in. from the face of a supporting member. Maximum spacing of the hoops shall not exceed (a) [d/4], (b) eight times the diameter of the smallest longitudinal bars, (c) 24 times the diameter of the hoop bars, and (d) 12 in.

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 as specified 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-degree 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-degree 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 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.

21.4—Frame members subjected to bending and axial load

21.4.1—Scope

The requirements of this section apply to frame members (a) resisting earthquake-induced forces and (b) having a factored axial force exceeding ($A_g f'_c$/10). These frame members shall also satisfy the following conditions:

21.4.1.1 The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 12 in.

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 ($A_g f'_c$/10) shall satisfy 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 shall conform to 21.7.

21.4.2.2 The flexural strengths of the columns shall satisfy Eq. (21-1)

$$\sum M_e \ge (6/5)\sum M_g \qquad (21\text{-}1)$$

$\sum M_e$ = sum of moments, at the center of the joint, corresponding to the design 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 center of the joint, corresponding to the design flexural strengths of the girders framing into that joint.

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 21.4.4 over their full height.

21.4.3—Longitudinal reinforcement

21.4.3.1 The reinforcement ratio $\mathbf{r}_g$ shall not be less than 0.01 and shall not exceed 0.06.

21.4.3.2 Welded splices and mechanical connections shall conform to 21.2.6.1. Lap splices shall be permitted only within the center half of the member length and shall be proportioned as tension splices.

21.4.4—Transverse reinforcement

21.4.4.1 Transverse reinforcement as specified below shall be provided unless a larger amount is required by 21.4.5.

(a) The volumetric ratio of spiral or circular hoop reinforcement $\mathbf{r}_s$, shall not be less than that indicated by Eq. (21-2)

$$\rho_s = 0.12 f_c'/f_{yh} \qquad (21\text{-}2)$$

and shall not be less than that required by Eq. (10-6);

(b) The total cross-sectional area of rectangular hoop reinforcement shall not be less than that given by Eq. (21-3) and (21-4);

$$A_{sh} = 0.3(\, sh_c f_c'/f_{yh})[(A_g/A_{ch}) - 1] \qquad (21\text{-}3)$$

$$A_{sh} = 0.09 sh_c f_c'/f_{yh} \qquad (21\text{-}4)$$

(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 to be used. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement; and

(d) If the design strength of member core satisfies the requirement of the specified loading combinations including earthquake effect, Eq. (21-3) and (10-6) need not be satisfied.

21.4.4.2 Transverse reinforcement shall be spaced at distances not exceeding (a) one-quarter of the minimum member dimension and (b) 4 in.

21.4.4.3 Crossties or legs of overlapping hoops shall not be spaced more than 14 in. 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 in connection with inelastic lateral displacements of the frame. The length ℓ_o shall not be less than (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) 18 in.

21.4.4.5 Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with transverse reinforcement as specified 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 $(A_g f_c'/10)$. Transverse reinforcement as specified 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 specified 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 reinforcement in the column at the point of termination. If the column terminates on a footing or mat, transverse reinforcement as specified in 21.4.4.1 through 21.4.4.3 shall extend at least 12 in. into the footing or mat.

21.4.4.6 Where transverse reinforcement, as specified in 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 6 in.

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; and

(b) The factored axial compressive force including earthquake effects is less than $A_g f_c'/20$.

21.5—Joints of 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 $1.25 f_y$.

21.5.1.2 Strength of joint shall be governed by the appropriate strength reduction factors specified 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 20 times the diameter of the largest longitudinal bar for normal weight concrete.

21.5.2—Transverse reinforcement

21.5.2.1 Transverse hoop reinforcement, as specified in 21.4.4 shall be provided within the joint, unless the joint is confined by structural members as specified 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 specified in 21.4.4.2(b) shall be permitted to be increased to 6 in.

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

The nominal shear strength of the joint shall not be taken greater than the forces specified below for normal weight aggregate concrete.

For joints confined on all four faces ....20 $\sqrt{f_c'}A_j$

For joints confined on three faces

or on two opposite faces ....15 $\sqrt{f_c'}A_j$

For others ....12 $\sqrt{f_c'}A_j$

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.

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-degree hook in normal weight-aggregate concrete shall not be less than $8d_b$, 6 in., and the length required by Eq. (21-5).

$$\ell_{dh} = f_y d_b / (65\sqrt{f_c'}) \qquad (21\text{-}5)$$

for bar sizes No. 3 through No. 11.

The 90-degree hook shall be located within the confined core of a column or of a boundary element.

21.5.4.2 For bar sizes No. 3 through No. 11, the development length ℓ_d for a straight bar shall not be less than (a) two-and-a-half (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 12 in. and (b) three-and-a-half (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 12 in.

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 specified in 12.2.4 or 12.3.5.5.

21.6—Structural walls, diaphragms, and trusses

21.6.1—Scope

The requirements of this section apply to structural walls and trusses serving as parts of the earthquake-force-resisting systems as well as to diaphragms, struts, ties, chords and collector elements which transmit forces induced by earthquake. For shearwalls with h_w/ℓ_w of less than 2.0, provisions of 21.6.6 can be waived.

21.6.2—Reinforcement

21.6.2.1 The reinforcement ratio ρ_v for structural walls shall not be less than 0.0025 along the longitudinal and transverse axes. If the design shear force does not exceed $A_{cv}\sqrt{f_c'}$, the minimum reinforcement for structural walls shall be in conformance with 14.3. The minimum reinforcement ratio for structural diaphragms shall be in conformance with 7.12. Reinforcement spacing each way in structural walls and diaphragms shall not exceed 18 in. Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane.

21.6.2.2 At least two curtains of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $2A_{cv}\sqrt{f_c'}$.

21.6.2.3 Structural-truss elements, struts, ties, and collector elements with compressive stresses exceeding $0.2f_c'$ shall have special transverse reinforcement, as specified in 21.4.4, over the total length of the element. The special transverse reinforcement is allowed to be discontinued at a section where the calculated compressive stress is less than 0.15 f_c'. Stresses shall be calculated for the factored forces using a linearly elastic model and gross-section properties of the elements considered.

21.6.2.4 All continuous reinforcement in structural walls, diaphragms, trusses, struts, ties, chords, and collector elements shall be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in 21.5.4.

21.6.3—Design forces

The design shear force V_u shall be obtained from the lateral load analysis in accordance with the factored loads and combinations specified in 9.2.

21.6.4—Diaphragms

21.6.4.1—Minimum thickness of diaphragms

Concrete diaphragms and composite topping slabs serving as diaphragms used to transmit earthquake forces shall not be less than 2 in. thick.

21.6.4.2—Cast-in-place composite topping slab diaphragms

A composite topping slab cast-in-place on a precast floor or roof system shall be permitted to be used as a 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 resisting elements. The surface of the previously hardened concrete on which the topping slab is placed shall be clean, free of laitance, and shall be intentionally roughened.

21.6.5—Shear strength

21.6.5.1 Nominal shear strength of structural walls and diaphragms shall be determined using either 21.6.5.2 or 21.6.5.3.

21.6.5.2 Nominal shear strength V_n of structural walls and diaphragms shall be assumed not to exceed the shear force calculated from

$$V_n = A_{cv}(2\sqrt{f_c'} + \rho_n f_y) \qquad (21\text{-}6)$$

21.6.5.3 For walls (diaphragms) and wall (diaphragm) segments having a ratio of (h_w/ℓ_w) less than 2.0, nominal shear strength of wall (diaphragm) shall be determined from Eq. (21-7)

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

where the coefficient $\mathbf{a}_c$ varies linearly from 3.0 for $(h_w/\ell_w) =$ 1.5 to 2.0 for $(h_w/\ell_w) = 2.0$.

21.6.5.4 In 21.6.5.3, value of ratio (h_w/ℓ_w) used for determining V_n for segments of a wall or diaphragm shall be the larger of the ratios for the entire wall (diaphragm) and the segment of wall (diaphragm) considered.

21.6.5.5 Walls (diaphragms) shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall (diaphragm). If the ratio (h_w/ℓ_w) does not exceed 2.0, reinforcement ratio $\mathbf{r}_v$ shall not be less than reinforcement ratio $\mathbf{r}_n$.

21.6.5.6 Nominal shear strength of all wall piers sharing a common lateral force shall not be assumed to exceed $8A_{cv}\sqrt{f_c'}$, where A_{cv} is the total cross-sectional area, and the nominal shear strength of any one of the individual wall piers shall not be assumed to exceed $10A_{cp}\sqrt{f_c'}$, where A_{cp} represents the cross-sectional area of the pier considered.

21.6.5.7 Nominal shear strength of horizontal wall segments shall not be assumed to exceed $10A_{cp}\sqrt{f_c'}$, where A_{cp} represents the cross-sectional area of a horizontal wall segment.

21.6.6—Boundary elements for structural walls

21.6.6.1 Boundary elements shall be provided at boundaries and edges around openings of structural walls when the maximum extreme fiber stress, corresponding to factored forces including earthquake effect, exceeds $0.2f_c'$ unless the entire wall is reinforced to satisfy 21.4.4.1 through 21.4.4.3. The 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.

21.6.6.2 Boundary elements, where required, shall have transverse reinforcement as specified in 21.4.4.1 through 21.4.4.3.

21.6.6.3 Boundary elements shall be proportioned to resist all factored gravity loads on the wall, including tributary loads and self-weight, as well as the vertical force required to resist overturning moment calculated from factored forces related to earthquake effect.

21.6.6.4 Transverse reinforcement in walls with boundary elements shall be anchored within the confined core of the boundary element to develop the specified yield strength f_y of the transverse reinforcement.

21.6.6.5 Except when V_u in the plane of the wall is less than $A_{cv}\sqrt{f_c'}$, transverse 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 transverse reinforcement.

21.6.6.6 Welded splices and mechanical connections of longitudinal reinforcement of boundary elements shall conform to 21.2.6.1.

21.6.7—Boundary elements of structural diaphragms

21.6.7.1 Boundary elements of structural diaphragms shall be proportioned to resist the sum of the factored axial force 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.

21.6.7.2 Splices of tensile reinforcement in the boundaries and collector elements of all diaphragms shall develop the yield strength of the reinforcement. Welded splices and mechanical connections shall conform to 21.2.6.1.

21.6.8—Construction joints

All construction joints in walls and diaphragms shall conform to 6.4 and contact surfaces shall be roughened as specified in 11.7.9.

21.6.9—Discontinuous walls

Columns supporting discontinuous walls shall be reinforced in accordance with 21.4.4.5.

21.7—Frame members not proportioned to resist forces induced by earthquake motions

21.7.1 Frame members assumed not to contribute to lateral resistance shall be detailed according to 21.7.2 or 21.7.3 depending on the magnitude of moments induced in those members when subjected to twice the lateral displacements under the factored lateral forces.

21.7.2 When the induced moments and shears under lateral displacements of 21.7.1 combined with the factored gravity moment and shears do not exceed the design moment and shear strength of the frame member, the following conditions shall be satisfied.

21.7.2.1 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.

21.7.2.2 Members with factored gravity axial forces exceeding $(A_g f_c'/10)$ shall satisfy 21.4.3, 21.4.4.1(3), 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 6 in., whichever is smaller.

21.7.2.3 Members with factored gravity axial forces exceeding $0.35P_o$ shall satisfy 21.7.2.2 and the amount of transverse reinforcement provided shall be one-half of that required by 21.4.4.1 not to exceed a spacing s_o for the full height of the column.

21.7.3 If the induced moment or shear under lateral displacements of 21.7.1 exceed the design moment or shear strength of the frame member, or if induced moments are not calculated, the following conditions shall apply:

21.7.3.1 Materials shall satisfy 21.2.4 and 21.2.5. Splices of reinforcement shall satisfy 21.2.6.

21.7.3.2 Members with factored gravity axial forces not exceeding $(A_g f_c'/10)$ shall satisfy 21.3.2.1 and 21.3.4. Stirrups shall be spaced at not more than $d/2$ throughout the length of the member.

21.7.3.3 Members with factored gravity axial forces exceeding $(A_g f_c'/10)$ shall satisfy 21.4.4, 21.4.5, and 21.5.2.1.

APPENDIX A—Thermal Considerations

A.1—Scope

A.1.1 Nuclear safety related reinforced concrete structures shall conform to the minimum provisions of this Code and to the special provisions of this Appendix for structural members subjected to time-dependent and position-dependent temperature variations.

A.1.2 The provisions of this Appendix apply to concrete structures which are subjected to normal operating conditions as well as thermal accident conditions and which have restraint such that thermal strains would result in thermal stresses.

A.1.3 The design provisions of this Appendix are based on the strength design method. The assumptions, principles, and requirements specified in 10.1 and 10.2 are applicable for both normal operating and accident conditions.

A.1.4 This Appendix does not address temperature requirements during curing, nor does it address temperature and shrinkage reinforcement.

A.2—Definitions

Base temperature—The temperature at which a concrete member is cured.

Temperature distribution—The variation of the total temperature across a section at a point in time.

Mean temperature distribution—A uniform distribution of temperature across a section evaluated to be an average of the temperature distribution.

Gradient temperature distribution—The temperature distribution minus the mean temperature distribution across a section at a point in time.

Thermal strain—Strain produced by thermal expansion or contraction due to a thermal gradient and the difference between the base and mean temperature.

Thermal stress—Stress produced by restraint of thermal strain.

A.3—General design requirements

A.3.1 The effects of the gradient temperature distribution and the difference between mean temperature distribution and base temperature during normal operation or accident conditions shall be considered.

A.3.2 Time-dependent variations of temperature distributions shall be considered in evaluating thermal strains for both normal operating conditions and accident conditions.

A.3.3 Thermal stress shall be evaluated considering the stiffness of the member and the rigidity of the section and the degree of restraint of the structure. The evaluation may be based on cracked section properties, provided the following conditions are met:

(a) The tensile stress for any section exceeds the tensile stress at which the section is considered cracked;

(b) Redistribution of internal forces and strains due to cracking are included;

(c) All concurrent loads, as specified in 9.2, are considered; and

(d) The coefficient of thermal expansion may be taken as 5.5×10^{-6} per degree F unless other values are substantiated by "tests."

A.3.4 When thermal stress is combined with the stress due to other loads to determine a design stress, the magnitude of the design stress must not be less than the magnitude of the stress due to other loadings alone unless the following are considered:

(a) The effect of cracking in the tensile zone of flexural members on reduction of the flexural rigidity and on the redistribution of stress;

(b) The reduction of long term stresses due to creep; and

(c) Stress combinations that reduce the magnitude of the stress due to other loads utilizing actual temperatures and temperature distributions which act concurrently with the other loads.

A.4—Concrete temperatures

A.4.1 The following temperature limitations are for normal operation or any other long term period. The temperatures shall not exceed 150 F except for local areas, such as around penetrations, which are allowed to have increased temperatures not to exceed 200 F.

A.4.2 The following temperature limitations are for accident or any other short term period. The temperatures shall not exceed 350 F for the surface. However, local areas are allowed to reach 650 F from steam or water jets in the event of a pipe failure.

A.4.3 Higher temperatures than those given in A.4.1 and A.4.2 above may be allowed for concrete if tests are provided to evaluate the reduction in strength and this reduction is applied to design allowables. Also, evidence shall be provided which verifies that the increased temperatures do not cause deterioration of the concrete either with or without load.

APPENDIX B—Anchoring to Concrete

B.0—Notation

A_{brg} = bearing area of the head of stud or anchor bolt, in.^{2}

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 B.5.2.1, in.^{2} [See Fig. RB.5.1(a)]

A_N = projected concrete failure area of an anchor or group of anchors, for calculation of strength in tension, as defined in B.5.2.1, in.^{2} A_N shall not be taken greater than nA_{No} . [See Fig. RB.5.1(b)]

A_{se} = effective cross-sectional area of anchor, in.^{2}

A_{sl} = effective cross-sectional area of expansion or under-cut anchor sleeve, if sleeve is within shear plane, in.^{2}

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 B.6.2.1, in.^{2} [(See Fig. RB.6.2(a)]

A_V = projected concrete failure area of an anchor or group of anchors, for calculation of strength in shear, as defined in B.6.2.1, in.^{2} A_V shall not be taken greater than nA_{Vo} . [See Fig. RB.6.2(b)]

C = the compressive resultant force between the embedment and the concrete resulting from factored moment and factored axial load applied to the embedment, lb

c = distance from center of an anchor shaft to the edge of concrete, in.

c_1 = distance from the center of an anchor shaft to the edge of concrete in one direction, in.; where shear force is applied to anchor, c_1 is in the direction of the shear force. [See Fig. RB.6.2(a)]

c_2 = distance from center of an anchor shaft to the edge of concrete in the direction orthogonal to c_1, in.

c_{max} = the largest edge distance, in.

c_{min} = the smallest edge distance, in.

d_o = outside diameter of anchor or shaft diameter of headed stud, or headed bolt, in.

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, in.; e_N' is always positive [See Fig. RB.5.2(b and c)]

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, in.

f_c' = specified compressive strength of concrete, psi

f_{ct} = specified tensile strength of concrete, psi

f_r = modulus of rupture of concrete, psi (See 9.5.2.3)

f_t = calculated tensile stress in a region of a member, psi

f_y = specified yield strength of anchor steel, psi

f_{ut} = specified tensile strength of anchor steel, psi

f_{utsl} = specified tensile strength of anchor sleeve, psi

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

h_{ef} = effective anchor embedment depth, in. (See B.8.5

and Fig. RB.1)

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 $8d_o$, in.

 = 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, as defined in B.5.2.2, lb

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

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

N_n = nominal strength in tension, lb

N_p = pullout strength in tension of a single anchor in cracked concrete, as defined in B.5.3.4, lb

N_{pn} = nominal pullout strength in tension of a single anchor, as defined in B.5.3.1, lb

N_s = nominal strength in tension of a single anchor or group of anchors as governed by the steel strength, as defined in B.5.1.1 or B.5.1.2, lb

N_{sb} = side-face blowout strength of a single anchor, lb

N_{sbg} = side-face blowout strength of a group of anchors, lb

N_u = factored tensile load, lb

P_a = factored external axial load on the embedment, lb

s = anchor center-to-center spacing, in.

s_o = spacing of the outer anchors along the edge in a group, in.

t = thickness of washer or plate, in.

V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete, as defined in B.6.2.2 or B.6.2.3, lb

V_{cb} = nominal concrete breakout strength in shear of a single anchor, as defined in B.6.2.1, lb

V_{cbg} = nominal concrete breakout strength in shear of a group of anchors, as defined in B.6.2.1, lb

V_{cp} = nominal concrete pryout strength, as defined in B.6.3, lb

V_n = nominal shear strength, lb

V_s = nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, as defined in B.6.1.1 or B.6.1.2, lb

V_u = factored shear load, lb

ϕ = strength reduction factor (see B.4.4)

Ψ_1 = modification factor, for strength in tension, to account for anchor groups loaded eccentrically, as defined in B.5.2.4

Ψ_2 = modification factor, for strength in tension, to account for edge distances smaller than $1.5h_{ef}$, as defined in B.5.2.5

Y_3 = modification factor, for strength in tension, to account for cracking, as defined in B.5.2.6 and B.5.2.7

Y_4 = modification factor, for pullout strength, to account for cracking, as defined in B.5.3.1 and B.5.3.5

Y_5 = modification factor, for strength in shear, to account for anchor groups loaded eccentrically, as defined in B.6.2.5

Y_6 = modification factor, for strength in shear, to account for edge distances smaller than $1.5c_1$, as defined in B.6.2.6

Y_7 = modification factor, for strength in shear, to account for cracking, as defined in B.6.2.7

B.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, headed studs, expansion anchors, undercut anchors, or specialty inserts.

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 load from the anchor.

Brittle steel element—An element with a tensile test elongation of less than 14%, or reduction in area of less than 30%, or both.

Cast-in anchor—A headed bolt or headed stud 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 embedment—An embedment designed for a ductile steel failure in accordance with B.3.6.1.

Ductile steel element—An element with a tensile test elongation of at least 14% and reduction in area of at least 30%. A steel meeting 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 bolts and headed studs, the effective

embedment depth is measured from the bearing contact surface of the head. (See Fig. RB.1)

Embedment—A steel component embedded in the concrete to transmit applied loads to or from the concrete structure. The embedment may be fabricated of plates, shapes, anchors, reinforcing bars, shear connectors, specialty inserts, or any combination thereof.

Expansion anchor—A post-installed anchor inserted into hardened concrete that transfers loads into 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.

5% fractile—statistical term meaning 90% confidence that there is 95% probability of the actual strength exceeding the nominal strength.

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.

Supplementary reinforcement—Reinforcement proportioned to tie a potential concrete failure prism to the structural member.

Undercut anchor—A post-installed anchor that derives its tensile strength by 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.

B.2—Scope

B.2.1 This Appendix provides design requirements for structural embedments in concrete used to transmit structural loads from attachments into concrete members or from one connected concrete member to another by means of tension, shear, bearing, or a combination thereof. Safety levels specified are intended for in-service conditions, rather than for short term handling and construction conditions.

B.2.2 This Appendix applies to both cast-in anchors and post-installed anchors. 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.

B.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 1.4 N_p [where N_p is given by Eq. (B-10)] are included. Post-installed anchors are included provided that B.3.3 is satisfied.

B.2.4 Load applications that are predominantly high-cycle fatigue are not covered by this Appendix.

B.2.5 In addition to meeting the requirements of this appendix, consideration shall be given to the effect of the forces applied to the embedment on the behavior of the overall structure.

B.2.6 The jurisdiction of this code covers steel material below the surface of the concrete and the anchors extending above the surface of concrete. The requirements for the attachment to the embedment shall be in accordance with applicable codes and are beyond the scope of this Appendix.

B.3—General requirements

B.3.1 The embedment and surrounding concrete or grout 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. Assumptions used in distributing loads within the embedment shall be consistent with those used in the design of the attachment.

B.3.2 The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in 9.2.

B.3.3 Post-installed structural anchors shall be tested before use to verify that they are capable of sustaining their design strength in cracked concrete under seismic loads. These verification tests shall be conducted by an independent testing agency and shall be certified by a professional engineer with full description and details of the testing programs, procedures, results, and conclusions.

B.3.4 All provisions for anchor axial tension and shear strength apply to normalweight concrete only.

B.3.5 The values of f_c' used for calculations in this Appendix shall not exceed 10,000 psi for cast-in anchors and 8000 psi for post-installed anchors.

B.3.6—Embedment design

B.3.6.1 Embedment design shall be controlled by the strength of embedment steel. The design strength shall be determined using the strength reduction factor specified in B.4.4(a). It shall be permitted to assume that design is controlled by the strength of embedment steel where the design concrete breakout tensile strength of the embedment, the design side blowout strength of the embedment, and the design pullout strength of the anchors exceed the specified ultimate tensile strength of the embedment steel and when the design concrete breakout shear strength exceeds 65% of the specified ultimate tensile strength of the embedment steel. The de-

sign concrete tensile strength, the design side blow out strength, the design pullout strength, and the design concrete breakout shear strength shall be taken as 0.85 times the nominal strengths.

B.3.6.2 As an alternate to B.3.6.1, the attachment shall be designed to yield at a load level corresponding to anchor forces not greater than 75% of the anchor design strength specified in B.4.1.2. The anchor design strength shall be determined using the strength reduction factors specified in B.4.4 (b) or (c).

B.3.6.3 It shall be permitted to design anchors as nonductile anchors. The design strength of such anchors shall be taken as 0.60 ϕN_n and 0.60 ϕV_n, where ϕ is given in B.4.4 and N_n and V_n are determined in accordance with B.4.1.

B.3.7 Material and testing requirements for embedment steel shall be specified by the engineer so that the embedment design is compatible with the intended function of the attachment.

B.3.8 Embedment materials for ductile anchors other than reinforcing bars shall be ductile steel elements.

B.3.9 Ductile anchors that incorporate a reduced section in the tension or shear load path shall satisfy one of the following conditions:

(a) The ultimate tensile strength of the reduced section shall be greater than the yield strength of the unreduced section; or

(b) For bolts, the length of thread in the load path shall be at least two anchor diameters.

B.3.10 The design strength of embedment materials may be increased in accordance with Appendix C for embedments subject to impactive and impulsive loads.

B.3.11 Plastic deformation of the embedment is permitted for impactive and impulsive loading provided the strength of the embedment is controlled by the strength of the embedment steel as specified in B.3.6.

B.4—General requirements for strength of structural anchors

B.4.1 Strength design of structural anchors shall be based either on computation using design models that satisfy the requirements of B.4.2 or on test evaluation using the 5% fractile of test results for the following:

(a) steel strength of anchor in tension (B.5.1);

(b) steel strength of anchor in shear (B.6.1);

(c) concrete breakout strength of anchor in tension (B.5.2);

(d) concrete breakout strength of anchor in shear (B.6.2);

(d) pullout strength of anchor in tension (B.5.3);

(e) concrete side-face blowout strength of anchor in tension (B.5.4); and

(f) concrete pryout strength of anchor in shear (B.6.3).

In addition, anchors shall satisfy the required edge distances, spacings, and thicknesses to preclude splitting failure as prescribed in B.8.

B.4.1.1 For the design of anchors

$$\phi N_n \geq N_u \tag{B-1}$$

$$\phi V_n \geq V_u \tag{B-2}$$

B.4.1.2 In Eq. (B-1) and (B-2), $\mathbf{f}N_n$ and $\mathbf{f}V_n$ are the lowest design strengths determined from all appropriate failure modes. $\mathbf{f}N_n$ is the lowest design strength in tension of an anchor or group of anchors as determined from consideration of $\mathbf{f}N_s$, $\mathbf{f}N_{pn}$, either $\mathbf{f}N_{sb}$ or $\mathbf{f}N_{sbg}$, and either $\mathbf{f}N_{cb}$ or $\mathbf{f}N_{cbg}$. $\mathbf{f}V_n$ is the lowest design strength in shear of an anchor or a group of anchors as determined from consideration of $\mathbf{f}V_s$, either $\mathbf{f}V_{cb}$ or $\mathbf{f}V_{cbg}$, and $\mathbf{f}V_{cp}$.

B.4.1.3 When both N_u and V_u are present, interaction effects shall be considered in accordance with B.4.3.

B.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% 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 accounted for. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

B.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 B.4.2.

B.4.2.2 For anchors with diameters not exceeding 2 in., and tensile embedments not exceeding 25 in. in depth, the concrete breakout strength requirements shall be considered satisfied by the design procedure of B.5.2 and B.6.2.

B.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 B.7.

B.4.4 Strength reduction factor $\mathbf{f}$ for anchoring to concrete shall be as follows when the load combinations of 9.2 are used:
(a) Anchor governed by strength of a ductile steel element
   i) Tension loads. 0.80
   ii) Shear loads 0.75
(b) Anchor governed by strength of a brittle steel
   element
   i) Tension loads. 0.70
   ii) Shear loads 0.65
(c) Anchor governed by concrete breakout, side-face blowout, pullout, or pryout strength 0.75

B.4.5—Bearing strength

B.4.5.1 A combination of bearing and shear friction mechanisms shall not be used to develop the nominal shear strength defined in accordance with 9.2. If the requirements of 9.2.3 are satisfied, however, it is permitted to use the available confining force afforded by the tension anchors in combination with acting (or applied) loads used in determining the shear strength of embedments with shear lugs.

B.4.5.2 The design bearing strength used for concrete or grout placed against shear lugs shall not exceed $1.3\mathbf{f}f_c'$ using

a strength reduction factor $\mathbf{f}$ of 0.70. For grouted installations, the value of f_c' shall be the compressive strength of the grout or the concrete, whichever is less.

B.5—Design requirements for tensile loading

B.5.1—Steel strength of anchor in tension

B.5.1.1 The nominal strength of an anchor in tension as governed by the steel N_s shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

B.5.1.2 The nominal strength of an anchor or group of anchors in tension N_s shall not exceed

$$N_s = nA_{se}f_{ut} \tag{B-3}$$

where f_{ut} shall not be taken greater than $1.9f_y$ or 125,000 psi.

B.5.2—Concrete breakout strength of anchor in tension

B.5.2.1 The nominal concrete breakout strength N_{cb} of an anchor or group of anchors in tension shall not exceed:
(a) for a single anchor

$$N_{cb} = \frac{A_N}{A_{No}}\psi_2\psi_3 N_b \tag{B-4a}$$

(b) for a group of anchors

$$N_{cbg} = \frac{A_N}{A_{No}}\psi_1\psi_2\psi_3 N_b \tag{B-4b}$$

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

$$A_{No} = 9h_{ef}^2 \tag{B-5}$$

B.5.2.2 The basic concrete breakout strength N_b of a single anchor in tension in cracked concrete shall not exceed

$$N_b = k\sqrt{f_c'}\, h_{ef}^{1.5} \tag{B-6a}$$

where k = 24 for cast-in anchors, k = 17 for post-installed anchors.

Alternatively, for cast-in headed studs and headed bolts with 11 in. $\leq h_{ef} \leq$ 25 in., the basic concrete breakout strength of a single anchor in tension in cracked concrete shall not exceed

$$N_b = 16\sqrt{f_c'}\, h_{ef}^{5/3} \tag{B-6b}$$

B.5.2.3 For the special case of anchors in an application with three or four edges and the largest edge distance $c_{max} \leq 1.5h_{ef}$, the embedment depth h_{ef} used in Eq. (B-5), (B-6), (B-7), and (B-8) shall be limited to $c_{max}/1.5$.

B.5.2.4 The modification factor for eccentrically loaded anchor groups is

$$\psi_1 = \frac{1}{\left(1 + \dfrac{2\,e'_N}{3h_{ef}}\right)} \leq 1 \qquad \text{(B-7)}$$

Eq. (B-7) is valid for $e'_N \leq s/2$.

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. (B-7).

In the case where eccentric loading exists about two axes, the modification factor $\mathbf{\psi_1}$ shall be computed for each axis individually and the product of these factors used as $\mathbf{\psi_1}$ in Eq. (B-4b).

$\mathbf{\psi_1}$ is equal to 1.0 for a ductile embedment analyzed using only linear (elastic) analysis techniques.

B.5.2.5 The modification factor for edge effects is

$$\psi_2 = 1 \text{ if } c_{min} \geq 1.5h_{ef} \qquad \text{(B-8a)}$$

$$\psi_2 = 0.7 + 0.3\frac{c_{min}}{1.5h_{ef}} \text{ if } c_{min} < 1.5h_{ef} \qquad \text{(B-8b)}$$

B.5.2.6 When an anchor is located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ under the load combinations specified in 9.2 with load factors taken as unity, the following modification factor shall be permitted

$$\psi_3 = 1.25 \text{ for cast-in anchors}$$

$$\psi_4 = 1.4 \text{ for post-installed anchors.}$$

When analysis indicates cracking under the load combinations specified in 9.2 with load factors taken as unity, $\mathbf{\psi_3}$ shall be taken as 1.0 for both cast-in anchors and post-installed anchors. 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.

B.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 anchor, where t is the thickness of the washer or plate.

B.5.2.8 For post-installed anchors, it shall be permitted to use a coefficient k in Eq. (B-6a) or (B-6b) based on the 5% fractile of results from product-specific tests. For such cases,

the modification factor $\mathbf{\psi_3}$ shall be based on a direct comparison between the average ultimate failure loads and the characteristic loads based on the 5% fractile of product-specific testing in cracked concrete and otherwise identical product-specific testing in uncracked concrete.

B.5.3—Pullout strength of anchor in tension

B.5.3.1 The nominal pullout strength N_p of an anchor in tension shall not exceed

$$N_{pn} = \psi_4 N_p \qquad \text{(B-9)}$$

B.5.3.2 For post-installed expansion and undercut anchors, it is not permissible to calculate the pullout strength in tension. Values of N_p shall be based on the 5% fractile of results of tests performed and evaluated according to B.3.3.

B.5.3.3 For single cast-in headed studs and headed bolts, it shall be permitted to evaluate the pullout strength in tension using B.5.3.4.

B.5.3.4 The pullout strength in tension of a single headed stud or headed bolt N_p for use in Eq. (B-9), shall not exceed

$$N_p = A_{brg}8f'_c \qquad \text{(B-10)}$$

B.5.3.5 For an anchor located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ under the load combinations specified in 9.2 with load factors taken as unity, the following modification factor shall be permitted

$$\psi_4 = 1.4$$

Otherwise, $\mathbf{\psi_4}$ shall be taken as 1.0.

B.5.4—Concrete side-face blowout strength of a headed anchor in tension

B.5.4.1 For a single headed anchor with deep embedment close to an edge, the nominal side-face blowout strength N_{sb} shall not exceed

$$N_{sb} = 160\, c\sqrt{A_{brg}}\sqrt{f'_c} \qquad \text{(B-11)}$$

If the single 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 $1 \leq c_2/c \leq 3$.

B.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 outer anchors along the edge in the group for a side-face blowout failure N_{sbg} shall not exceed

$$N_{sbg} = \left(1 + \frac{s_o}{6c}\right)N_{sb} \qquad \text{(B-12)}$$

where s_o = spacing of the outer anchors along the edge in the group and N_{sb} is obtained from Eq. (B-11) without modification for a perpendicular edge distance. The nominal strength of the group of fasteners shall be taken as the nominal strength of the outer anchors along the edge multiplied by the number of rows parallel to the edge.

B.6—Design requirements for shear loading
B.6.1—Steel strength of anchor in shear
B.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.

B.6.1.2 The nominal strength V_s of an anchor or group of anchors in shear shall not exceed

(a) for cast-in headed stud anchors

$$V_s = nA_{se}f_{ut} \qquad (B-13)$$

where f_{ut} shall not be taken greater than $1.9f_y$ or 125,000 psi.

(b) for cast-in headed bolt anchors

$$V_s = n0.6A_{se}f_{ut} \qquad (B-14)$$

where f_{ut} shall not be taken greater than $1.9f_y$ or 125,000 psi.

(c) for post-installed anchors

$$V_s = n(0.6A_{se}f_{ut} + 0.4A_{s\ell}f_{uts\ell}) \qquad (B-15)$$

where f_{ut} shall not be taken greater than $1.9f_y$ or 125,000 psi.

When the anchor is installed so that the critical failure plane does not pass through the sleeve, the area of the sleeve in Eq. (B-15) shall be taken as zero.

B.6.1.3 Where anchors are used with built-up grout pads, the nominal strengths of B.6.1.2 shall be multiplied by a 0.80 factor.

B.6.1.4 Friction between the baseplate and concrete may be considered to contribute to the nominal shear strength of the connection. The nominal shear strength resulting from friction between the baseplate and concrete (that is, without any contribution from anchors) may be taken as $0.40C$.

B.6.2—Concrete breakout strength of anchor in shear
B.6.2.1 The nominal concrete breakout strength V_{cb} in shear of an anchor or group of anchors shall not exceed:

(a) for shear force perpendicular to the edge on a single anchor

$$V_{cb} = \frac{Av}{Avo}\psi_6\psi_7V_b \qquad (B-16a)$$

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

$$V_{cbg} = \frac{Av}{Avo}\psi_5\psi_6\psi_7V_b \qquad (B-16b)$$

(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. (B-16a or b) 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

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 $1.5c_1$

$$A_{vo} = 4.5c_1^2 \qquad (B-17)$$

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 which is selected as critical, and all of the shear shall be assumed to be carried by this critical anchor row alone.

B.6.2.2 The basic concrete breakout strength V_b in shear of a single anchor in cracked concrete shall not exceed

$$V_b = 7\left(\frac{\ell}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}\,c_1^{1.5} \qquad (B-18a)$$

B.6.2.3 For cast-in headed studs, or headed bolts, that are rigidly welded to steel attachments having a minimum thickness equal to the greater of 3/8 in. 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

$$V_b = 8\left(\frac{\ell}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}\,c_1^{1.5} \qquad (B-18b)$$

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 2.5 in.; and

(c) supplementary reinforcement is provided at the corners if $c_2 \pounds 1.5h_{ef}$.

B.6.2.4 For the special case of anchors influenced by three or more edges, the edge distance c_1 used in Eq. (B-17), (B-18), (B-19), and (B-20) shall be limited to $h/1.5$.

B.6.2.5 The modification factor for eccentrically loaded anchor groups is

$$\psi_5 = \frac{1}{1 + \dfrac{2e_v'}{3c_1}} \le 1 \qquad \text{(B-19)}$$

Eq. (B-19) is valid for $e_v' \le s/2$.

B.6.2.6 The modification factor for edge effects is

$$\psi_6 = 1 \text{ if } c_2 \ge 1.5c_1 \qquad \text{(B-20a)}$$

$$\psi_6 = 0.7 + 0.3\frac{c_2}{1.5c_1} \text{ if } c_2 < 1.5c_1 \qquad \text{(B-20b)}$$

B.6.2.7 For anchors located in a region of a concrete member where analysis indicates no cracking ($f_t < f_r$) under the load combinations specified in 9.2 with load factors taken as unity, 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 under the load combinations specified in 9.2 with load factors taken as unity, the following modification factors shall be permitted:

$\mathbf{y}_7 = 1.0$ for anchors in cracked concrete with no supplementary reinforcement or supplementary reinforcement smaller than a No. 4 bar;

$\mathbf{y}_7 = 1.2$ for anchors in cracked concrete with supplementary reinforcement of a No. 4 bar or greater between the anchor and the edge; and

$\mathbf{y}_7 = 1.4$ for anchors in cracked concrete with supplementary reinforcement of a No. 4 bar or greater between the anchor and the edge and with the supplementary reinforcement enclosed within stirrups spaced at not more than 4 in.

To be considered as supplementary reinforcement, the reinforcement shall be designed to intersect the concrete breakout failure surface defined in B.5.2.1.

B.6.3—Concrete pryout strength of anchor in shear

B.6.3.1 The nominal pryout strength V_{cp} shall not exceed

$$V_{cp} = k_{cp}N_{cb} \qquad \text{(B-21)}$$

where k_{cp}=1.0 for $h_{ef} < 2.5$ in.; k_{cp}= 2.0 for $h_{ef} \ge 2.5$ in.; and N_{cb} shall be determined from Eq. (B-4a).

B.7—Interaction of tensile and shear forces

Unless determined in accordance with B.4.3, anchors or groups of anchors that are subjected to both shear and axial loads shall be designed to satisfy the requirements of B.7.1 through B.7.3. The value of $\mathbf{f}N_n$ shall be as defined in B.4.1.2.

B.7.1 If $V_u \le 0.2\mathbf{f}V_n$, then full strength in tension shall be permitted: $\mathbf{f}N_n \ge N_u$.

B.7.2 If $N_u \le 0.2\mathbf{f}N_n$, then full strength in shear shall be permitted: $\mathbf{f}V_n \ge V_u$.

B.7.3 If $V_u > 0.2\mathbf{f}V_n$ and $N_u > 0.2\mathbf{f}N_n$, then

$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \le 1.2 \qquad \text{(B-22)}$$

B.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 B.8.1 through B.8.6, unless supplementary reinforcement is provided to control splitting.

B.8.1 Minimum center-to-center spacing of cast-in anchors shall be $4d_o$ for untorqued anchors and $6d_o$ for torqued anchors.

B.8.2 Minimum center-to-center spacing of post-installed anchors shall be based on tests performed according to B.3.3.

B.8.3 Minimum edge distances for cast-in anchors that will not be torqued shall satisfy the minimum cover requirements for reinforcement in 7.7. Minimum edge distances for cast-in anchors that will be torqued shall be based on the greater of the minimum cover requirements for reinforcement in 7.7 or $6d_o$.

B.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 performed according to B.3.3, and shall not be less than two times the maximum aggregate size.

B.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 4 in.

B.8.6 Project drawings and project specifications shall specify use of anchors with a minimum edge distance as assumed in design.

B.9—Installation of anchors

B.9.1 Anchors shall be installed in accordance with the project drawings and project specifications and the requirements stipulated by the anchor manufacturer.

B.9.2 The engineer shall establish an inspection program to verify proper installation of the anchors.

B.9.3 The engineer shall establish a welding procedure to avoid excessive thermal deformation of an embedment that, if welded to the attachment, could cause spalling or cracking of the concrete or pullout of the anchor.

B.10—Structural plates, shapes, and specialty inserts

B.10.1 The design strength of embedded structural shapes, fabricated shapes, and shear lugs shall be determined based on fully yielded conditions, and using a $\mathbf{f}$ factor of 0.9 for tension, compression, and bending (and combinations thereof), and 0.55 for shear.

B.10.2 For structural shapes and fabricated steel sections, the web shall be designed for the shear and the flang-

es shall be designed for the tension, compression, and bending.

B.10.3 The nominal strength of specialty inserts shall be based on the 5% fractile of results of tests performed and evaluated according to B.3. Embedment design shall be according to B.3 with strength reduction factors according to B.4.4.

B.11—Shear capacity of embedded plates and shear lugs

B.11.1—General

The shear strength of grouted or cast-in embedments with shear lugs shall include consideration of the bearing strength of the concrete or grout placed against the shear lugs, the direct shear strength of the concrete or grout placed between shear lugs and the confinement afforded by the tension anchors in combination with external loads acting across potential shear planes. Shear loads toward free edges and displacement compatibility between shear lugs shall also be considered. When multiple shear lugs are used to establish the design shear strength in a given direction, the magnitude of the allotted shear to each lug shall be in direct proportion to the total shear, the number of lugs, and the shear stiffness of each lug.

B.11.2—Shear toward free edge

For shear lugs bearing toward a free edge, unless reinforcement is provided to develop the required strength, the design shear strength for each lug shall be determined based on a uniform tensile stress of $4\phi\sqrt{f_c'}$ acting on an effective stress area defined by projecting a 45 degree plane from the bearing edges of the shear lug or base plate to the free surface. The bearing area of the shear lug or plate edge shall be excluded from the projected area. The ϕ factor shall be taken as 0.85.

B.11.3—Shear strength of embedments with embedded base plates

For embedments having a base plate whose contact surface is below the surface of the concrete, shear strength shall be calcu-lated using the shear-friction provisions of 11.7 (as modified by this section), using the following shear-friction coefficients:

Base plate without shear lugs 0.9

Base plate with shear lugs that is designed to

remain elastic 1.4

The tension anchor steel area required to resist external loads shall be added to the tension anchor steel area required due to shear friction.

B.12—Grouted embedments

B.12.1 Grouted embedments shall meet the applicable requirements of this Appendix.

B.12.2 For general grouting purposes, the material requirements for cement grout shall be in accordance with Chapter 3. The use of special grouts, containing epoxy or other binding media, or those used to achieve properties such as high strength, low shrinkage or expansion, or early strength gain, shall be qualified for use by the engineer and specified in contract documents.

B.12.3 Grouted embedments shall be tested to verify embedment strength. Grouted embedments installed in tension zones of concrete members shall be capable of sustaining design strength in cracked concrete. Tests shall be conducted by an independent testing agency and shall be certified by a professional engineer with full description and details of the testing programs, procedures, results, and conclusions.

B.12.4 Grouted embedments shall be tested for the installed condition by testing randomly selected grouted embedments to a minimum of 100% of the required strength. The testing program shall be established by the engineer.

B.12.5 The tests required by B.12.3 and B.12.4 may be waived by the engineer if tests and installation data are available to demonstrate that the grouted embedment will function as designed or if the load transfer through the grout is by direct bearing or compression.

APPENDIX C—Special Provisions for Impulsive and Impactive Effects

C.0—Notation

A_c = area of core of spirally reinforced column measured to the outside diameter of the spiral, sq in.

A_{ch} = area of rectangular core of column measured out-to-out of hoop, sq in.

A_g = gross area of section, sq in.

A_s = area of tension reinforcement within the width b, sq in.

A_s' = area of compression reinforcement within the width b, sq in.

b = width of compression face, in.

c = distance from extreme compressive fiber to neutral axis at ultimate strength, in.

d = effective depth of section (distance from extreme compressive fiber to centroid of tensile reinforcement), in.

f_c' = specified compressive strength of concrete, psi

f_y = specified yield strength of nonprestressed reinforcement, psi

h = overall thickness of member, in.

I_{cr} = moment of inertia of cracked section transformed to concrete

I_s = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement

ℓ_h = maximum unsupported length of rectangular hoop measured between perpendicular legs of the hoop or supplementary crossties, in.

r_q = rotational capacity, radians

R = resistance (i.e., load capacity)

R_m = maximum resistance

s_h = center-to-center spacing of hoops, in.

X_m = maximum acceptable displacement

X_y = displacement at effective yield point

r = reinforcement ratio = A_s / bd

r' = reinforcement ratio = A_s' / bd

r_s = ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out)

m_d = ductility ratio, dimensionless

C.1—Scope

C.1.1 Nuclear safety related concrete structures shall be designed for impulsive and impactive loads using this code and the special provisions of this appendix. These loads must be combined with other loads in accordance with 9.1 of this Code and in accordance with requirements of C.8 of this Appendix. Impactive and impulsive effects are treated separately herein because of the nature of the effects as well as the response characteristics of the structural elements subjected to these loads.

C.1.2 The provisions of this appendix apply to those structural elements directly affected by the impactive and impulsive loads and where failure of the structural elements must be precluded.

C.1.3 Applicable theoretical or experimental evidence may be used to justify requirements less conservative than those of this Appendix.

C.1.4 Impactive loads are time-dependent loads due to collision of masses which are associated with finite amounts of kinetic energy. Impactive loading may be defined in terms of time-dependent force or pressure. Impactive loads to be considered shall include, but not be limited to, the following types of loading:

(a) tornado-generated missiles;
(b) whipping pipes;
(c) aircraft missiles;
(d) fuel cask drop; and
(e) other internal and external missiles.

C.1.5 Impulsive loads are time-dependent loads which are not associated with collision of solid masses. Impulsive loads to be considered shall include, but not be limited to, the following types of loading:

(a) jet impingement;
(b) blast pressure;
(c) compartment pressurization; and
(d) pipe-whip restraint reactions.

C.2—Dynamic strength increase

C.2.1 Dynamic increase factors (DIF) appropriate for the strain rates involved may be applied to static material strengths of steel and concrete for purposes of determining section strength but shall not exceed the following:

Material	DIF
Reinforcing steel	
$\quad f_y$ = 40 ksi	1.20
$\quad f_y$ = 50 ksi	1.15
$\quad f_y$ = 60 ksi	1.10
Prestressing steel	1.00
Concrete	
$\quad$ Axial and flexural compression	1.25
$\quad$ Shear	1.10

C.3—Deformation

C.3.1 Permissible ductility ratio μ_d is defined as the ratio of the maximum acceptable displacement X_m to the displacement at the effective yield point X_y of the structural element (see Fig. C.3.1). In order to establish the effective yield displacement, the cross sectional moment of inertia shall be taken as 0.5 $(I_g + I_{cr})$. In addition to the deformation limits imposed under C.3.3 and C.3.4, the maximum deformation shall not result in the loss of intended function of the structural element nor impair the safety related function of other systems and components.

C.3.2 For impulsive loads, resistance available for impulsive loads shall be at least 20% greater than the magnitude of any portion of the impulsive loading which is approximately constant for a time equal to or greater than
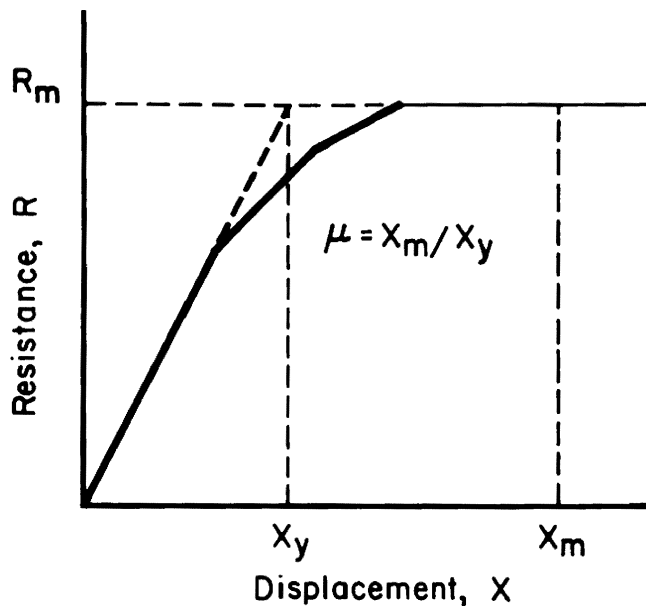
Fig. C.3.1—Idealized displacement-resistance-ductility relationship

the first fundamental period of the structural element. Consideration shall also be given to the requirements of C.8.

C.3.3 For beams, walls, and slabs where flexure controls design, the permissible ductility ratio shall either be taken as $0.05/(r - r')$ not to exceed 10, or shall be determined from the rotational capacity as defined in C.3.4.

C.3.4 When flexure controls design, the rotational capacity r_θ in radians of any yield hinge shall be limited to $0.0065\,(d/c)$ but shall not exceed 0.07 radians.

C.3.5 The permissible ductility ratio in flexure shall not exceed 3.0 for loads such as blast and compartment pressurization which could affect the integrity of the structure as a whole.

C.3.6 For flexure to control the design, thus allowing the ductility ratios or rotational capacities given in C.3.3 and C.3.4 and C.3.5 to be used, the load capacity of a structural element in shear shall be at least 20% greater than the load capacity in flexure, otherwise, the ductility ratios given in C.3.7 or C.3.9 shall be used.

C.3.7 For beams, walls, and slabs where shear controls design, the permissible ductility ratio shall be taken as:

(a) For shear carried by concrete alone, the permissible ductility ratio shall be 1.3;

(b) For shear carried by concrete and stirrups or bent bars, the permissible ductility ratio shall be 1.6; or

(c) For shear carried completely by stirrups, the permissible ductility ratio shall be 3.0.

C.3.8 For beam-columns, walls, and slabs carrying axial compression loads and subject to impulsive or impactive loads producing flexure, the permissible ductility ratio in flexure shall be as follows:

(a) When compression controls the design, as defined by an interaction diagram, the permissible ductility ratio shall be 1.3;

(b) When the compression load does not exceed $0.1 f_c' A_g$ or one-third of that which would produce balanced conditions, whichever is smaller, the permissible ductility ratio shall be as given in C.3.3 or C.3.4; and

(c) The permissible ductility ratio shall vary linearly from 1.3 to that given in C.3.3 or C.3.4 for conditions between those specified in (a) and (b).

C.3.9 For axial compressive impulsive or impactive loads, the permissible axial ductility ratio shall be 1.3.

C.4—Requirements to assure ductility

C.4.1 The minimum specified concrete strength f_c' shall be 3000 psi. The maximum specified yield strength of reinforcement f_y shall be 60,000 psi. Grade and area of flexural reinforcement used shall be only that specified; substitution of higher grades or larger areas shall not be permitted.

Reinforcement in structural elements resisting flexural or tension forces designed according to this Appendix shall comply with ASTM A 706. ASTM A 615 Grades 40 and 60 reinforcement are allowed in these members if:

(a) the actual yield strength based on mill tests does not exceed the specified yield strength by more than 18,000 psi (retests shall not exceed this value by more than an additional 3000 psi); and

(b) the ratio of the actual ultimate tensile strength to the actual tensile yield strength is not less than 1.25.

C.4.2 The maximum value of $(r - r')$ shall not exceed 0.5 times the ratio producing balanced conditions as defined by 10.3.2 and 10.3.3. Both top and bottom reinforcement for beams shall consist of not less than two bars and shall have a minimum r of $200/f_y$ throughout the entire length of the beam.

C.4.3 At least one-third of the larger amount of the tension reinforcement required at either end of a member shall be continuous throughout the length of the member. At least one-third of the maximum tension reinforcement required in the mid-region of a member shall be continuous throughout the length of the member and fully developed in tension beyond the face of the supports to its specified yield strength. These requirements apply to each direction of two-way structural elements.

C.4.4 The vertical reinforcement ratio in columns shall be limited to a minimum of 1.0% and a maximum of 6.0%. 10.8.4 does not apply.

C.4.5 Confinement reinforcement in columns consisting of spiral or hoop reinforcement shall be supplied above and below connections over a minimum length from the face of the connection at least equal to the overall depth h (h being the longer dimension in the case of rectangular columns or the diameter of a round column), 18 in., and one sixth of the clear height of the column.

C.4.5.1 Where a spiral is used, the volumetric ratio r_s shall be not less than indicated by Eq. (10-6), but not less than $0.12 f_c'/f_y$.

C.4.5.2 Where rectangular hoop reinforcement is used, the required area of the bar shall be computed by:

$$A_{xh} = \frac{\ell_h \mathbf{r}_s s_h}{2} \qquad \text{(C.4-1)}$$

where $\mathbf{r}_s$ is the volumetric ratio required by C.4.5.1 with A_{ch} substituted for A_c and with f_y the yield strength of the hoop reinforcement. The center-to-center spacing between hoops or the pitch of continuous hoops shall not exceed 4 in. Minimum bar size shall be that required for ties by 7.10.5.1. Supplementary crossties of the same bar size as the hoop may be used to reduce the unsupported length, ℓ_h. Each end of the supplementary crossties shall engage the periphery hoop with a standard semicircular hook, and shall be secured to a longitudinal bar to prevent displacement of the crosstie during construction. Minimum cover of supplementary crossties reinforcement shall be 1/2 in.

C.5—Shear strength

The shear strength of slabs and walls under local loads shall consider both punching shear adjacent to the load and reaction shear at supports. Local loads may be impulsive or impactive, except that for certain impactive loads satisfaction of criteria for perforation replaces punching shear requirements (see C.7.2.3).

The shear strength of concrete beams and columns shall be determined in accordance with 11.1 to 11.5 of this code increased by the DIF of C.2. These provisions shall also apply in cases of reaction shear at supported edges of slabs and walls. Punching shear strength of slabs and walls shall be determined in accordance with 11.12 of this Code, increased by the DIF of C.2.

C.6—Impulsive effects

C.6.1 Impulsive loads shall be considered in combination with other loads as required by 9.1 of this Code and in accordance with C.8 of this Appendix.

C.6.2 When reinforced concrete structural elements or systems of elements are subjected to impulsive loads, the structural response may be determined by one of the following methods:

(a) The dynamic effects of the impulsive loads may be considered by calculating a dynamic load factor (DLF). The resistance available for the impulsive load must be at least equal to the peak of the impulsive load transient multiplied by the DLF.

The calculation of the DLF shall be based on the ductility criteria in C.3 and the dynamic characteristics of the structure and impulsive load transient;

(b) The dynamic effects of impulsive loads may be considered by using impulse, momentum, and energy balance techniques. Strain energy capacity is limited by the ductility criteria in C.3; or

(c) The dynamic effects of impulsive loads may be considered by performing a time-history dynamic analysis. Mass and inertial properties shall be included as well as the nonlinear stiffnesses of structural elements under consideration. Simplified bilinear definitions of stiffness are acceptable.

Maximum predicted response is governed by the ductility criteria in C.3.

C.7—Impactive effects

C.7.1 Design for impactive loads shall satisfy the criteria for both local effects and for overall structural response.

C.7.2 Local impact effects may include penetration, perforation, scabbing, and punching shear.

C.7.2.1 The penetration depth and required concrete thickness to prevent perforation shall be based upon applicable formulas or pertinent test data. When perforation of concrete structural elements must be precluded, the concrete thickness shall be at least 20% greater than that required to prevent perforation.

C.7.2.2 Concrete structural elements protecting required system or equipment which could be damaged by secondary missiles (fragments of scabbed concrete) shall be designed to prevent scabbing, or a properly designed scab shield shall be based on applicable formulas or pertinent test data. In the absence of scab shields, the concrete thickness shall be at least 20% greater than that required to prevent scabbing.

C.7.2.3 When it can be demonstrated by applicable formulas or pertinent test data that the concrete thickness is at least 20% greater than that required to prevent perforation and hence punching shear failure, design for punching shear in accordance with C.5 is not required.

C.7.2.4 For concrete slabs or walls subjected to missile impact effects where the concrete thickness is less than twice that required to prevent perforation, the minimum percentage of reinforcement shall be 0.2% each way, each face.

C.7.3 When reinforced concrete structural elements or systems of elements are subjected to impactive loads, the structural response may be determined by the methods described in C.6.2.

C.8—Impactive and impulsive loads

Impactive and impulsive loads must be considered *concurrent* with other loads (e.g., dead and live load) in determining the required resistance of structural elements.

APPENDIX D—SI Metric Equivalents of U.S. Customary Units

The following is not part of this standard, but SI metric equivalents of all the dimensional values in this code and conversions of nonhomogeneous equations are given below for convenience of users.

In this tabulation, SI metric units are based on the standards given in ASTM E380-76 and the preferred units in that standard. These metric units are those conforming to the requirements of the U.S. Metric Standards Act of 1975.

METRIC EQUIVALENTS OF DIMENSIONAL UNITS

Area

U.S. customary	Metric
5000 ft^2	465 m^2

Moment of inertia

U.S. customary	Metric
1 in.^{4}	416,000 mm^4

Temperature

U.S. customary, deg F	Metric, deg C
40	4.4
50	10
60	16
80	27
90	32
95	35
150	66

Weight (density)

U.S. customary, lb/ft^3	Metric, kg/m^3
1	16
70	1120
90	1440
115	1840
120	1920
145	2320
155	2420

Length

U.S. customary	Metric	U.S. customary	Metric
1 in.	25.4 mm*	6 in.	152 mm
0.15 in.	3.8 mm	8 in.	203 mm
$^1/_4$ in.	6.4 mm	10 in.	254 mm
$^3/_8$ in.	9.5 mm	12 in.	305 mm
$^1/_2$ in.	13 mm	16 in.	406 mm
$^5/_8$ in.	16 mm	18 in.	457 mm
$^3/_4$ in.	19 mm	20 in.	508 mm
1-$^1/_4$ in.	32 mm	24 in.	610 mm
1-$^1/_2$ in.	38 mm	30 in.	762 mm
2 in.	51 mm	1 ft	0.3048 m*
2-$^1/_2$ in.	64 mm	3 ft	0.9 m
3 in.	76 mm	10 ft	3.0 m
3-$^1/_2$ in.	89 mm	12 ft	3.7 m
4 in.	102 mm	15 ft	4.6 m
5 in.	127 mm	25 ft	7.6 m

* Exact   * Exact

METRIC EQUIVALENTS OF DIMENSIONAL UNITS (continued)

Stress (pressure)

U.S. customary	SI-Metric	U.S. customary	SI-Metric
psi	kPa	psi	MPa
1	6.9	1,000	6.89
50	340	1,200	8.27
60	410	2,500	17.24
80	550	3,000	20.68
120	830	3,500	24.13
psi	MPa	4,000	27.58
145	1.0	4,500	31.03
150	1.03	5,000	34.47
200	1.38	9,000	62.10
250	1.72	20,000	137.9
300	2.07	24,000	165.5
350	2.41	30,000	206.8
400	2.76	40,000	275.8
500	3.45	50,000	344.7
550	3.79	60,000	413.7
600	4.14	80,000	551.6
700	4.83	100,000	690.0
800	5.52	29,000,000	200,000
900	6.21		

Load

U.S. customary	SI-Metric
1 lb	4.5 N
9000 lb	40,000 N
1 lb/ft	14.6 N/m
1 lb/ft^2	47.9 Pa

Volume

U.S. customary	Metric
1 in.^{4}	416,000 mm^4

SI METRIC EQUIVALENTS OF LIMITING VALUES

Units

	U.S. customary	Metric
Area	sq in.	mm^2
	sq ft	m^2
Density	lb per cu ft	kg/m^3
Load	lb	N
Size	in.	mm
	ft	m
Stress	psi	MPa

SI METRIC EQUIVALENTS OF LIMITING VALUES (continued)

Stress

U.S. customary	Metric	U.S. customary	Metric
$\sqrt{f_c'}$	$0.08\sqrt{f_c'}$	$3.3\sqrt{f_c'}$	$0.27\sqrt{f_c'}$
$0.5\sqrt{f_c'}$	$0.04\sqrt{f_c'}$	$3.5\sqrt{f_c'}$	$0.29\sqrt{f_c'}$
$0.6\sqrt{f_c'}$	$0.05\sqrt{f_c'}$	$4.0\sqrt{f_c'}$	$0.33\sqrt{f_c'}$
$(^2/_3)\sqrt{f_c'}$	$0.06\sqrt{f_c'}$	$4.4\sqrt{f_c'}$	$0.37\sqrt{f_c'}$
$1.1\sqrt{f_c'}$	$0.09\sqrt{f_c'}$	$5.0\sqrt{f_c'}$	$0.42\sqrt{f_c'}$
$1.2\sqrt{f_c'}$	$0.10\sqrt{f_c'}$	$5.5\sqrt{f_c'}$	$0.46\sqrt{f_c'}$
$1.25\sqrt{f_c'}$	$0.10\sqrt{f_c'}$	$6.0\sqrt{f_c'}$	$0.50\sqrt{f_c'}$
$1.5\sqrt{f_c'}$	$0.12\sqrt{f_c'}$	$6.3\sqrt{f_c'}$	$0.52\sqrt{f_c'}$
$1.6\sqrt{f_c'}$	$0.13\sqrt{f_c'}$	$6.5\sqrt{f_c'}$	$0.54\sqrt{f_c'}$
$1.7\sqrt{f_c'}$	$0.14\sqrt{f_c'}$	$7.0\sqrt{f_c'}$	$0.58\sqrt{f_c'}$
$1.9\sqrt{f_c'}$	$0.16\sqrt{f_c'}$	$7.5\sqrt{f_c'}$	$0.62\sqrt{f_c'}$
$2.0\sqrt{f_c'}$	$0.17\sqrt{f_c'}$	$8.0\sqrt{f_c'}$	$0.66\sqrt{f_c'}$
$2.4\sqrt{f_c'}$	$0.20\sqrt{f_c'}$	$10.\sqrt{f_c'}$	$0.83\sqrt{f_c'}$
$3.0\sqrt{f_c'}$	$0.25\sqrt{f_c'}$	$12.0\sqrt{f_c'}$	$1.00\sqrt{f_c'}$

U.S. customary expression	Metric equivalent	U.S. customary expression	Metric equivalent
Section 8.5.1		**Section 11.6.6.2**	
$w_c^{1.5}33\sqrt{f_c'}$	$w_c^{1.5}0.043\sqrt{f_c'}$	$1+\dfrac{1}{500}\dfrac{N_u}{A_g}$	$1+0.29\dfrac{N_u}{A_g}$
$57,000\sqrt{f_c'}$	$4730\sqrt{f_c'}$	**Section 11.7.5**	
Table 9.5(b)—Footnote		$800A_c$	$5.5A_c$
$0.4+\dfrac{f_y}{100,000}$	$0.4+\dfrac{f_y}{690}$	**Section 12.2.2**	
Table 9.5(c)—Footnote		$\dfrac{0.04A_bf_y}{\sqrt{f_c'}}$	$\dfrac{0.019A_bf_y}{\sqrt{f_c'}}$
$\dfrac{800-0.005f_y}{1100}$	$\dfrac{1104-f_y}{1518}$	$0.0004d_bf_y$	$0.058d_bf_y$
Section 10.5.1		$\dfrac{0.085f_y}{\sqrt{f_c'}}$	$\dfrac{26f_y}{\sqrt{f_c'}}$
$\dfrac{200}{f_y}$	$\dfrac{1.4}{f_y}$	$\dfrac{0.11f_y}{\sqrt{f_c'}}$	$\dfrac{34f_y}{\sqrt{f_c'}}$
Section 10.6.4		$\dfrac{0.03d_bf_y}{\sqrt{f_c'}}$	$\dfrac{0.36d_bf_y}{\sqrt{f_c'}}$
175 kips per in.	30.6 MN/m		
145 kips per in.	25.4 MN/m		

SI METRIC EQUIVALENTS OF LIMITING VALUES (continued)

U.S. customary expression	Metric equivalent	U.S. customary expression	Metric equivalent

Section 12.2.3

$$2 - \frac{60,000}{f_y}$$

$$2 - \frac{414}{f_y}$$

Section 12.8.2

$$\frac{0.03d_b(f_y - 20,000)}{\sqrt{f_c'}}$$

$$\frac{0.36d_b(f_y - 138)}{\sqrt{f_c'}}$$

$$\frac{6.7\sqrt{f_c'}}{f_{ct}}$$

$$\frac{0.56\sqrt{f_c'}}{f_{ct}}$$

$$0.20\frac{A_w}{s_w}\frac{f_y}{\sqrt{f_c'}}$$

$$2.4\frac{A_w}{s_w}\frac{f_y}{\sqrt{f_c'}}$$

Section 12.3.2

Section 12.9

$$\frac{0.02f_yd_b}{\sqrt{f_c'}}$$

$$\frac{0.24f_yd_b}{\sqrt{f_c'}}$$

$$0.27\frac{A_w}{s_w}\frac{f_y}{\sqrt{f_c'}}$$

$$3.3\frac{A_w}{s_w}\frac{f_y}{\sqrt{f_c'}}$$

$$0.0003f_yd_b$$

$$0.044f_yd_b$$

Section 12.10.1

$$\left(f_{ps} - \frac{2}{3}f_{se}\right)d_b$$

$$0.145\left(f_{ps} - \frac{2}{3}f_{se}\right)d_b$$

Table 12.5.1—ξ values

Section 12.11.5.2

$$\frac{60b_ws}{f_y}$$

$$\frac{0.41b_ws}{f_y}$$

U.S. customary $f_h = \xi\sqrt{f_c'}$	Metric $f_h = \xi\sqrt{f_c'}$
220	18
330	27
360	30
420	35
450	37
480	40
540	45

Section 12.17.1

$$0.0005f_yd_b$$

$$0.073f_yd_b$$

Sections 17.5.2.1 and 17.5.2.2

$$80b_vd$$

$$0.55b_vd$$

Section 17.5.2.3

$$500b_vd$$

$$3.5b_vd$$

SI METRIC CONVERSIONS OF NONHOMOGENEOUS EQUATIONS

Eq. (8-1)

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{600}{600 + f_y}\right)$$

Eq. (11-5)

$$V_c = \frac{0.17\sqrt{f_c'}b_wd}{\sqrt{1 + \left(2.5C_t\frac{T_u}{V_u}\right)^2}}$$

Eq. (11-3)

$$V_c = 0.17\sqrt{f_c'}b_wd$$

Eq. (11-6)

$$V_c = \left(0.16\sqrt{f_c'} + 17.2\rho_w\frac{V_ud}{M_u}\right)b_wd$$

Eq. (11-4)

$$V_c = 0.17\left(1 + 0.073\frac{N_u}{A_g}\right)\sqrt{f_c'}b_wd$$

Eq. (11-8)

$$V_c = 0.29\sqrt{f_c'}b_wd\sqrt{1 + 0.29\frac{N_u}{A_g}}$$

SI METRIC CONVERSIONS OF NONHOMOGENEOUS EQUATIONS (continued)

Eq. (11-9)

$$V_c = 0.17\left(1 + 0.29\frac{N_u}{A_g}\right)\sqrt{f_c'}\,b_w d$$

Eq. (11-10)

$$V_c = \left(0.05\sqrt{f_c'} + 4.8\frac{V_u d}{M_u}\right)b_w d$$

Eq. (11-11)

$$V_{ci} = 0.05\sqrt{f_c'}\,b_w d + V_d + \frac{V_i M_{cr}}{M_{max}}$$

Eq. (11-12)

$$M_{cr} = (I/y_t)(0.5\sqrt{f_c'} + f_{pe} - f_d)$$

Eq. (11-13)

$$V_{cw} = (0.29\sqrt{f_c'} + 0.3 f_{pc})b_w d + V_p$$

Eq. (11-14)

$$A_v = 0.34\frac{b_w s}{f_y}$$

Eq. (11-16)

$$(A_v + 2A_t) = 0.34\frac{b_w s}{f_{yv}}$$

Eq. (11-22)

$$T_c = \frac{0.07\sqrt{f_c'}\sum x^2 y}{\sqrt{1 + \left(\dfrac{0.4 V_u}{C_t T_u}\right)^2}}$$

Eq. (11-25)

$$A_\ell = \left[\frac{2.76 xs}{f_y}\left(\frac{T_u}{T_u + \dfrac{V_u}{3C_t}}\right) - 2A_t\right]\left(\frac{x_1 + y_1}{s}\right)$$

Eq.: Section 11.6.9.3

$$0.34\frac{b_w s}{f_y}\ \text{for } 2A_t$$

Eq. (11-28)

$$V_n = 0.06\left(10 + \frac{\ell_n}{d}\right)\sqrt{f_c'}\,b_w d$$

Eq. (11-30)

$$V_c = \left(3.5 - 2.5\frac{M_u}{V_u d}\right)\left(0.16\sqrt{f_c'} + 17\rho_w\frac{V_u d}{M_u}\right)b_w d$$

Eq. (11-32)

$$V_c = 0.27\sqrt{f_c'}\,hd + \frac{N_u d}{4\ell_w}$$

Eq. (11-33)

$$V_c = \left[0.05\sqrt{f_c'} + \frac{\ell_w\left(0.1\sqrt{f_c'} + 0.2\dfrac{N_u}{\ell_w h}\right)}{\dfrac{M_u}{V_u} - \dfrac{\ell_w}{2}}\right]hd$$

Eq. (11-36)

$$V_c = 0.083\left(2 + \frac{4}{\beta_c}\right)\sqrt{f_c'}\,b_o d$$

Eq. (11-37)

$$V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f_c'}\,b_o d/12$$

Eq. (11-38)

$$V_c = \sqrt{f_c'}\,b_o d/3$$

Eq. (11-39)

$$V_c = (\beta_p\sqrt{f_c'} + 0.3 f_{pc})b_0 d + V_p$$

Eq. (18-4)

$$f_{ps} = f_{se} + 69 + \frac{f_c'}{100\rho_p}$$

SI METRIC CONVERSIONS FOR APPENDIX B CODE AND COMMENTARY

Section	Metric conversion	Section	Metric conversion
B.0	in. shall be mm; in.^{2} shall be mm^2; psi shall be MPa; and lb shall be N		In Eq. (B-18b), Change 8 to 0.66
		B.6.2.7	Change No. 4 to No. 13 and 4 in. to 100 mm
B.1	Change 2 in. to 50 mm		
B.3.5	Change 10,000 psi to 70 MPa and Change 8000 psi to 55 MPa	**B.6.3.1**	Change 2.5 in. to 65 mm
		B.8.5	Change 4 in. to 100 mm
B.4.2.2	Change 2 in. to 50 mm and Change 25 in. to 625 mm	**RB.5.2.2**	Change 11 in. to 280 mm and Change 25 in. to 635 mm
B.5.1.2	Change 125,000 psi to 860 MPa	**RB.5.2.3**	Change 4 in. to 100 mm and Change 2.67 in. to 67 mm
B.5.2.2	Change k = 24 to k = 10; Change k = 17 to k = 7; Change k = 16 to k = 3.8; Change 11 in. to 280 mm; and Change 25 in. to 635 mm	**RB.5.2.6**	Change 0.012 in. to 0.3 mm
		RB.6.2.2	Change "constant 7" to "constant 0.6"
B.5.4.1	In Eq. (B-11), Change 160 to 13.3	**RB.6.3**	Change 2.5 in. to 65 mm
B.6.1.2	Change 125,000 psi to 860 MPa	**Fig. RB.5.2(a)**	Change 4 in. to 100 mm; Change 8 in. to 200 mm; and Change 2.67 in. to 67 mm
B.6.2.2	In Eq. (B-18a), Change 7 to 0.6		
B.6.2.3	Change 3/8 in. to 10 mm;		

Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349R-01)

Reported by ACI Committee 349

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This commentary discusses some of the considerations of ACI Committee 349 in developing "Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01)." This information is provided in the commentary because the Code is written as a legal document and cannot therefore present background details or suggestions for carrying out its requirements.

ACI 349-01 is based on "Building Code Requirements for Structural Concrete (ACI 318-95)," except for Chapter 12, which is based on ACI 318-99. This commentary provides discussion of those requirements in ACI 349 that differ from those of ACI 318.

Keywords: admixtures; aggregates; anchorage (structural); beam-column frame; beams (supports); building codes; cements; cold weather construction; columns (supports); combined stress; composite construction (concrete and steel); composite construction (concrete to concrete); compressive strength; concrete construction; concretes; concrete slabs; construction joints; continuity (structural); cover; curing; deep beams; deflection; earthquake resistant structures; edge beams; embedded service ducts; flexural strength; floors; folded plates; footings; formwork (construction); frames; hot weather construction; inspection; joists; loads (forces); load tests (structural); mixing; mixture proportioning; modulus of elasticity; moments; nuclear power plants; nuclear reactor containment; pipe columns; pipes (tubes); placing; precast concrete; prestressed concrete; prestressing steels; quality control; reinforced concrete; reinforcing steels; roofs; safety; serviceability; shear strength; shearwalls; shells (structural forms); spans; specifications; splicing; stability standards; strength; stresses; structural analysis; structural design; T-beams; temperature; tensile strength; torsion; walls; water; welded wire fabric.

CONTENTS

INTRODUCTION

This commentary discusses some of the considerations of Committee 349 in developing the provisions contained in "Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01)" hereinafter called the Code. The Code is based on "Building Code Requirements for Reinforced Concrete (ACI 318-95)," which is hereinafter referred to as the Building Code. This commentary discusses provisions in the Code which differ from the Building Code. In preparing ACI 349-01, the committee has followed the text of the Building Code wherever appropriate.

In the following commentary, all references to the Building Code and its commentary are to the 1995 edition unless specifically noted otherwise. Provisions of ACI 318R apply, except replace the term "building official" with "engineer."

CHAPTER R1—GENERAL REQUIREMENTS

The commentary on ACI 318 is applicable to this chapter except as described below:

R1.1—Scope

The American Concrete Institute "Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01)," referred to as "the Code," provides minimum requirements for reinforced concrete design or construction in applications where protection against potential radioactive releases is a concern. The scope of the Code has historically addressed new construction; however, this scope has recently been extended to providing recommendations for the analysis, design, construction, testing, and evaluation of new and existing concrete nuclear structures. While the requirements of this Code pertain primarily to new concrete structures, corresponding recommendations for the evaluation of existing concrete nuclear structures are provided in ACI 349.3R-96.[1.3]

The Owner is to identify nuclear safety related structures and establish which of them are covered by "Code for Concrete Reactor Vessels and Containments (ACI 359)" and its latest revisions in lieu of this Code.

Some special structures involve unique problems that are not covered by the Code. While the requirements of this Code pertain primarily to new concrete structures, corresponding recommendations for the evaluation of existing concrete nuclear structures have been developed by the committee separately (ACI 349.3R-96).

The Code is applicable to radioactive waste repository structures; however, considerations of thermal loads, load combinations, and long-term durability should be considered.

R1.2—Drawings, specifications, and calculations

The design of plain concrete is not included in this Code. Details of all contraction joints and isolation joints in reinforced-concrete structures, however, are considered important to the as-built condition of the structure.

Guidelines for the preparation and retention of design documents are covered by ANSI/ASME NQA-1.[1.1] Any documentation that uniquely reflects the as-built condition of the concrete nuclear structure should be considered for retention as a permanent record for the life of the structure.

Drawings and specifications should be prepared under the direction of a licensed (or registered) engineer competent in the field of design of concrete structures, who is required to sign these documents signifying his approval. This Code requires that the Owner be responsible for drawings and calculations, but does not preclude him from assigning the function of detailed implementation to others.

R1.2.3 Whenever the Building Code uses the term "building official," it shall be replaced (when using ACI 349R) with "engineer."

R1.3—Inspection and record keeping

This Code requires that the Owner be responsible for inspection but does not explicitly preclude him from assigning the function of detailed implementation to others.

Inspection personnel should be qualified by the Owner. ANSI/ASME NQA-1[1.1] or ACI 359 Appendix VII may be used to qualify inspectors.

Requirements for the retention of inspection records should follow ANSI/ASME NQA-1.[1.1]

R1.4—Approval of special systems of design or construction

New methods of design, new materials, and new uses of materials must undergo a period of development before being specifically covered in a code. Hence, good systems or components might be excluded from use by implication if means were not available to obtain acceptance. This section permits proponents to submit data substantiating the adequacy of their system or component to the Regulatory Authority, which presently is the United States Nuclear Regulatory Commission (USNRC) in the U.S.

R1.5—Quality assurance program

Title 10 CFR 50, Appendix B, requires that the Owner have a quality assurance program approved by the Regulatory Authority and states that the Owner is responsible for the establishment and execution of programs developed by his engineers, construction contractors, and suppliers. More detailed requirements for development and implementation of a quality assurance program are contained in ANSI/ASME NQA-1[1.1] and ANSI/ASME NQA-2.[1.2]

References

1.1. American National Standards Institute (ANSI) and American Society of Mechanical Engineers (ASME), "Quality Assurance Requirements for Nuclear Facilities," NQA-1, 1989.

1.2 ANSI/ASME, "Quality Assurance Requirements for Nuclear Facility Applications," NQA-2, 1989.

1.3 ACI Committee 349, "Evaluation of Existing Nuclear Safety Related Concrete Structures (ACI 349.3R-96)," American Concrete Institute, Farmington Hills, Mich., 1996, 19 pp.

CHAPTER R2—DEFINITIONS

Definitions in this Code are the same as those in the Building Code (ACI 318) except for a few that are added or modified to meet the structures and materials concerned in this Code.

The definitions that differ from or are not listed in ACI 318 are:

Cementitious material

Creep

Embedment

Engineer

Evaluation

Effective prestress
Load, dead
Load, live
Load, sustained
Massive concrete
Operating basis earthquake (OBE)
Operating basis wind
Owner
Regulatory Authority
Safe shutdown earthquake (SSE)
Shrinkage
Stress relaxation
Tendon
Unbonded tendons
Wall

In the definition of the term "engineer," the phrase "other documents" is used. "Other documents" include quality assurance plans, design guidelines, and other similar documents for which the engineer is responsible. The user of ASCE 7-95 for ACI 349 applications should be warned of the differences, such as the need to recreate Table 6.1 for 100 year recurrence using Table C6.5 and its commentary.

CHAPTER R3—MATERIALS

The commentary on ACI 318 is applicable to this chapter except as described below:

R3.1—Tests of materials

R3.1.3 The Owner shall designate the period of retention for all records of tests of materials and of concrete used in concrete nuclear structures. Typically, any documentation that uniquely describes tests of materials and of concrete used in concrete nuclear structures should be retained for the life of the plant.

R3.2—Cements

R3.2.1 Adequate performance of expansive cements should be demonstrated for the particular application before selecting such materials, such as in areas of high temperature and irradiation. Additionally, the user should address potential adverse reactions with dissimilar metals that can exist in certain expansive hydraulic cements.

R3.2.3 The requirement for receipt of certified 7-day mill test materials reports in advance of use is imposed to inform the materials engineer of changes in cement strength to provide guidance in altering concrete mixtures when significant changes in strength occur. Such alterations can both achieve increased assurance against low strengths and reduce the standard deviation of strengths, providing a means of optimizing the cement contents and reducing the heat of hydration effects in these relatively massive structures.

R3.3—Aggregates

R3.3.1 The reference to lightweight aggregates has been deleted. The minimum thickness of most concrete members in nuclear plant construction is based on shielding requirements that are dependent on the density of the concrete. Lightweight aggregates would require larger minimum thicknesses because the sizing of columns and spacing of walls, is usually controlled by live load, seismic, or shielding requirements and not floor dead loads. There appears to be no advantage in using lightweight aggregates in nuclear structures.

R3.3.3 Minimum testing requirements are specified to assure aggregate quality.

R3.3.3.3 Anytime there is a basic change in aggregate source, the aggregate should be tested to determine suitability for use. Retests for soundness (ASTM C 88), abrasion resistance (ASTM C 131), and potential reactivity (ASTM C 289) are required because they are not included in the routine testing of aggregates.

R3.4—Water

Guidance for water quality may be obtained from Subparagraph CC-2223 of ACI 359-92.

R3.5—Steel reinforcement

R3.5.3 Zinc used in the galvanizing process may negatively react with alkaline materials commonly found in concrete. In addition, potential galvanic corrosion with other embedded metals, as well as hydrogen generation and potential for hydrogen embrittlement suggest that such coatings may be detrimental. Research conducted by Sergi[3.2] concluded that zinc coatings provide little value in providing long-term protection of reinforcing steel, and cautionary statements in ACI Report 201.2R-92[3.3] support this position. These industry concerns have prompted the ACI Committee 349 to prohibit use of zinc coatings on reinforcing steel in nuclear safety related structures until adequate data justifying its use can be reviewed.

R3.5.3.1 and **R3.5.4.1** Use of rail and axle steel reinforcing bars is excluded because of inadequate traceability.

R3.5.3.2 Testing requirements for reinforcing steel are given to conform to regulatory guides. A minimum of one tensile test for each 50 tons of each bar size produced from each heat of steel shall be used, whichever governs.

R3.5.3.3, R3.5.3.5, R3.5.3.6, R3.5.3.7, and **R3.5.4.2** To limit the sizes of cracks in the relatively massive concrete sections, design yield strengths are limited to a maximum of 60,000 psi for deformed bar reinforcement. For further comments on the 60,000 psi reinforcement limit, refer to R.9.4. For further comments on the design yield strength limitation, refer to R9.4.

R3.5.3.8 Current industry studies have not quantified the long-term durability of epoxy coatings in areas of elevated temperature and irradiation. Degradation of the epoxy coating, under certain environmental conditions, may adversely affect reinforcing steel performance and anchorage, resulting in splitting effects. Adequate performance of the epoxy-coated reinforcement and concrete should be demonstrated for the particular environmental application before selecting epoxy-coated reinforcement.

R3.6—Admixtures

R3.6.7 Adequate performance of ground-granulated blast-furnace slag should be demonstrated for the particular application before selecting such materials, such as in areas of high temperature and irradiation.

R3.6.10 For quality assurance, all admixtures should be fully tested and verified to be in compliance with the referenced ASTM test methods prior to use.

R3.7—Storage and identification of materials

R3.7.1 Expanded emphasis on the protection and traceability of materials in storage is given to assure that the proper materials are used and to minimize deterioration of these materials during storage.

Certified Material Test Reports (MTR) provided for concrete material, reinforcing system material, prestressing system material, and welding and brazing material, should include the following:

(a) Certified reports of the actual results of all required chemical analyses, physical tests, mechanical tests, examinations (including radiographic film), repairs, and heat treatments (including times and temperatures) performed on the material.

(b) A statement listing any chemical analyses, tests, examinations, and heat treatment required by the material specification, which were not performed.

(c) A statement giving the manner in which the material is identified, including a specific marking.

R3.7.2 Concrete materials should be handled and stored in accordance with Chapter 2 of ACI 304R[3.1]

R3.7.4 To prevent detrimental corrosion, prestressing systems should normally be stored in a completely enclosed building.

References

3.1. ACI Committee 304, "Guide for Measuring, Mixing, Transporting, and Placing Concrete (ACI 304R-89)," American Concrete Institute, Farmington Hills, Mich., 1989, 49 pp. Also, *ACI Manual of Concrete Practice*.

3.2. Sergi, G.; Short, N.; and Page, C., "Corrosion of Galvanized and Galvannealed Steel in Solutions of pH 9.0 to 14.0," National Association of Corrosion Engineers, Nov. 1985.

3.3. ACI Committee 201, "Guide to Durable Concrete (ACI 201.2R-92)," *ACI Materials Journal,* V. 88, No. 5, Sept.-Oct. 1991, pp. 544-582. Also, *ACI Manual of Concrete Practice*.

CHAPTER R4—DURABILITY REQUIREMENTS

The commentary on ACI 318-95 is applicable to this chapter. Certain sections of the Commentary in ACI 318R-95, Chapter 4, have been repeated in this Commentary chapter to maintain correspondence with footnote numbers in the References section. All references to lightweight aggregate concrete have been deleted.

R4.2—Freezing and thawing exposures

In 4.2 on freezing and thawing exposures, the quantity of fly ash and other pozzolans used in the calculation of the water-cementitious materials ratio is subject to the percentage limits in 4.2.3.

R4.2.1—A table of required air contents for frost-resistant concrete is included in the Code, based on "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete" (ACI 211.1).[4.3] Values are provided for both severe and moderate exposures depending on the exposure to moisture or deicing salts. Entrained air will not protect concrete containing coarse aggregates that undergo disruptive volume changes when frozen in a saturated condition. In Table 4.2.1, a severe exposure is where the concrete in a cold climate may be in almost continuous contact with moisture prior to freezing, or where deicing salts are used. Examples are pavements, bridge decks, sidewalks, parking garages, and water tanks. A moderate exposure is where the concrete in a cold climate will be only occasionally exposed to moisture prior to freezing, and where no deicing salts are used.

Examples are certain exterior walls, beams, girders, and slabs not in direct contact with soil. Section 4.2.1 permits 1% lower air content for concrete with f_c' greater than 5000 psi. Such high-strength concretes will have lower water-cementitious materials ratios and porosity and, therefore, improved frost resistance. References to lightweight aggregates and lightweight concrete have been omitted from these Code and Commentary discussions.

R4.2.2 For normalweight aggregate concrete, use of both minimum strength and maximum water-cementitious materials ratio provide additional assurance that a high-quality cement paste will be obtained.

Table 4.2.2 and Code Section 4.2.3 establish limitations on the amount of fly ash and other pozzolans that can be included in the calculation of water-cementitious materials ratios for concrete exposed to deicing chemicals.[4.4-4.6] Recent research has demonstrated that the use of fly ash and silica fume produce concrete with a finer pore structure and, therefore, lower permeability.[4.7-4.9] For concretes exposed to deicing salts, the inclusion of maximum percentages of fly ash in the calculation of water-cementitious materials ratio is considered preferable to inclusion of minimum cement content required by the Code.

R4.2.3 The limitations of this section apply only to concrete exposed to deicing chemicals and are intended to provide protection against deicer scaling in the presence of freezing and thawing.

R4.3—Sulfate exposures

The commentary for this section on ACI 318-95 is applicable to this section. All references to lightweight aggregate concrete have been deleted. Further, in 4.3 for sulfate exposures, the pozzolan should be Class F by ASTM C 618,[4.1] or have been tested by ASTM 1012[4.2] or determined by service record to improve sulfate resistance.

R4.4—Corrosion protection of reinforcement

Additional information on the effects of chlorides on the corrosion of reinforcing steel is given in "Guide to Durable Concrete" reported by ACI Committee 201[4.10] and "Corrosion of Metals in Concrete" reported by ACI Committee 222.[4.11] Test procedures must conform to those given in ASTM C 1218. The FHWA test referenced[4.15] is also applicable for determining chloride ion content in concrete. An initial evaluation may be obtained by testing individual concrete ingredients for total chloride ion content. If total chloride ion content, calculated on the basis of concrete proportions, exceeds those permitted in Table 4.4.1, it may be necessary to test samples of the hardened concrete for water soluble chloride ion content described in the guide. Some of the total chloride ions present in the ingredients will either be insoluble or will react with the cement during hydration and become insoluble under the test procedures described.

When concretes are tested for soluble chloride ion content the tests should be made at an age of 28 to 42 days. The limits in Table 4.4.1 are to be applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.

The chloride ion limits in Table 4.4.1 differ from those recommended in ACI 318, ACI 201.2R, and ACI 222R. The

limits for reinforced and prestressed concrete of 0.06 and 0.15%, respectively, represent the lowest value presented in these documents. For simplicity and to reflect the more critical nature of safety related structures, more restrictive limits have been adopted by the committee.

R4.4.2 When concretes are exposed to external sources of chlorides, the water-cementitious materials ratio and specified compressive strength $f_c'c$ of 4.2.2 are the minimum requirements that must be considered. The designer should evaluate conditions in structures where chlorides may be applied or in structures near seawater. Epoxy-coated bars or cover greater than the minimum required in 7.7 may be desirable. Use of fly ash meeting ASTM C 618 and increased levels of specified strength provide increased protection. Silica fume, conforming to ASTM C 1240, when combined with an appropriate high-range water reducer (ASTM C 494, Types F and G, or ASTM C 1017), can also provide additional protection.[4.13] Performance tests for chloride permeability by AASHTO T 277[4.14] or ASTM C 1202 of concrete mixtures prior to use will also provide additional assurance.

References

4.1. Dikeou, J. T., "Fly Ash Increases Resistance of Concrete to Sulfate Attack," *Research Report* No. C-1224, Concrete and Structures Branch, Division of Research, U.S. Bureau of Reclamation, Jan. 1967, 25 pp.

4.2. ASTM C 1012-89, "Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution," *ASTM Book of Standards*, Part 04.01, ASTM, West Conshohocken, Pa., p. 5.

4.3. ACI Committee 211, "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete (ACI 211.1-91)," American Concrete Institute, Farmington Hills, Mich., 1991, 38 pp. Also, *ACI Manual of Concrete Practice*, Part 1.

4.4. Drahushak-Crow, Roselle, "Freeze-Thaw Durability of Fly Ash Concrete," EPRI *Proceedings*, Eighth International Ash Utilization Symposium, V. 2, Oct. 1987, p. 37-1.

4.5. Sivasundarum, V.; Carette, G. G.; and Malhotra, V. M., "Properties of Concrete Incorporating Low Quantity of Cement and High Volumes of Low-Calcium Fly Ash," *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete*, SP-114, V. 1, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, Mich., 1989, pp. 45-71.

4.6. Whiting, D., "Deicer Scaling and Resistance of Lean Concrete Containing Fly Ash," *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete*, SP-114, V. 1, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, Mich., 1989, pp. 349-372.

4.7. Rosenberg, A., and Hanson, C. M., "Mechanisms of Corrosion of Steel in Concrete," *Materials Science in Concrete I*, American Ceramic Society, Westerville, Ohio, 1989, p. 285.

4.8. Berry, E. E., and Malhotra, V. M., *Fly Ash in Concrete*, CANMET, Ottawa, 1985.

4.9. Li, S., and Roy, D. M., "Investigation of Relations between Porosity, Pore Structure and CL Diffusion of Fly Ash and Blended Cement Pastes," *Cement and Concrete Research*, V. 16, No. 5, Sept. 1986, pp. 749-759.

4.10. ACI Committee 201, "Guide to Durable Concrete (ACI 201.2R-92)," *ACI Materials Journal*, V. 88, No. 5, Sept.-Oct. 1991, 38 pp. Also, *ACI Manual Of Concrete Practice*, Part 1.

4.11. ACI Committee 222, "Corrosion of Metals in Concrete (ACI 222R-89)," American Concrete Institute, Farmington Hills, Mich., 1989, 30 pp. Also, *ACI Manual of Concrete Practice*, Part 1.

4.12. ASTM C 1218.

4.13. Ozyildirim, C., and Halstead, W., "Resistance to Chloride Ion Penetration of Concretes Containing Fly Ash, Silica Fume, or Slag," *Permeability of Concrete*, SP-108, D. Whiting and A. Walitt, eds., American Concrete Institute, Farmington Hills, Mich., 1988, pp. 35-61.

4.14. AASHTO T 277-83, "Rapid Determination of the Chloride Permeability of Concrete," American Association of State Highway and Transportation Officials, Washington, D.C.

4.15. FHWA-RD-77-85, "Sampling and Testing for Chloride Ion in Concrete," Federal Highway Administration, Washington, D.C.

CHAPTER R5—CONCRETE QUALITY, MIXING, AND PLACING

The commentary on ACI 318 is applicable to this chapter except as described below. Certain sections of the Commentary in ACI 318R-95, Chapter 5, have been repeated in this Commentary chapter to maintain correspondence with footnote numbers in the References section.

R5.0—Notation
The splitting tensile strength requirements have been eliminated since they only apply to lightweight aggregates, which also have been eliminated.

R5.1—General
R5.1.3 Concrete members 24 in. or more thick will retain sufficient moisture throughout the first 12 months to assure continuous curing and hydration of the major portion of the cross section. These large members rarely receive full service loads for many months and, therefore, the test age designated for the determination of compliance with the specified strength may be later than the usual 28 days. The use of such later age strength requirements may permit the use of a lower cement content and, therefore, help limit or control temperature rise due to hydration and the danger of cracking that may occur as these large members cool to ambient temperature levels. The need to control early temperature rise increases in proportion to the minimum thickness of the section.

R5.1.4 Splitting tensile strength tests are associated with lightweight aggregate concrete of ACI 318-95, which has been omitted from discussion in this Code and commentary.

R5.1.5 See R1.2.1.

R5.4—Proportioning by water-cementitious materials ratio
Figure R5.3 is a flow chart outlining the mixture selection and documentation procedure.

R5.4.1 Proportioning mixtures by water-cementitious materials ratio is deleted as an option because the size of the concrete operation warrants trial mixture investigation, and the resulting higher cement contents with use of the table is inconsistent with the need to control hydration heat in these structures.

R5.6—Evaluation and acceptance of concrete
R5.6.1 The frequency of testing has little influence on the accuracy of calculating standard deviations after 25 or more tests of one mixture have been performed on a given class of concrete. It, therefore, has little effect on the level of concrete production required to assure strength requirements of Section 5.3.3. When the calculated standard deviation for 30 tests indicates better than normal control, then the frequency of tests has even less effect on the predictability of the strengths. The reduced testing frequency is, therefore, given to reward good control and lessen the requirements where lower concrete strength allowables are used to reduce hydration heat in the thick concrete members. In such areas, concrete strength generally has little effect on design strength.

R5.6.3.1 Field-cured cylinders do not represent the in-place strength of typical concrete members used in nuclear
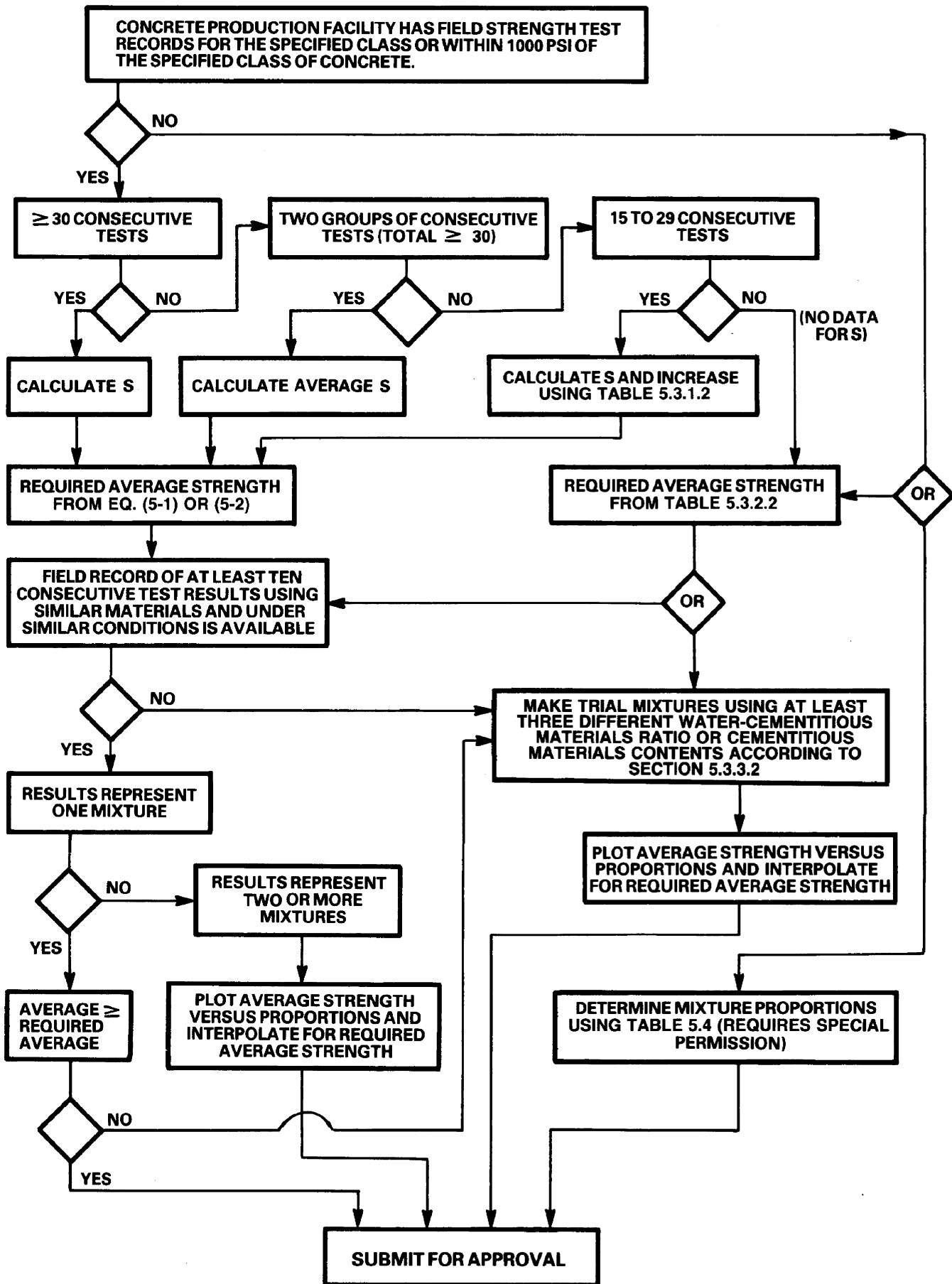
```
┌─────────────────────────────────────────────────────┐
│ CONCRETE PRODUCTION FACILITY HAS FIELD STRENGTH TEST │
│ RECORDS FOR THE SPECIFIED CLASS OR WITHIN 1000 PSI OF│
│ THE SPECIFIED CLASS OF CONCRETE.                     │
└─────────────────────────────────────────────────────┘
```

NO

YES

```
┌──────────────────┐   ┌──────────────────────┐   ┌──────────────────┐
│ ≥ 30 CONSECUTIVE │   │ TWO GROUPS OF CONSECUTIVE│ │ 15 TO 29 CONSECUTIVE│
│ TESTS            │   │ TESTS (TOTAL ≥ 30)   │   │ TESTS            │
└──────────────────┘   └──────────────────────┘   └──────────────────┘
```

YES NO   YES NO   YES NO
(NO DATA FOR S)

```
┌──────────────┐   ┌─────────────────────┐   ┌────────────────────────────┐
│ CALCULATE S  │   │ CALCULATE AVERAGE S │   │ CALCULATE S AND INCREASE   │
│              │   │                     │   │ USING TABLE 5.3.1.2        │
└──────────────┘   └─────────────────────┘   └────────────────────────────┘
```

```
┌──────────────────────────────┐   ┌──────────────────────────────┐
│ REQUIRED AVERAGE STRENGTH     │   │ REQUIRED AVERAGE STRENGTH     │
│ FROM EQ. (5-1) OR (5-2)       │   │ FROM TABLE 5.3.2.2            │
└──────────────────────────────┘   └──────────────────────────────┘
```
OR

```
┌──────────────────────────────────┐
│ FIELD RECORD OF AT LEAST TEN      │
│ CONSECUTIVE TEST RESULTS USING    │
│ SIMILAR MATERIALS AND UNDER       │
│ SIMILAR CONDITIONS IS AVAILABLE   │
└──────────────────────────────────┘
```
OR

NO

YES

```
┌──────────────────────────┐
│ RESULTS REPRESENT         │
│ ONE MIXTURE              │
└──────────────────────────┘
```

NO

```
┌──────────────────────────┐
│ RESULTS REPRESENT         │
│ TWO OR MORE               │
│ MIXTURES                  │
└──────────────────────────┘
```

YES

```
┌──────────────────────────────────────────────┐
│ MAKE TRIAL MIXTURES USING AT LEAST            │
│ THREE DIFFERENT WATER-CEMENTITIOUS            │
│ MATERIALS RATIO OR CEMENTITIOUS               │
│ MATERIALS CONTENTS ACCORDING TO               │
│ SECTION 5.3.3.2                               │
└──────────────────────────────────────────────┘
```

```
┌──────────────────────────────────┐
│ PLOT AVERAGE STRENGTH VERSUS      │
│ PROPORTIONS AND INTERPOLATE       │
│ FOR REQUIRED AVERAGE STRENGTH     │
└──────────────────────────────────┘
```

```
┌──────────────┐   ┌──────────────────────────┐
│ AVERAGE ≥    │   │ PLOT AVERAGE STRENGTH     │
│ REQUIRED     │   │ VERSUS PROPORTIONS AND    │
│ AVERAGE      │   │ INTERPOLATE FOR REQUIRED  │
│              │   │ AVERAGE STRENGTH          │
└──────────────┘   └──────────────────────────┘
```

```
┌──────────────────────────────────┐
│ DETERMINE MIXTURE PROPORTIONS     │
│ USING TABLE 5.4 (REQUIRES SPECIAL │
│ PERMISSION)                       │
└──────────────────────────────────┘
```

NO

YES

```
┌──────────────────────────┐
│ SUBMIT FOR APPROVAL      │
└──────────────────────────┘
```

Fig. R5.3—Flowchart for selection and documentation of concrete proportions.

plant construction, particularly during the first 24 h following placement, because of the effects of hydration heat. The in-place strengths may be several times that of the field-cured cylinders, particularly during cold weather placements. Nondestructive testing, using methods such as sonic methods or other similar techniques, may be used by the engineer to better understand the in-place strength and concrete quality in the structure.

R5.6.4—*Investigation of low-strength test results*

Instructions are provided concerning the procedure to be followed when strength tests have failed to meet the specified acceptance criteria. For obvious reasons, these instructions cannot be dogmatic. The engineer must apply judgment as to the true significance of low test results and whether they indicate need for concern. If further investigation is deemed necessary, such investigation may include nondestructive tests, or in extreme cases, strength tests of cores taken from the structure.

Nondestructive tests of the concrete in place, such as by probe penetration, impact hammer, ultrasonic pulse velocity, or pullout may be useful in determining whether or not a portion of the structure actually contains low-strength concrete. Such tests are of value primarily for comparisons within the same job rather than as quantitative measures of strength. For cores, if required, conservatively safe acceptance criteria are provided that should assure structural adequacy for virtually any type of construction.[5.4-5.7] Lower strength may, of course, be tolerated under many circumstances, but this again becomes a matter of judgment on the part of the engineer. When the core tests fail to provide assurance of structural adequacy, it may be practical, particularly in the case of floor or roof systems, for the engineer to require a load test (Chapter 20). Before load testing, if time and conditions permit, an effort may be made to improve the strength of the concrete in place by supplemental wet curing. Effectiveness of such a treatment must be verified by further strength evaluation using procedures previously discussed.

It should be noted that core tests having an average of 85% of the specified strength are entirely realistic. To expect core tests to be equal to f_c' is not realistic because differences in the size of specimens, conditions of obtaining samples, and procedures for curing do not permit equal values to be obtained.

The Code, as stated, concerns itself with assuring structural safety, and the instructions in 5.6 are aimed at that objective. It is not the function of the Code to assign responsibility for strength deficiencies, whether or not they are such as to require corrective measures.

Under the requirements of this section, cores taken to confirm structural adequacy will usually be taken at ages later than those specified for determination of f_c'.

R5.7—Preparation of equipment and place of deposit

Code Sections 5.7.1(f) and 5.7.1(g) requires free water deeper than 1/4 in. in hollows be removed from the forms. It is not necessary that all water be removed. The requirement for specifying the method for cleaning joints in construction

specifications was added to ensure that only an approved method or methods of joint cleanup would be used.

R5.9—Conveying

R5.9.3 Conveyance of concrete by pumping through aluminum pipe is not permitted because hydrogen gas bubbles are produced by the reaction of aluminum abraded from the pipe with the alkalis in the concrete. These gas bubbles are retained in the hardened concrete and reduce concrete strength.[5.1, 5.2]

R5.10—Depositing

R5.10.4 The wording has been changed from ACI 318 to be more specific and to exclude the use of retempered concrete.

R5.10.9 Section 5.4.4 of the 1971 Building Code contained the requirement that "where conditions make consolidation difficult or where reinforcement is congested, batches of mortar containing the same proportions of cement, sand, and water as used in the concrete, shall first be deposited in the forms to a depth of at least 1 in." The requirement has been deleted from the 1977 Code because the conditions for which it was applicable could not be defined precisely enough to justify its inclusion as a code requirement. The practice, however, has merit and is retained since the 1980 edition of ACI 349. The use of reproportioned batches will aid in preventing honeycomb and poor bonding of the concrete with the reinforcement. The reproportioned concrete or mortar should be placed immediately before depositing the concrete containing larger aggregate and must be plastic (neither stiff nor fluid) when the concrete is placed.

R5.11—Curing

R5.11.4 In many areas, protective coatings are required, and no curing procedure should be used that may be incompatible with those coatings.

R5.11.5 The method or methods of curing should not be left to the discretion of the contractor or to arbitration. Acceptable methods should be clearly stated in the construction specifications.

R5.12—Cold weather requirements

Recommendations for cold weather concreting are given in detail in "Cold Weather Concreting" reported by ACI Committee 306.[5.3] (Present requirements and methods for producing satisfactory concrete during cold weather.)

R5.13—Hot weather requirements

The method or methods of curing should not be left to the discretion of the contractor or to arbitration. Acceptable methods should be clearly stated in the construction specifications.

The designer should also consult ACI Committee 207 report "Effect of Restraint, Volume Change, and Reinforcement of Cracking of Massive Concrete"[5.5] for additional information on crack control.

References

5.1. Newlon, H., Jr., and Ozol, A., "Delayed Expansion of Concrete Delivered by Pumping Through Aluminum Pipe Line," *Concrete Case Study* No. 20, Virginia Highway Research Council, Oct. 1969, 39 pp.

5.2. Fowler, E. E., and Holmgren, E. F., "Expansion of Concrete Pumped Through Aluminum Pipeline," ACI JOURNAL, *Proceedings* V. 68, Dec. 1971, pp. 950-953; and Discussion, ACI JOURNAL, *Proceedings* V. 69, June 1972, pp. 357-360.

5.3. ACI Committee 306, "Cold Weather Concreting (ACI 306R-88)," American Concrete Institute, Farmington Hills, Mich. Also, *ACI Manual of Concrete Practice*, Part II.

5.4. ACI Committee 305, "Hot Weather Concreting (ACI 305R-89)," American Concrete Institute, Farmington Hills, Mich. Also, *ACI Manual of Concrete Practice*, Part II.

5.5. ACI Committee 207, "Effect of Restraint, Volume Change and Reinforcement on Cracking of Massive Concrete (ACI 207.2R-73) (Reapproved 1986)," ACI JOURNAL, *Proceedings* V. 70, Jul. 1973, pp. 445-470. Also *ACI Manual of Concrete Practice*.

CHAPTER R6—FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION JOINTS

The commentary on ACI 318 is applicable to this chapter except as described below:

R6.1—Design of formwork

R6.1.7 The use of steel liners as formwork requires considerations in addition to tolerances.

R6.1.8 and R6.2.5 Form release agents and surface effects of wood type used in the formwork must be compatible with coating systems to assure a durable coating system.

R6.2—Removal of forms, shores, and reshoring

Provisions of ACI 318R apply, except replace the term "building official" with "engineer" whenever it occurs.

R6.3—Conduits and pipes embedded in concrete

R6.3.8 The testing requirements were changed to ensure that there would be no conflict in code applications for the many different piping systems used in nuclear plant construction.

R6.3.9 If an entire pipe system cannot be tested as a unit, a minimum concrete strength of 1000 psi is required for concrete encasing a previously tested length of pipe that has to be filled with liquid, gas, or vapor for testing an added length of the pipe in accordance with Section 6.3.8.

R6.3.13 Piping systems that are embedded in concrete will be inaccessible for normal maintenance. Certain mechanical connections, such as a screwed pipe connection, may not be as reliable as a sealed welded pipe connection and are therefore prohibited from use.

R6.3.14 The design limits specified for concrete temperatures are those in general use in nuclear structures. See, for example, paragraph CC-3440 of ACI 359. Provision is made for the use of higher temperatures provided the effects of higher temperatures on concrete strengths have been established by tests and these effects are incorporated into the design.

R6.4—Construction joints

R6.4.3 Indiscriminate location of construction joints should not be allowed in these structures. The engineer is responsible for all construction joint locations.

CHAPTER 7—DETAILS OF REINFORCEMENT

The commentary on ACI 318 is applicable to this chapter except as described below.

R7.4—Surface conditions of reinforcement

R7.4.1 For an explanation of the service conditions (i.e. temperature and radiation) under which an epoxy coating

may be qualified, see the Section R3.5.3.8 of the commentary in this Code.

R7.4.3 All tendons must be protected against deleterious corrosion from the time they are manufactured to the time they are incorporated in the work (that is, during storage). In the event the tendons are to be grouted (bonded) and coatings or rust inhibitors are used to provide protection against deleterious corrosion, complete removal of the coating or rust inhibitor should be required unless it can be demonstrated that such coatings or inhibitors do not affect the required bond values.

R7.12—Minimum reinforcement

This section has been expanded to include minimum reinforcement for all exposed concrete surfaces. For the purpose of Section 7.12 and as defined in Section 7.12.1, an exposed concrete surface is any concrete surface that is not cast against existing concrete or against rock.

Minimum reinforcement is required to control cracking and to tie the structure together to assure its acting as assumed in the design. Minimum reinforcement is required at each surface in approximately perpendicular directions and the spacing is limited to 18 in. to assist in controlling surface cracking. The section has been expanded by inclusion of the requirements for walls and shells previously identified in Chapters 10, 14, and 19. It also includes requirements for sections having a thickness of 48 in. or greater based on the recommendations made by ACI Committee 207.[7.1]

R7.12.2 and R7.12.3 If the concrete sections referred to in 7.12.2 and 7.12.3 are classifiable as beams or structural slabs, the minimum reinforcement provisions of Sections 10.5.1 and 10.5.2 (beams), Section 10.5.3 (one-way structural slabs), and Section 13.4.1 (two-way structural slabs) apply in addition to the provisions of Sections 7.12.2 and 7.12.3. Minimum reinforcement required for beams is given directly in Section 10.5.1; whereas that required for structural slabs is given in Section 7.12.5 by way of reference from Section 10.5.3 and Section 13.4.1.

R7.12.3 The reinforcement stress f_s in Section 7.12.3 does not represent the same effects as that in Section 10.6. Section 7.12.3 applies only to massive concrete sections. The quantity $A_{s,min}$ is the minimum amount of reinforcement required to limit the widths of surface shrinkage cracks in these sections. This can be achieved by keeping the stresses in the reinforcement after the concrete cracks well below yield. A value of $0.60 f_y$ was selected for this purpose.

R7.12.4 The application of massive concrete is common in design and construction of concrete nuclear safety structures. The requirements for minimum reinforcement and physical limitation of massive concrete, however, have not been clear to the designers. ACI Committee 207 7.2 has studied and developed much useful information and data in regard to massive concrete structures. It is highly recommended that the designer obtain and study the specific reports by ACI Committee 207 for detailed and up-to-date information.

R7.13—Requirements for structural integrity

The integrity of the overall structure is not affected, even if local strengths are exceeded, provided there is no loss of intended function of any safety-related systems as stated in 9.2.6.

References

7.1. ACI Committee 207, "Effect or Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete (ACI 207.2R-73)," ACI JOURNAL, *Proceedings* V. 70, July 1970, pp. 445-470. Also *ACI Manual of Concrete Practice*.

7.2. ACI Committee 207, "Mass Concrete for Dams and other Massive Structures," ACI JOURNAL, *Proceedings* V. 67, No. 4, Apr. 1973, pp. 273-309.

CHAPTER R8—ANALYSIS AND DESIGN: GENERAL CONSIDERATIONS

R8.0—General

Chapter 8 parallels the general considerations for analysis and design presented in Chapter 8 of ACI 318. Some modifications have been made that reflect particular requirements applicable to concrete nuclear safety structures. Reference to the alternative design method and to the use of lightweight concrete and permanent fillers has not been made in this standard. In addition, the load requirements have been appropriately altered. Chapter 8 of the commentary on ACI 318 should be referenced for concrete nuclear safety structures except as noted herein.

R8.1—Design methods

The strength design method of ACI 318 has been referred to exclusively, deleting all reference to the alternate design method of Section 8.1.2 of ACI 318.

R8.2—Loading

The commentary on ACI 318 is not applicable for concrete nuclear safety structures. Commentary regarding design loads and load combinations is presented in Chapter 9 of this document.

R8.3—Method of analysis

Reference to the alternative design method in the commentary on ACI 318 is not applicable.

R8.3.3(e) This requirement has been added to indicate that approximate analysis coefficients are not appropriate for members with haunches and varying cross sections.

R8.5—Modulus of elasticity

Reference to lightweight concrete in ACI 318 has been deleted.

R8.6—Stiffness

Section 8.6.1 differs from ACI 318 in order to be more specific.

R8.11—Joist construction

Reference to permanent burned clay or concrete tile fillers in ACI 318 has been deleted.

CHAPTER R9—STRENGTH AND SERVICEABILITY REQUIREMENTS

Chapter 9 parallels the requirements for strength and serviceability presented in Chapter 9 of ACI 318. The general requirements and the section on required strength have been completely revised to reflect the requirements regarding loads and load combinations applicable to concrete nuclear safety structures. The maximum specified yield strength of non-prestressed reinforcement f_y has been limited to 60,000 psi. Deflection limitations have been revised. In addition, requirements for the use of lightweight concrete in ACI 318 do not appear in this standard. Chapter 9 of the commentary on

ACI 318 should be referenced for concrete nuclear safety related structures except as noted herein.

R9.0—Notation

In the notation for D, E_o, E_{ss}, R_a, and R_o, "equipment" is to include all items that are permanently attached to the concrete structure, such as cable trays and conduits.

R9.1—General

Even though Section 9.1 of ACI 318 has been completely revised, the commentary on ACI 318 for this section is generally applicable. The loads appropriate for nuclear safety structures are defined in Section 9.0 and categorized in Section 9.1 as follows: normal loads, severe environmental loads, extreme environmental loads, and abnormal loads.

Piping and equipment reactions to be included as R_o are those reactions produced by normal operating temperatures acting on the piping system or equipment; piping reactions generated by normal operation flow transients; and any other reactions occurring during normal operation or shutdown. Note that dead load and earthquake reactions are not included in R_o.

When considering concentrated impactive loads such as those associated with postulated rupture of high energy pipes or missiles, local section strength capacities may be exceeded. Under these concentrated loads, elastoplastic behavior may be assumed with appropriate ductility ratios provided resulting deformation will not result in loss of function of any safety related system.

Live loads associated with elevators, machinery, and craneways should be increased to allow for impact in accordance with the recommendations of ASCE 7.

The discharge of safety relief valves into a suppression pool generates loads that are unique to BWR power plant structures. Specific classification of these loads is not given by the Code at this time due to ongoing efforts by the industry to quantify them.

R9.2—Required strength

Section 9.2 of ACI 318 and the commentary on this section have been completely revised for concrete nuclear safety related structures. The load combinations and load factors of this section reflect consideration of the likelihood of individual and combined-event occurrences as well as possible excess load effects such as variations in loads, assumptions in the structural analysis, and simplifications in the calculations.

Load Combinations 1, 2, and 3 involve normal loads and normal loads in combination with severe environmental loads. Similar to ACI 318, this standard uses load factors of 1.4 and 1.7 for dead and live loads, respectively, in these load combinations. In addition, a load factor of 1.4 was assigned to lateral and vertical liquid pressure, and a load factor of 1.7 was assigned to normal-operation pipe reactions, lateral earth pressure, and the operating basis wind loads. Because the plant could remain operational when subjected to the effects of severe environmental loads such as the operating basis earthquake or operating basis wind, these loads are treated the same as other operating loads. The load factor on the earthquake is 1.7 rather than 1.87 required by ACI 318. ACI 318 assumes a design method similar to that of the Uniform Building Code.

Nuclear safety related concrete structures are designed to significantly more conservative seismic criteria, and the resulting loads are less likely to be exceeded. Hence, the additional factor of 1.1 is considered unnecessary.

Unit load factors are used in Load Combinations 4 and 5 involving extreme environmental loads. This recognizes that the loads caused by the safe shutdown earthquake or the design basis tornado are extreme and are of very low probability.

Load Combinations 6, 7, and 8 are directed towards abnormal loads in combination with normal, severe, and extreme environmental loads, respectively. Abnormal loads are generated by a postulated high-energy, pipe-break accident. This accident could generate differential pressures, thermal loads, pipe and equipment reactions on supports, pipe rupture reaction forces, jet impingement loads, and missile impact effects. Load Combination 8 has unit load factors on all loads because it represents an extremely unlikely combination of events. Load Combination 6 has a load factor of 1.25 on the pressure because, although this is a very unlikely event, it affects a larger portion of the overall structure than the local conditions included in Combination 8. Load Combination 7 is intermediate between 6 and 8 and is included because it has been specified historically. In Combinations 7 and 8, unit load factor is specified for the concentrated effects of pipe rupture. These events are less probable than a differential pressure loading and produce only localized effects.

The load factors for E_o and P_a in Combinations 6 and 7 are lower than the corresponding factors in ACI 359. The reasons for this difference are:

(a) The structures do not function as leakage barriers to contain the effects of the loss of coolant accident;

(b) The pressure loading is a one-time-accident loading. For the corresponding one time loadings of both the safe shutdown earthquake and the tornado the load factor is unity; and

(c) The design pressure is larger than the calculated pressure so that a margin of safety is included therein.

The last three Load Combinations, 9, 10, and 11, are similar to Load Combinations 1, 2, and 3. These Combinations, however, consider normal-operation thermal loads, allowing a reduction of 25% in the required strength. This reduction is in recognition of the fact that such thermal loads tend to be self-relieving. A 25% reduction in required strength for load combinations involving thermal loads is also used in ACI 318.

It should be noted that, in applying the load combinations given in Section 9.2.1 of the Code, due regard should be given to sign because the standard direction of loads may on occasion be of opposing direction, thus producing tensile axial loads, negative reactions, or reverse bending. The provisions of Section 9.2.2 consider the effect of differential settlement, creep, and shrinkage where necessary.

When impact due to operating loads is present, impact effects should be considered and impact loads, if any, included with live loads in the various equations for required strength. As the effects of pipe ruptures P_a, T_a, R_a, Y_r, Y_j, and Y_m, may not necessarily occur simultaneously, Section 9.2.5 of the Code permits a time-history analysis to account for the time lag between these loads.

R9.2.7 Apart from the extreme environmental loads generated by the safe shutdown earthquake and by the design basis

tornado, other extreme environmental loads may also be required for the plant design. Examples of such loads are those induced by flood, aircraft impact, or an accidental explosion.

These environmental loads should be treated individually in a manner similar to the loads generated by the design basis tornado in determining the required strength according to the equations in Section 9.2.1. Abnormal loads are not considered concurrently with the above extreme environmental loads.

R9.5—Control of deflections

This section contains different deflection requirements than ACI 318. The deflection requirements in ACI 318 are based on comfort levels for human occupancy, cracks in ceiling, and other considerations applicable to residential buildings. The Building Code also states that if nonstructural elements (such as equipment) are attached to the structure, the deflection requirements are to be satisfied by calculation and it is not sufficient just to use the given l/d ratios.

Almost all nuclear safety related concrete structures support nonstructural elements. In addition, because the member sizes used in ACI 349 structures are generally larger than those used in common building structures, it was felt that it is too restrictive to require deflection calculations for all these structural elements.

Table R9.5(a) indicates the deflections that could be expected if no calculations are performed.

Table R9.5(a)—Maximum deflections for unfactored loads

Type	Loading equation number (Section 9.2.1)	Beams	Slabs*
1	Eq. (1), (2), and (3)	$l/400$	$l/320$
2	Eq. (4) and (5)	$l/250$	$l/200$

*For two-way construction l shall be replaced by l_s

Consistent with ACI 349 loading combinations, the table indicates anticipated deflection values for normal, severe environmental, and extreme environmental loading combinations. The Code then states that deflection combinations are to be made only if the requirements given by the manufacturers of the nonstructural elements are more stringent than the anticipated deflections given in Table R9.5(a).

For normal and severe environmental loading conditions, the anticipated deflections are selected to be l/400 for beams and ribbed one-way slabs and l/320 for two-way slabs and solid one-way slabs. The Building Code also makes a distinction between the appropriate deflection limits for these two groups of structural elements. A more liberal or a more stringent criteria for the anticipated deflection can be chosen, however, the l/d ratio has to be modified accordingly. The deflection limits in the Code will provide a reasonable l/d ratio and at the same time will not require deflection calculations for a large number of structural elements.

The l/d ratios given in the Code were established from:

1. Recommendations by Committee 435, Deflection of Concrete Building Structures; and

2. A review of the minimum member sizes commonly used in ACI 349 structures.

Based on these l/d ratios, the anticipated deflections were computed and given in the table.

In these computations, the following assumptions were made:

1. The stress in the reinforcing steel is $0.6f_y$, for unfactored service load conditions; and

2. The immediate deflections are multiplied by a factor of 0.5 and 1.5 to obtain the additional long-term deflections due to structural dead load and equipment dead load, respectively. This assumes that the equipment would be placed at least a year after the structure has been placed. Therefore part of the long-term deflection due to structural dead load has already occurred at the time the equipment is placed.

The Code defines the deflections that need to be considered. Immediate deflection is no longer a consideration. Because the minimum l/d ratios are more stringent than ACI 318, the immediate deflection limits of ACI 318 will be satisfied.

The considerations for camber in prestressed concrete members is included in the Code.

A deflection requirement for walls is also included. Because the walls in ACI 349 structures frequently carry axial load and flexure, the walls should satisfy the requirements given for nonprestressed one-way or two-way or composite or prestressed construction, as applicable.

Similar assumptions were made to establish the Building Code deflection requirements. To suit ACI 349 applications, however, slightly different numbers were used. To simplify the deflection computations, a deflection reduction factor **g** and moment of inertia modification factor I_g / I_e have been included. These factors can be used to modify the results of the factored load analysis performed for the strength design of the structure. Reference to ASCE 7 strength reduction factors and load combinations is not applicable to safety-related concrete structures. Thus, a separate analysis for deflection computation is no longer implied.

CHAPTER R10—FLEXURE AND AXIAL LOADS

Chapter 10 is identical to that of ACI 318 except as described below for Sections 10.6.

R10.6—Distribution of flexural reinforcement in beams and one-way slabs

All subsections of Section 10.6 are identical to those of ACI 318 except 10.6.4.

In lieu of stress calculations for f_s, $0.40f_y$ may be used. This is less than the value of $0.60f_y$ specified by ACI 318. This difference exists because, unlike ACI 318 structures, the reinforcement design for nuclear safety structures is controlled by load combinations other than those corresponding to a service condition. Consequently, at service loads, the reinforcement stresses for nuclear safety structures are not likely to be as high as those for ACI 318 structures.

CHAPTER R11—SHEAR AND TORSION

The commentary in ACI 318 is applicable to this chapter except as noted herein.

R11.12—Special provisions for slabs and footings

R11.12.2.3 Subsection 11.12.2.3 is added to this section. In concrete nuclear safety structures, slabs or walls may be subject to loads producing punching shear forces that have to be considered in combination with forces in the plane of the slab or wall. The forces in the plane of the
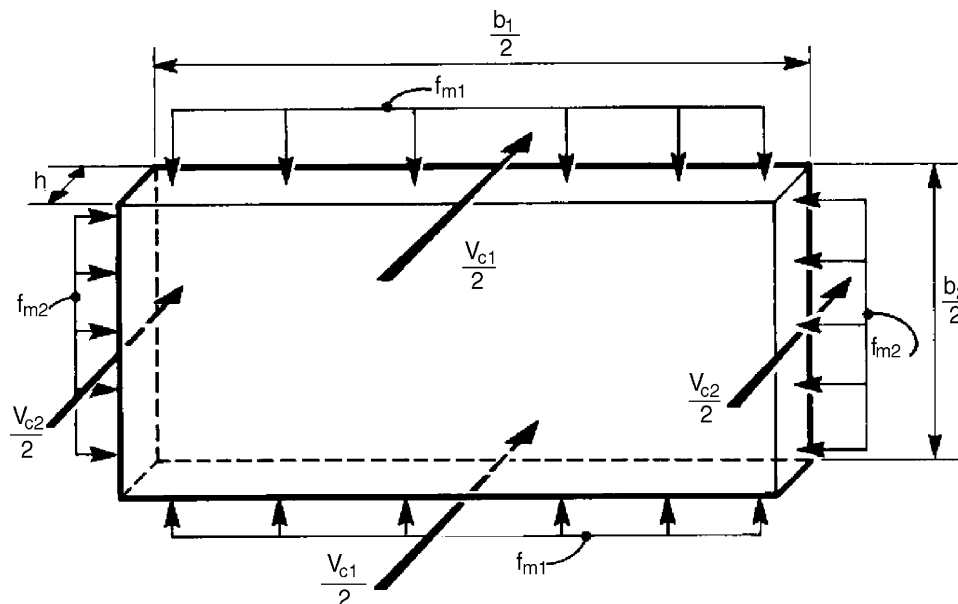
—Concrete shear strengths over perimeters b_1 and b_2 are V_{c1} and V_{c2}, respectively.
—Total concrete shears strength, $V_c = V_{c1} + V_{c2}$.
—f_{m1} and f_{m2} are negative as shown

Fig. R11.1—Concrete shear strength in presence of membrane tensile stresses

slab or wall could be dynamic (seismic) or static (shear due to wind, tornado, equipment, etc.). The concrete shear strength for this condition is given in Subsection 11.12.2.3, and the expressions for V_{c1} and V_{c2} are based on punching shear tests of biaxially tensioned wall elements, reported in References 11.1 and 11.2. These tests indicated that biaxial tension levels up to nearly 0.9 f_y produced only minor decreases in the punching shear strength for elements without any shear reinforcement. For membrane tension stress ratio $f_{m1}/r\varsigma_1 f_y$ or $f_{m2}/r\varsigma_2 f_y$ exceeding 0.9, the corresponding components of shear strength are taken as 0.5 $\sqrt{f_c\varsigma} \, b\varsigma_1 \, h$, and 0.5 $\sqrt{f_c\varsigma} \, b\varsigma_2 \, h$, which are assumed lower bounds for very high levels of membrane tension. For membrane compressive stresses, the provisions of Section 11.12.2.2 are applicable.

Even if no shear reinforcement is required, the provisions in Sections 11.12.2.3.3 and 11.12.2.3.4 apply when the membrane tensile stresses f_{m1} or f_{m2} exceed 0.6 $r\varsigma_1 f_y$ or 0.6 $r\varsigma_2 f_y$, respectively. The membrane reinforcement is required to be increased to resist the punching shear force V_u as if it were an additional membrane tension force. These provisions are a conservative application of the test results (Reference 11.1) that indicated a slight increase in the forces in the membrane reinforcement caused by the shear force, at higher levels of applied membrane tension.

References

11.1. Jan, W. C.; White, R. N.; and Gergely, P., *Behavior of Reinforced Concrete Slabs Subjected to Combined Punching Shear and Biaxial Tension*, NUREG/CR-2920, Sept. 1982.

11.2. Jan, W. C.; White, R. N.; and Gergely, P.; "Peripheral Shear Strength of Biaxially Tensioned Reinforced Concrete Wall Elements," *Nuclear Engineering and Design*, V. 69, No. 2, p. 271.

CHAPTER R12—DEVELOPMENT OF REINFORCEMENT

The Commentary of ACI 318-95 is applicable to this chapter except that references to lightweight concrete are to be deleted except as described below:

R12.6—Mechanical anchorage

The provisions of paragraphs 12.6.2 and 12.6.3 of ACI 318-95 have been replaced in ACI 349 by a reference to Appendix B that provides minimum requirements for anchorage of steel embedments. Mechanical anchorage should generally be designed to develop the full capacity of the reinforcement. Combinations of mechanical anchorage plus additional embedment length of reinforcement, as permitted in paragraph 12.6.3 of ACI 318, should only be used if substantiated by experimental or detailed analytical investigation as permitted by paragraph B.1.3.

R12.14—Splices of reinforcement: General

R12.14.3.4.1 Mechanical connections for reinforcing bars can be made by various commercially available mechanical connections, as described in Reference 12.1. These connections include the following: (1) providing a sleeve around reinforcing bars to be connected and filling it with grout or ferrous filling metal; (2) taper threaded sleeves; (3) swaging the sleeves (hot or cold) around reinforcing bars;

and (4) threaded sleeves over thread-deformed reinforcing base. Each of the methods of mechanical connections may have its limiting conditions, under which it would perform satisfactorily. The range of variabilities to be considered in subparagraph (a) of the Code may be satisfied by reviewing large samples (min. of 100) of conformance tests on mechanical connections constructed under actual field conditions (or under simulated field conditions) considering variations in weather, materials, installation crews, or other construction variables that may effect the performance of a given type of mechanical connection. Mechanical connections may also be qualified to the requirements of paragraph 12.14.3.4 by a test program performed in accordance with Section CC-4333.2 of ACI 359. Previously performed qualification test results may be used if the test variables reflect the conditions of the particular project.

R12.14.3.7 Splices of lower modulus than that of the reinforcing steel may reduce the ultimate moment capacity of the member at the splice location and will contribute significantly to wider-than-normal cracks at the splice location. For members subject to tensile forces, this increase in crack width may result in loss of shear strength.

Reference

12.1. ACI Committee 439, "Mechanical Connections of Reinforcing Bars (ACI 439.3)."

CHAPTERS R13 THROUGH R17

There are only minor differences between ACI 318 and the standard for:

Chapter 13—Two-Way Slab Systems;

Chapter 14—Walls;

Chapter 15—Footings;

Chapter 16—Precast Concrete; and

Chapter 17—Composite Concrete Flexural Members.

The changes are given below.

R14.3 The minimum reinforcement requirements of Section 7.12 are more applicable for the thick concrete nuclear safety structures than the corresponding requirements of ACI 318. See Commentary Section 7.12 for discussion.

R15.8.1.1 and R17.5.2 Reference to the Alternate Design Method in ACI 318 Commentary is not applicable for the ACI 349 Commentary.

CHAPTER R18—PRESTRESSED CONCRETE

The commentary on ACI 318 is applicable to this chapter. Section 18.1.4 is added to define service load conditions to include both normal mechanical and severe environmental loads.

Section 18.4.2(a) permits a higher extreme fiber compressive stress if T_o is included.

CHAPTER R19—SHELLS

Chapter 19 of ACI 318-95 is intended to provide design provisions for the wide range of shell forms expected to be used for commercial structures. These include thin shells, folded plates, and ribbed shells to name a few. In nuclear safety related concrete structures, the shell forms usually encountered are limited to basic cylinders with dome shapes, having thicknesses not less than 12 in. The design provisions of Chapter 19 of the Code are intended to be specifically ap-

plicable to this type of shell structure. The Commentary to Chapter 19 of ACI 318-95 is applicable to the ACI 349 Code, supplemented by the discussion below.

R19.1—Scope and definitions

R19.1.1 The type of shells used in nuclear safety related concrete structures are typically of a thickness greater than those used in conventional structures, due to the nature of loading and the requirements for shielding. Chapter 19 has, therefore, been written to apply only to shells having a thickness greater than or equal to 12 in. If needed, users can refer to Chapter 19 of ACI 318.

R19.2—General

R19.2.1, 19.2.2, and 19.2.3 Nonlinear analysis may be necessary when a cracked concrete section due to load combinations of thermal, earthquake, and others listed in Section 9.2.1 is considered and the redistribution of stresses takes place in relationship to the depth of cracks. Tensile resistance of the cracked concrete is not relied on. The cracks could occur in meridional, hoop, or other directions depending on the reinforcement patterns and loadings.

The Code does not intend to require the nonlinear crack analysis for all possible cracked conditions, but requires the engineer to review the possibility of the resulting redistribution of forces and initiate analysis if deemed necessary.

R19.2.7 Besides thickening of the concrete, rings made of structural steel with embedded anchors in composite action may also be used around penetrations or openings to satisfy strength requirements.

Both cases of either thickening the concrete or reinforcing the penetration with steel embedment may induce more thermal reactions that need to be considered.

R19.2.8 In addition to comments in Sections 19.2.1 through 19.2.3, the variation in the membrane stresses across the thickness shall be taken into consideration for thick shells.

The variation in stresses across the shell thickness becomes significant for thick shells. Thick shells are generally considered as shells whose thicknesses are more than one-tenth the minimum radius of curvature. The radius may be based on the mean value of the inner and outer shell surfaces.

R19.2.10 For shell structures, analysis must be carried out to support and supplement the results of the model test. This is required to cover the loading conditions presented in Chapter 9 of this Code.

R19.4—Section design and reinforcement requirements

R19.4.2 Due to the large number of different load combinations, it is not considered practicable to place the reinforcement parallel to the line of the principal tensile stress and it is, therefore, required that reinforcement be placed in two or more directions.

R19.4.4 The minimum reinforcement requirements of Section 7.12 are more applicable for the thick concrete nuclear safety structures than the corresponding requirements of ACI 318. See Commentary Section R7.12 for discussion.

R19.4.6 Stirrups or other types of web reinforcing may be used, if required, to tie back curved reinforcement and trans-

fer the forces imposed inside the radius of the curved reinforcement away from the penetrations and thus to prevent cracking around penetrations.

CHAPTER R20—STRENGTH EVALUATION OF EXISTING STRUCTURES

R20.1—Strength evaluation: General

Chapter 20 contains requirements and commentary on the use of strength evaluation methods such as load testing to characterize the strength of an existing nuclear safety related concrete structure. Because of the massive size and complex design requirements for most safety related structures, the use of in-place strength evaluation methods may have limited application. Local load testing of structural components such as anchorages, embedments, and post-tensioned reinforcing steel is practical and has been utilized to establish adequacy. Other techniques that have been used in evaluating the physical condition and performance of existing safety related structures are cited in Reference 20.1. The engineer shall identify the appropriate evaluation program to be followed if the safety or strength of an existing structure is questioned. Chapter 20 of the ACI 349 Code differs from that in ACI 318 because of the size and type of construction of structures in nuclear power plants and the added requirement for preservation of public safety.

R20.2—Analytical investigations: General

In an analytical investigation, the analysis must be based on data gathered concerning the actual dimensions of the structure, the strength of the materials in place and all other pertinent details. The field examination should be thorough. For example, if coring of the concrete is required, sufficient samples should be taken to obtain reliable average strength indications and to detect possible flaws at critical locations. (Typically, core tests provide about 85% of the strength of laboratory-cured cylinders for the same concrete.)

In some cases, the engineer may deem the analytical investigation to be preferable to load testing. In other cases, analytical evaluation may be the only practicable procedure. Certain members, such as columns and walls, may be difficult to load and the interpretation of the load test results equally as difficult unless severe damage or actual collapse occurs.

R20.2.1 The Code states that the analysis shall demonstrate to the engineer's satisfaction that the intent of the Code has been satisfied. The intent of the Code is to ensure public safety. The load factors and strength reduction factors $\mathbf{f}$ provide for possible loads in excess of the specific design loads, complexities involved in the analysis, workmanship variations, materials variations, and similar factors that separately may be within tolerances but that cumulatively might adversely affect the strength of the structure or member. In general, it should be shown that the structure or member will have strength close to or in excess of that envisioned in the original design or as required by the Code. This is a matter of judgment involving consideration of relevant factors such as the possible consequences of collapse.

R20.2.2 For individual elements, amount, size, arrangement, and location must be determined at the critical sections for reinforcement and/or tendons designed to resist applied load. Nondestructive investigation methods are acceptable.

In large structures, determination of these data for approximately 5% of the reinforcement or tendons in critical regions may suffice if these measurements confirm the data provided in the construction drawings.

R20.2.3 The number of tests may depend on the size of the structure and the sensitivity of structural safety to concrete strength for the problem. In cases where the potential problem involves flexure only, investigation of concrete strength can be minimal for a lightly reinforced section ($\mathbf{r}f_y/f_c\mathbf{c} \le 0.15$ for rectangular section).

R20.2.4 The number of tests required depends on the uniformity of the material and is best determined by the engineer for the specific application.

R20.2.5 Strength reduction factors given in 20.2.5 are larger than those specified in Chapter 9. These increased values are justified by the use of accurate field-obtained material properties, actual in-place dimensions, and well-understood methods of analysis. Additional information and criteria that may be used in the analytical investigation of a structure is contained in ACI 349.3R-96 (Reference 20.1). The reference to 1.4 addresses the Regulatory Authority's participation.

R20.3—Load tests: General

The selection of the portion of the structure to be tested, the test procedure, and the interpretation of the results should be done under the direction of a qualified engineer experienced in structural investigations and field tests and measurements. Additional guidance on the performance of load tests is contained in Reference 20.2.

R20.4—Load test procedure

R20.4.1 It is important to apply the load at locations so that its effects on the suspected defect are a maximum and the probability of unloaded members sharing the applied load is a minimum. In cases where it is shown by analysis that adjoining unloaded elements will help carry some of the load, the load must be placed to develop effects consistent with the intent of the load factor.

R20.4.2 The required load intensity follows previous load test practice. The live load should be increased to compensate for resistance provided by unloaded portions of the structure in question. The increase in live load is determined from analysis of the loading conditions in relation to the selected pass/fail criterion for the test.

R20.5—Loading criteria

R20.5.2 It is advisable to inspect the structure after each load increment.

R20.5.3 "Arching" refers to the tendency for the load to be transmitted nonuniformly to the flexural element being tested. For example, if a slab is loaded by a uniform arrangement of bricks with the bricks in contact, "arching" would result in reduction of the load on the slab near the midspan of the slab.

R20.6—Acceptance criteria

R20.6.1 A general acceptance criterion for the behavior of a structure under the test load is that it shall not show "evidence of failure." Evidence of failure will include cracking, spalling, and/or deflection of such magnitude and extent that the observed result is obviously excessive and incompatible with the safety requirements of the structure. No simple rules can be developed for application to all types of structures and conditions. If sufficient damage has occurred that the structure is considered to have failed that test, retesting is not permitted because it is considered that damaged members should not be put into service even at a lower rating.

Local spalling or flaking of the compressed concrete in flexural elements related to casting imperfections need not indicate overall structural distress. Crack widths are good indicators of the state of the structure and should be observed to help determine whether the structure is satisfactory. Exact prediction or measurement of crack widths in reinforced concrete elements, however, is not likely to be achieved under field conditions. It is advisable to establish criteria before the test, relative to the types of cracks anticipated, where the cracks will be measured, how they will be measured, and to establish approximate limits or criteria to evaluate new cracks or limits for the changes in crack width.

R20.6.2 Specified deflection limits and the retest option follow past practice. If the structure shows no evidence of failure, "recovery of deflection" after removal of the test load is used to determine whether the strength of the structure is satisfactory. In the case of a very stiff structure, however, the errors in measurements under field conditions may be of the same order as the actual deflections and recovery. To avoid penalizing a satisfactory structure in such a case, recovery measurements are waived if the maximum deflection is less than $\ell_t^2/(20{,}000h)$. The residual deflection $\mathbf{D}_{rmax}$ is the difference between the initial and final (after load removal) deflections for the load test or the repeat load test.

R20.6.3 Forces are transmitted across a shear crack plane by a combination of aggregate interlock at the interface of the crack, which is enhanced by clamping action of transverse stirrup reinforcing and by dowel action of stirrups crossing the crack. As crack lengths increase to approach a horizontal projected length equal to the depth of the member and concurrently widen to the extent that aggregate interlock cannot occur, and as transverse stirrups if present begin to yield or display loss of anchorage so as to threaten their integrity, the member is assumed to be approaching imminent shear failure.

R20.6.4 The intent of 20.6.4 is to make certain that the professionals in charge of the test will pay attention to the structural implication of observed inclined cracks that may lead to brittle collapse in members without transverse reinforcement.

R20.6.5 Cracking along the axis of the reinforcement in anchorage zones may be related to high stresses associated with the transfer of forces between the reinforcement and the concrete. These cracks may be indicators of pending brittle failure of the element if they are associated with the main reinforcement. It is important that their causes and consequences be evaluated.

References

20.1. ACI Committee 349, "Evaluation of Existing Nuclear Safety-Related Concrete Structures (349.3R-96)," American Concrete Institute, Farmington Hills, Mich., 1996, 20 pp.

20.2. ACI Committee 437, "Strength Evaluation of Existing Concrete Buildings (ACI 437R-91)," American Concrete Institute, Farmington Hills, Mich., 1991, 24 pp. Also, *ACI Manual of Concrete Practice*, Part III.

CHAPTER R21—SPECIAL PROVISIONS FOR SEISMIC DESIGN
R21.2—General requirements

ACI 349's intended scope is the design of safety related structures of commercial nuclear power facilities and other such facilities as may be required by the appropriate governing bodies. The predominant structural form is shear wall and slab construction of general heavy proportions. The safety related structures and structural elements of nuclear power plants are proportioned and reinforced so as to exhibit elastic behavior under all loads (except impulsive and impactive loads) and load combinations including those associated with the design basis SSE. This structural form and the elastic design requirements are known to ensure a high level of confidence in the retention of structural integrity for the design basis SSE loads.

ACI 318 is meant to be used as part of a legally adopted building code, covering all building of the usual types and providing minimum requirements necessary to provide public health and safety. The predominant structural form is moment resisting frame. The structure and structural elements are designed to remain elastic for all loads and load combinations except those associated with strong motion earthquakes. United States building codes permit a seismic design based on loads corresponding to an inelastic response to ground motion. Accordingly, the design must exhibit a robustness characterized by retention of substantial proportion of its strength as it is inelastically cycled. The requisite toughness can only be achieved by requiring special reinforcing steel detailing; hence the special provisions of Chapter 21 of ACI 349.

Recognizing that significant differences in design and structural forms exist in the applications of respective codes, the high seismic risk provisions of ACI 318, Chapter 21 are adopted here for the following reasons. First, the adoption of these provisions for nuclear plants provides additional assurance that structural integrity is maintained in the unlikely event of an earthquake beyond the design basis SSE or other unforeseen circumstances. Second, the adoption of these provisions provides reinforcing bar detailing requirements consistent with the toughness needs of structural elements designed for Special Facilities class of structures of the Department of Energy's non-reactor nuclear production plants wherein limited inelastic response to design basis earthquake is permitted. ACI 349 is cited as the design code by the governing design criteria document of these facilities. Third, and last, adoption of the high seismic risk provisions of ACI 318 Chapter 21 maintains the intended maximum possible compatibility between ACI 349 and ACI 318.

R21.6—Structural walls, diaphragms, and trusses

In a recent paper, the extremely good performance of shearwalls during earthquakes was described. In addition to the seismic capability of shearwalls, the relatively small lateral deflections of shearwalls are more consistent with the requirements that safety related systems must be capable of continued operation during and after seismic events. At a height to depth ratio less than 2, the behavior of the wall is predominantly in shear with insignificant bending deformation. Because bound-ary elements are essentially required to provide adequate deformability and confinement for flexure, shear walls with aspect ratios (h_w/l_w) of less than 2 do not require them.

Reference

21.1 Fintel, M., "Shearwalls—An Answer for Seismic Resistance?" *Concrete International*, V. 13, No. 7, July 1991, pp. 48-53.

APPENDIX A—THERMAL CONSIDERATIONS
RA.1—Scope

Appendix A gives requirements for the design of concrete nuclear safety related structures (other than reactor vessels and containments) subjected to thermal loading.

RA.1.1 Appendix A also divides the temperature variations into two types:

(a) *Position dependent*—Variations through the thickness and along the geometric center of the member; and

(b) *Time dependent*—Transient temperature distribution and the final steady state condition.

These temperature variations are produced by combinations of the following:

(a) *Ambient temperatures*—These are outside atmospheric temperatures that depend upon the location of the plant and vary with meteorological changes;

(b) *Operating temperatures*—The temperatures as obtained in various locations inside the nuclear power plant under normal operating conditions. Under steady state normal operating conditions these temperatures usually produce linear temperature distributions across structural sections; and

(c) *Accident or abnormal temperatures*—These are short duration or transient temperatures, which usually produce nonlinear temperature distributions across structural sections.

Typical nuclear safety related structures subjected to significant ambient, operating, and/or accident temperatures are:

(a) Major shielding members;

(b) External walls and slabs;

(c) Spent fuel pool walls and slabs; and

(d) Pipe supports and restraints.

Any structure in the vicinity of a high energy piping system may be subjected to both a steam environment and a water/steam jet. The jet will usually be of short duration; steam environment will normally also be of short duration when venting to atmosphere occurs. In the case of containment internal structures, the steam environment can be of long duration because it is contained by the containment.

RA.1.2 The procedures discussed in Appendix A and this commentary are valid for all of the temperature conditions described above. The designer, however, should consider the following when using these procedures:

1. Linear thermal strain causes stress only under conditions of restraint and a portion of such stress may be self-relieving. Mechanisms for such relief are: cracking, yielding, relaxation, creep, and other time-dependent deformations; and

2. Accident temperature transients may be of such short duration that the resulting temperature distributions and corresponding stress changes are not significant. Therefore, these temperature transients may not adversely affect the safe shutdown capacity of the plant.

Thus it can be seen that judgment is required when evaluating the effects of accident temperatures because they are dependent on the duration and location of the thermal transient, as well as the performance requirements for the structure.

RA.1.4 Minimum reinforcement as specified in other sections of this Code should be provided even if the calculations, as per this Appendix, indicate lesser amounts of reinforcement. The minimum reinforcement required by Section 7.12 may be used to resist the thermal and other loads.

RA.2—Definitions

The various definitions are illustrated in Fig. RA-1.

Base temperature—This is the temperature at which it is assumed the material is free of thermal stresses. Seventy F can be used as a base temperature for most calculations.
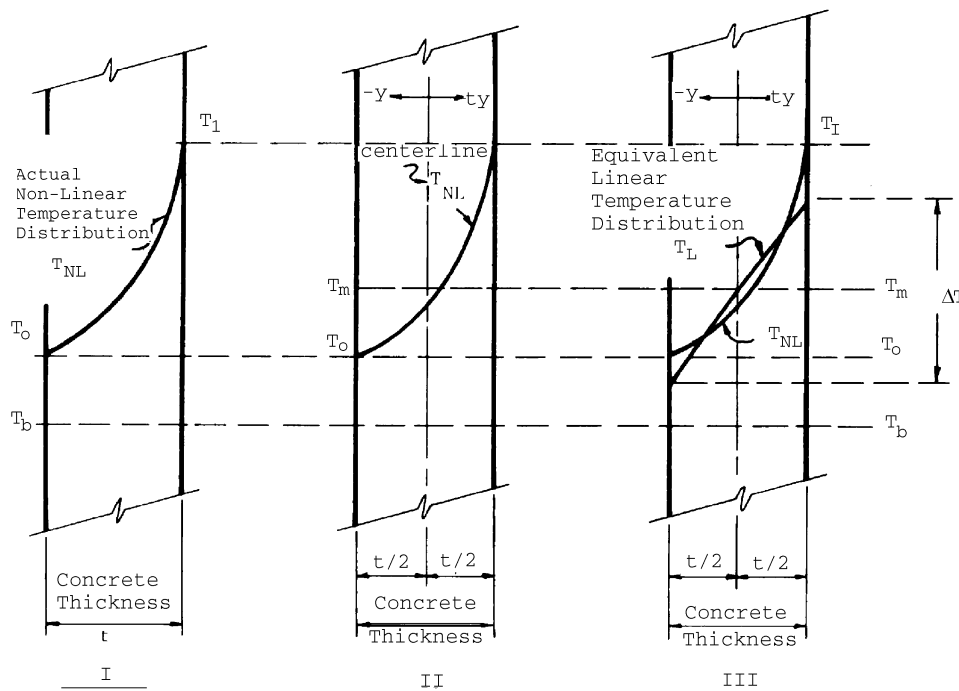
Temperature distribution—The temperature distribution across a section can vary with time as well as varying along the length of the member. For such variations, the engineer should evaluate the effects of temperature distribution at a number of sections and for a number of time durations.

In cases where the temperature distribution on a section is nonlinear and the section is predicted to crack, the nonlinear temperature distribution may be converted to an equivalent linear temperature distribution for use in design of the section. Use of the equivalent linear temperature distribution in these cases will not have a significant effect on the capacity of the member. The method for this conversion is given in Figure RA-1 for a rectangular section. The line T_L in Figure RA-1-III is the equivalent linear temperature distribution for use in design.

RA.3—General design requirements

RA.3.1 Fig. RA.1 illustrates a technique for considering the gradient temperature distribution effect and the difference between mean temperature distribution and base temperature.

Base Temperature	$= T_b$ as shown in Figure A1-I
Mean Temperature	$= T_m$ as shown in Figure AI-II and determined by Step 2
Temperature Distribution	= The curve T_{NL} or its linearized equivalent T_L as shown in Figure A1-III and determined by Step 3

Step 1. Obtain temperature distribution T_{NL} through the concrete thickness.

Step 2. Determine mean temperature distribution T_m as

$$T_m = \frac{\int_{-t/2}^{t/2}[T_{NL} - T_b]dy}{t}$$

The integral is the area bounded between T_{NL} and T_b.

Step 3. For the nonlinear temperature distribution T_{NL}, determine the equivalent linear temperature distribution, T_L, such that this equivalent distribution produces the same uncracked moment about the center line of the section as does the nonlinear temperature distribution. The equivalent linear temperature distribution has an average temperature of T_m and a gradient ΔT. For rectangular sections,

$$\Delta T = (12/t^2)\int_{-t/2}^{t/2} T_{NL}\, y\, dy$$

The integral is the moment of the area under T_{NL} about the section line.

Fig. RA.1—Temperature distributions

The gradient temperature distribution is represented by $\mathbf{D}T$ in Fig. RA.1-III, which is acceptable for cracked sections. The difference between the mean temperature distribution and the base temperature is $T_m - T_b$.

RA.3.2 The time-dependent variations discussed in this section refer to the direct variation of temperature with time, excluding relaxation and creep effects. The latter are considered in Section A.3.4.

RA.3.3 When evaluating thermal stress in flexural members, the calculations are strongly influenced by the rigidity of a given cross section, the total stiffness of the member in question, and the restraint against deformation offered by the structure. Thus, the cracking of each cross section (rigidity), the variation of cracking along the length of the member (stiffness), and the freedom of the member to move under thermal loads restraint must be considered.

In order to ensure serviceability of the structure, steady-state temperature conditions should be considered. The analysis should consider crack control on the tensile face, strain limitations for the section, and deflections of the structural elements.

The limiting reinforcement requirements of Section 10.3.2 provide sufficient rotational capacity at the ends of all members to accommodate some magnitude of thermal strains without influencing the capacity of the member to support mechanical loading.

In addition, deflection of structural elements may need to be considered in the design of nonstructural items attached to concrete members (see Section 9.5).

The thermal stress problem can be handled in any of the following three ways:

1. Most structural analyses treat thermal loads acting on a monolithic section and evaluate the rigidity of the section and the stiffness of the element based on full uncracked cross sections. Although fairly easy to perform, such an analysis may be overly conservative because it does not consider the self-relieving nature of thermal stress due to cracking and deformation;

2. Analyses may consider the cracking of concrete for all loads, mechanical and thermal. Although this approach probably is the most accurate and generally results in the largest degree of self-relieving thermal stress, it is very complex, involving significant non-linear analysis and iterative solutions not readily available to the engineer; and

3. The third alternative is to consider the structure uncracked for mechanical loads and only consider the effect of cracking on thermal loads. The difficult part of such an analysis is the determination of that part of the thermal load that causes cracking and that part then can be considered acting on a cracked section.

RA.3.4 One of the major concerns in the evaluation of stresses due to temperature is that they do not significantly reduce the magnitude of stress resulting from mechanical loads. One of the major reasons for this concern is that thermal stress may eventually relax with time. Thus, if any advantage is to be taken from thermal stresses reducing mechanical stress, loss of stress due to relaxation must be considered. The literature is replete with analyses and descriptions of creep and relaxation. One of the most used documents on the subject is ACI Publication SP-27, *De-signing for Effects of Creep, Shrinkage and Temperature in Concrete Structures.*

RA.4—Concrete temperatures

The concrete temperatures given in this section are identical to those given by ACI 359.

APPENDIX B—ANCHORING TO CONCRETE

ACI 349 Appendix B was developed in the mid 1970s following review of design methods and available test data. Since that time there has been extensive additional test data. In 1992, a task group was formed to compare the Appendix B methodology to that of the Concrete Capacity Design (CCD) Method for all available tests results. The review indicated that concrete breakout failures were predicted better (for example, for bolt groups and edge conditions) by the new prediction equations in the Concrete Capacity Design Method than by the current design method of Appendix B. After extensive review, ACI 349 chose to incorporate the Concrete Capacity Design Method to improve the requirements of the previous Appendix B. References B.1 and B.2 describe the background and show comparisons of this method against the methods specified in ACI 349-97 Appendix B.

Evaluations of the methodology of ACI 349 Appendix B and the Concrete Capacity Design Method are provided in References B.3 to B.6. These evaluations are based on the provisions included in the 1976, 1985, and 1997 editions of Appendix B. This work and additional testing is described in Reference B.7. Comparisons between the methods are shown in RB.13. These comparisons show the following key differences in the requirements:

• The concrete breakout strength increases with embedment depth. In Appendix B (ACI 349-97) the increase was proportional to the square of the embedment depth. In the Concrete Capacity Design Method the increase is proportional to the embedment depth to the power of 1.5. The methods give similar results at about 5 in. of embedment depth; the Concrete Capacity Design Method is more conservative for increased embedment depth; and

• The concrete breakout strength is affected by the spacing to adjacent anchors and edges. The Concrete Capacity Design Method assumes no interaction when the spacing of adjacent anchors is three times the embedment depth, whereas Appendix B (ACI 349-97) assumed two times the embedment depth. The Concrete Capacity Design Method assumes no interaction when the anchors are installed with edge distance greater than 1.5 times the embedment depth, while Appendix B (ACI 349-97) assumed 1.0 times the embedment depth.

RB.0—Notation

A_{se} = the effective stress area A_{se} may be different in tension and shear. Reductions in cross section due to threading or an expansion mechanism affect the tension area but may not affect the effective shear area. The effective cross-sectional area of an anchor should be provided by the manufacturer of expansion anchors with reduced cross-sectional area for the expansion mecha-
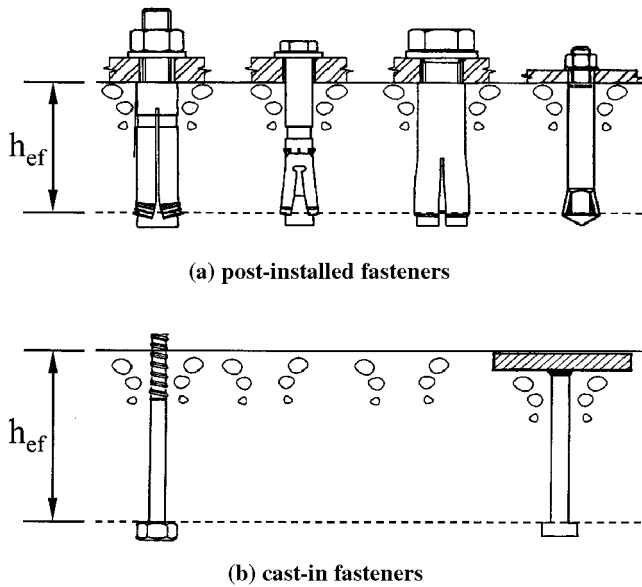
(a) post-installed fasteners

(b) cast-in fasteners

Fig. RB.1—Types of fasteners.

nism. For threaded bolts, ANSI/ASME B1.1[B.8] defines A_{se} as

$$A_{se} = \frac{\pi}{4}\left(d_o - \frac{0.9743}{n_t}\right)^2$$

where n_t is the number of threads per in.

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 areas of the group of anchors loaded in tension, in. [See Fig. RB.5.2((b) and (c))]

h_{ef} = effective embedment depths for a variety of anchor types are shown in Fig. RB.1.

K_c = confinement factor (RB.11)

RB.1—Definitions
Brittle steel element and ductile steel element—The 14% elongation should be measured over the gage length specified in the appropriate ASTM standard for the steel.
5% fractile—The determination of the coefficient K associated with the 5% fractile, $\bar{x} - K s$, depends on the number of tests n used to compute $\bar{x}$ and s. Values of K range, for example, from 1.645 for $n = \infty$, to 2.010 for $n = 40$, and 2.568 for $n = 10$.

RB.2—Scope
RB.2.1 ACI 349 uses the term embedments to cover a broad scope that includes anchors, embedded plates, shear lugs, grouted embedments, and specialty inserts. It covers the same scope as was included in the 1997 Code.

RB.2.3 Typical cast-in headed studs and headed bolts with geometries consistent with ANSI/ASME B1.1,[B.8] B18.2.1,[B.9] and B18.2.6[B.10] have been tested and have proven to behave predictably, so calculated pullout values are acceptable. Post-installed anchors do not have predictable pullout capacities, and therefore are required to be tested.

RB.2.6 Typical embedment configurations are shown in Fig. RB.2.6 (a), (b), (c), and (d). These figures also indicate the extent of the embedment within the jurisdiction of this Code.

RB.3—General requirements
RB.3.1 When the strength of an anchor group is governed by breakage of the concrete, the behavior is brittle and there is limited redistribution of the forces between the highly stressed and less-stressed anchors. In this case, the theory of elasticity is required to be used assuming the attachment that distributes loads to the anchors is sufficiently stiff. The forces in the anchors are considered to be proportional to the external load and its distance from the neutral axis of the anchor group.

If anchor strength is governed by ductile yielding of the anchor steel, significant redistribution of anchor forces can occur. In this case, an analysis based on the theory of elasticity will be conservative. References B.11 to B.13 discuss nonlinear analysis, using the theory of plasticity for the determination of the capacities of ductile anchor groups.

RB.3.3 Many anchors in a nuclear power plant must perform as designed with high confidence, even when exposed to significant seismic loads. To prevent unqualified anchors being used in connections that must perform with high confidence under significant seismic load, all anchors are required to be qualified for seismic-zone usage by passing simulated seismic tests. The qualification should be performed consistent with the provisions of this appendix and should be reviewed by a professional engineer experienced in anchor technology. Typical simulated seismic-testing methods are described in Reference B.7. For a post-installed anchor to be used in conjunction with the requirements of this appendix, the results of tests have to indicate that pullout failures exhibit an acceptable load-displacement characteristic, or that pullout failures are precluded by another failure mode. ACI 349 requires that all post-installed anchors be qualified, by independent tests, for use in cracked concrete. Anchors qualified for use only in uncracked concrete are not recommended in nuclear power plant structures.

The design of the anchors for impactive or impulsive loads is not checked directly by simulated seismic tests. An anchor that has passed the simulated seismic tests, however, should function under impactive tensile loading in cracked concrete.

RB.3.4 The provisions of Appendix B are applicable to normalweight concrete. The design of anchors in heavyweight concrete should be based on testing for the specific heavyweight concrete.

RB.3.5 A limited number of tests of cast-in and post-installed anchors in high-strength concrete[B.14] indicate that the design procedures contained in this appendix overpredict strength, particularly for cast-in anchors, at $f_c' >$ 10,000 psi. Until further tests are available, an upper limit of $f_c' =$ 10,000 psi has been imposed in the design of cast-in anchors. This is consistent with Chapters 11 and 12. Some post-installed anchors may have difficulty expand-
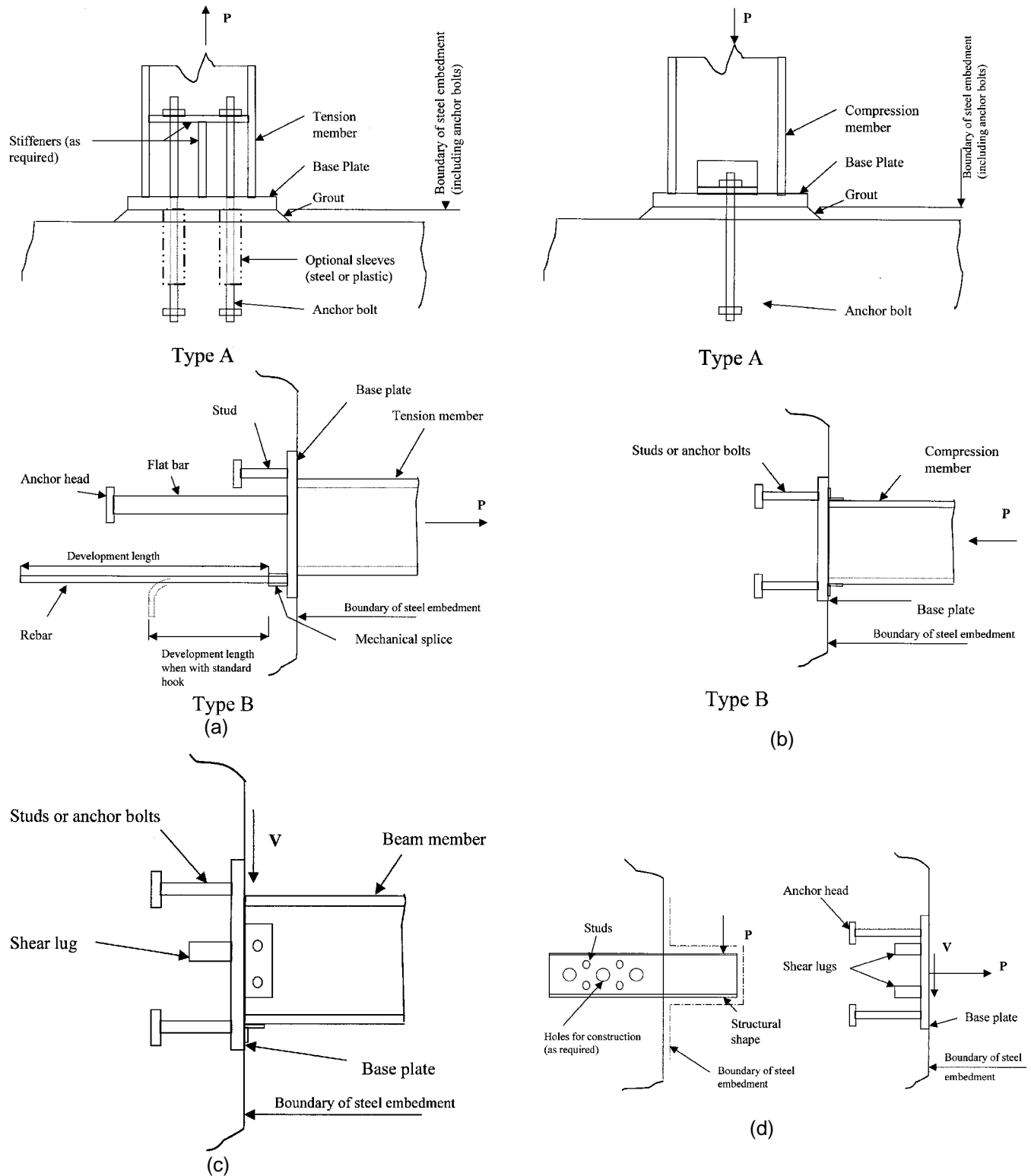
Fig. RB2.6—(a) Typical embedments for tension loads; (b) typical embedments for compression loads; (c) typical embedments for shear loads; and (d) typical embedments for combined loads.

ing in high-strength concrete. Because of this, f_c' is limited to 8000 psi in the design of post-installed anchors, unless testing is performed.

RB.3.6.1 The design provisions of ACI 349 Appendix B for anchors in nuclear power plants retain the philosophy of previous editions of ACI 349 by encouraging anchor designs to have a ductile-failure mode. This is

consistent with the strength-design philosophy of reinforced concrete in flexure. The failure mechanism of the anchor is controlled by requiring yield of the anchor prior to a brittle concrete failure. A ductile design provides greater margin than a nonductile design because it permits redistribution of load to adjacent anchors and can reduce the maximum dynamic load by energy absorption and re-

duction in stiffness. For such cases, the design strength is the nominal strength of the steel, multiplied by a strength reduction factor of 0.80.

The specified ultimate tensile strength of the embedment should be determined based on those portions of the embedment that transmit tension or shear loads into the concrete. The ultimate shear strength of the steel is taken as 65% of the ultimate tensile strength. It is not necessary to develop an embedment for full axial tension and full shear if it can be demonstrated that the embedment will be subjected to one type of loading (such as tension, shear, or flexure). An embedment need not be developed for tension or shear if the load is less than 20% of the full tension or shear capacity. This value of 20% is consistent with the value of 20% used in the equation in B.7.

An embedment may be considered subject to flexure only when the axial tension loads on the embedment are less than 20% of the nominal strength in tension.

RB.3.6.2 A ductile design can also be achieved by designing the attachment to yield before failure of the anchors. In such a case, the anchors can be nonductile so long as they are stronger than the yield strength of the attachment. This is established with a margin equivalent to that in B.3.6.1. B.3.6.2 is based on attachment yield strength f_y, whereas B.3.6.1 uses f_{ut} because attachments are typically of A36 material, and the strength is better characterized by the yield strength. The 0.75 factor allows for the actual yield versus specified minimum yield.

RB.3.6.3 There are situations where a ductile-failure mode cannot be achieved. Previous editions of ACI 349 included specific provisions for commercially available, nonductile expansion anchors that were penalized by specifying a lower strength reduction factor. The current Appendix B includes more general provisions for anchors for which a ductile-failure mode cannot be achieved. Such situations can occur for anchors in shallow slabs, close to edges, or close to other anchors. The factor of 0.60 is specified to account for the lower margins inherent in a nonductile design relative to those in a ductile design.

RB.3.8 Ductile steel elements are defined in B.1 to have a minimum elongation of 14%. This requirement is meant to ensure sufficient ductility in the embedment steel. The limit of 14% is based on ASTM A 325[B.15] and A 490[B.16] anchor materials that have been shown to behave in a ductile manner when used for embedment steel.

RB.3.9 Anchors that incorporate a reduced section (such as threads, notch, or wedge) in the load path (the term load path includes tension load path and shear load path) may fail in the reduced section before sufficient inelastic deformation has occurred to allow redistribution of anchor tension and shear forces, thus exhibiting low ductility. This can be prevented by requirement (a), which ensures that yield of the unreduced section will occur before failure of the reduced section. Shear failure can be affected significantly by reduced sections within five diameters of the shear plane (many wedge-type anchors). In this case, tests for the evaluation of the shear capacity are required. Tests reported in Reference B.11 for a limited number of attachment types, steel strength, and diameters

have shown that threaded anchors will exhibit sufficient ductility to redistribute tension and shear forces.

RB.3.10 The design provisions for impulsive and impactive loads in Appendix C may be used for embedments. Energy can be absorbed by deformation of anchors designed for ductile steel failure.

RB.4—General requirements for strength of structural anchors

RB.4.1 This section provides the requirements for establishing the strength of anchors to concrete. The various types of steel and concrete failure modes for anchors are shown in Fig. RB.4.1(a) and RB.4.1(b). Comprehensive discussions of anchor failure modes are included in References B.1, B.2, and B.17. Any model that complies with the requirements of B.4.2 and B.4.3 can be used to establish the concrete-related strengths. For anchors such as headed bolts, headed studs, and post-installed anchors, the concrete breakout design method of B.5.2 and B.6.2 is acceptable. The anchor strength is also dependent on the pullout strength of B.5.3, the side-face blowout strength of B.5.4, and the minimum spacing and edge distances of B.8. The design of anchors for tension recognizes that the strength of anchors is sensitive to appropriate installation; installation requirements are included in B.9.

Test procedures can also be used to determine the single anchor breakout strength in tension and in shear. The test results, however, are required to be evaluated on a basis statistically equivalent to that used to select the values for the concrete breakout method considered to satisfy provisions of B.4.2. The nominal strength cannot be taken greater than the 5% fractile. The number of tests has to be considered in determining the 5% fractile.

RB.4.2 and RB.4.3 B.4.2 and B.4.3 establish the performance factors for which anchor design models are required to be verified. Many possible design approaches exist, and the user is always permitted to design by test using B.4.2 as long as sufficient data are available to verify the model.

RB.4.2.1 The addition of supplementary reinforcement in the direction of the load, confining reinforcement, or both, can greatly enhance the strength and ductility of the anchor connection. Such enhancement is practical with cast-in anchors such as those used in precast sections.

The shear strength of headed anchors located near the edge of a member can be significantly increased with appropriate supplementary reinforcement. References B.17 to B.19 provide information on designing such reinforcement. The effect of supplementary reinforcement is not included in the concrete breakout calculation method of B.5.2 and B.6.2. The engineer has to rely on other test data and design theories to include the effects of supplementary reinforcement.

For anchors exceeding the limitations of B.4.2.2, or for situations where geometric restrictions limit breakout capacity, or both, reinforcement proportioned to resist the total load, oriented in the direction of load, within the breakout prism and fully anchored on both sides of the
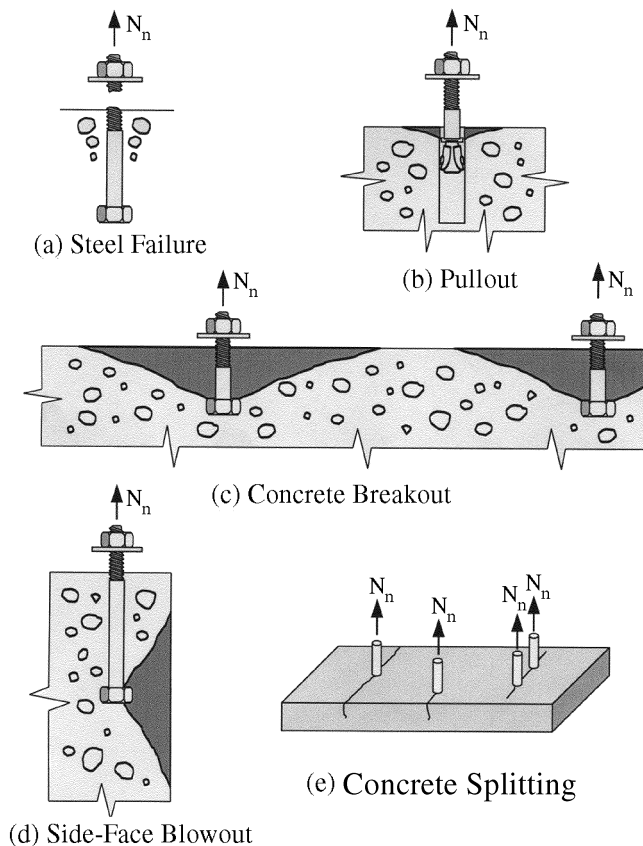
Fig. RB.4.1(a)—Failure modes for fasteners under tensile loading.

Fig. RB.4.1(b)—Failure modes for fasteners under shear loading.

breakout planes, may be provided instead of calculating breakout capacity.

The breakout strength of an unreinforced connection can be taken as an indication of the load at which significant cracking will occur. Such cracking can represent a serviceability problem if not controlled (see B.6.2.1).

RB.4.2.2 The method for concrete breakout design, included as considered to satisfy B.4.2, was developed from the Concrete Capacity Design (CCD) Method,[B.1,B.2] which was an adaptation of the κ Method,[B.20, B.21] and is considered to be accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD Method predicts the load capacity of an anchor or group of anchors by using a basic equation for tension or shear for a single anchor in cracked concrete, and multiplying by factors that account for the number of anchors, edge distance, spacing, eccentricity, and absence of cracking. The limitations on anchor size and embedment depth are based on the range of test data.

The breakout strength calculations are based on a model suggested in the **k** Method. It is consistent with a breakout prism angle of approximately 35 degrees[Fig. RB.4.2(a) and (b)].

RB.4.4 The **f** factors for steel strength are based on using f_{ut} to determine the nominal strength of the anchor (see B.5.1 and B.6.1) rather than f_y as used in the design of reinforced concrete members. Although the **f** factors for use with f_{ut} appear low, they result in a level of safety consistent with the use of higher **f** factors applied to f_y. The smaller **f** factors for shear than for tension do not reflect basic material differences, but rather account for the
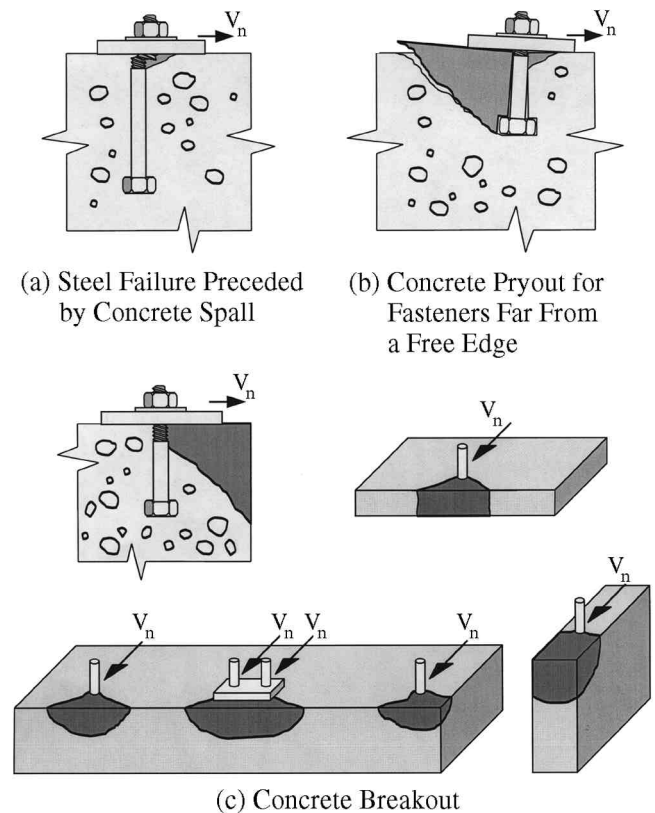
possibility of a nonuniform distribution of shear in connections with multiple anchors. It is acceptable to have a ductile failure of a steel element in the attachment if the attachment is designed so that it will undergo ductile yielding at a load level no greater than 75% of the minimum design strength of an anchor (See B.3.6.2). The **f** factor for anchors governed by the more brittle concrete breakout or blowout failure is lower than for a ductile steel failure. Even though the **f** factor for plain concrete uses a value of 0.65, the basic factor for brittle failures (**f** = 0.75) has been chosen based on the results of probabilistic studies.[B.3] For anchoring to concrete, the use of **f** = 0.65 with mean values of concrete-controlled failures produced adequate safety levels. The nominal resistance expressions, however, used in this appendix and in the test requirements are based on the 5% fractiles. Thus, the **f** = 0.65 value would be overly conservative. Comparison with other design procedures and probabilistic studies[B.3] indicated that the choice of **f** = 0.75 was justified.

RB.4.5—Bearing strength

RB.4.5.1 B.4.5.1 prohibits the engineer from combining shear strength of bearing (for example, a shear lug) and shear friction (such as shear studs) mechanisms. This exclusion is justified in that it is difficult to predict the distribution of shear resistance as a result of differential stiffness of the two mechanisms. This exclusion is required because of the displacement incompatibility of these two independent and nonconcurrent mechanisms. Tests show that the relatively smaller displacements asso-
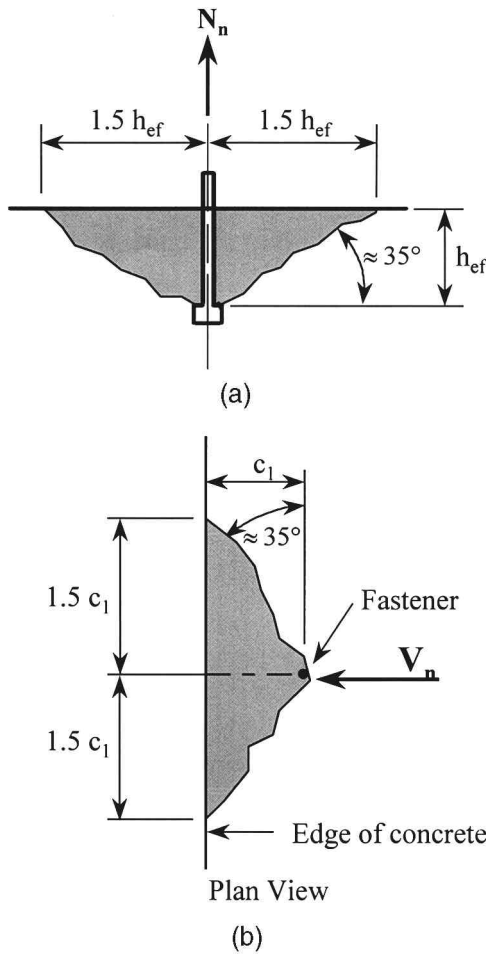
Fig. RB.4.2—Breakout cone for: (a) tension; and (b) shear.

ciated with the bearing mode preclude development of the shear-friction mode until after bearing mode failure.[B.22] As described in RB.11.1, however, the confining forces afforded by the tension anchors in combination with other concurrent external loads acting across potential shear planes can result in a significant and reliable increase in bearing mode shear capacity and can therefore be used.

RB.4.5.2 For shear lugs, the nominal bearing strength value of 1.3 $f_c\mathbf{\ell}$ is recommended based on the tests described in Reference B.22 rather than the general provisions of 10.15. The factor of 0.70 corresponds to that used for bearing on concrete in Chapter 9.

RB.5—Design requirements for tensile loading
RB.5.1—Steel strength of anchor in tension

RB.5.1.2 The nominal tension strength of anchors is best represented by $A_{se}f_{ut}$ rather than $A_{se}f_y$ because typical anchor materials do not exhibit a well-defined yield point. The American Institute of Steel Construction (AISC) has based tension strength of anchors on $A_{se}f_{ut}$ since the 1986 edition of their specifications. The use of Eq. (B-3) with the load factors of Section 9.2 and the $\mathbf{f}$ factors of B.4.4 gives results consistent with the AISC Load and Resistance Factor Design Specifications.

The limitation of $1.9f_y$ on f_{ut} is to ensure that under service load conditions the anchor does not exceed f_y. The

limit on f_{ut} of $1.9f_y$ was determined by converting the LRFD provisions to corresponding service level conditions. For ACI Section 9.2, the average load factor of 1.55 (from $1.4D + 1.7L$) divided by the highest $\mathbf{f}$ factor (0.8 for tension) results in a limit of f_{ut}/f_y of $1.55/0.8 = 1.94$. For consistent results, the serviceability limitation of f_{ut} was taken as $1.9f_y$. If the ratio of f_{ut} to f_y exceeds this value, the anchor may be subjected to service loads above f_y. Although not a concern for standard structural steel anchors (maximum value of f_{ut}/f_y is 1.6 for ASTM A 307), the limitation is applicable to some stainless steels.[B.23]

RB.5.2—Concrete breakout strength of anchor in tension

RB.5.2.1 The effects of multiple anchors, spacing of anchors, and edge distance on the nominal concrete breakout strength in tension are included by applying the modification factors A_N/A_{No} and $\mathbf{y_2}$ in Eq. (B-4).

Figure RB.5.1(a) shows A_{No} and the development of Eq. (B-5). A_{No} is the maximum projected area for a single anchor. Figure RB.5.1(b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. Because A_N is the total projected area for a group of anchors, and A_{No} is the area for a single anchor, there is no need to include n (the number of anchors) in Eq. (B-4a) or (B-4b). If anchor groups are positioned in such a way that their projected areas overlap, the value of A_N is required to be reduced accordingly.
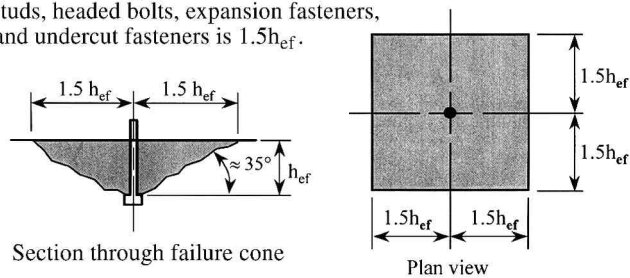
RB.5.2.2 The basic equation for anchor capacity was derived[B.1, B.2, B.18, B.21] assuming a concrete failure prism with an angle of about 35 degrees and considering fracture mechanics concepts.

The values of k were determined from a large database of test results in uncracked concrete[B.1] at the 5% fractile. The values were adjusted to corresponding k values for cracked concrete.[B.2, B.24] For anchors with a deep embedment depth ($h_{ef} > 11$ in.), some test evidence indicates using $h_{ef}^{1.5}$ can be overly conservative in some cases. Often, such tests have been performed with selected aggregates for special applications. An alternative expression (Eq. (B-6b)) is provided using $h_{ef}^{5/3}$ for evaluation of cast-in anchors with 11 in. $< h_{ef} < 25$ in. The limit of 25 in. corresponds to the upper range of test data. This expression can also be appropriate for some undercut post-installed anchors. B.4.2, however, should be used with test results to justify such applications.

RB.5.2.3 For anchors influenced by three or more edges where any edge distance is less than $1.5h_{ef}$, the tensile breakout strength computed by the ordinary CCD Method, which is the basis for Eq. (B-5), gives misleading results. This occurs because the ordinary definitions of A_N/A_{No} do not correctly reflect the edge effects. If the value of h_{ef} is limited to $c_{max}/1.5$, where c_{max} is the largest of the influencing edge distances that are less than or equal to the actual $1.5h_{ef}$, this problem is corrected. As shown by Lutz,[B.25] this limiting value of h_{ef} is to be used in Eq. (B-5) to (B-8). This approach is best understood when applied to an actual case. Figure RB.5.2(a) shows how the failure surface has the same area for any embedment depth beyond the proposed limit on h_{ef} (taken as $h\mathbf{\ell}_f$ in the figure). In this
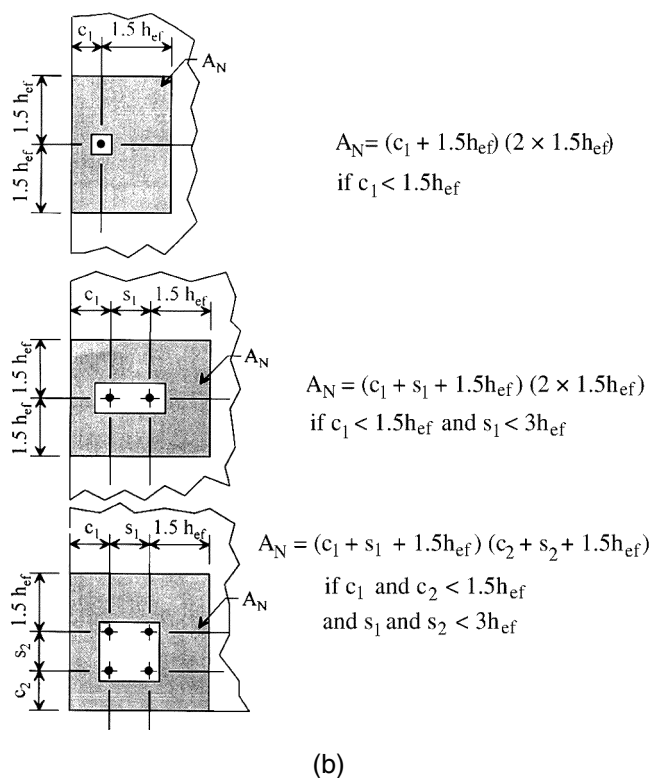
The critical edge distance for headed studs, headed bolts, expansion fasteners, and undercut fasteners is $1.5h_{ef}$.

$$A_{No} = 2 \times 1.5h_{ef} \times 2 \times 1.5h_{ef}$$
$$= 3h_{ef} \times 3h_{ef}$$
$$= 9h_{ef}^2$$

(a)

$A_N = (c_1 + 1.5h_{ef})(2 \times 1.5h_{ef})$
if $c_1 < 1.5h_{ef}$

$A_N = (c_1 + s_1 + 1.5h_{ef})(2 \times 1.5h_{ef})$
if $c_1 < 1.5h_{ef}$ and $s_1 < 3h_{ef}$

$A_N = (c_1 + s_1 + 1.5h_{ef})(c_2 + s_2 + 1.5h_{ef})$
if c_1 and $c_2 < 1.5h_{ef}$
and s_1 and $s_2 < 3h_{ef}$

(b)

Fig. RB.5.1—(a) Calculation of A_{No}; and (b) projected areas for single fasteners and groups of fasteners.

example, the proposed limit on the value of $h_{ef} = c_{max}/1.5$ to be used in the computations results in $h_{ef} = h_{ef}' = 4$ in./1.5 = 2.67 in. This would be the proper value to be used for h_{ef} in computing the resistance for this example, even if the actual embedment depth is larger.

RB.5.2.4 Figure RB.5.2(b) shows dimension $e_N' = e_N$ for a group of anchors that is in tension, but has a resultant force eccentric with respect to the centroid of the anchor group. Groups of anchors can be loaded in such a way that only some of the anchors are in tension (Fig. RB.5.2(c)). In this case, only the anchors in tension are to be considered in determining e_N'. The anchor loading has to be determined as the resultant anchor tension at an eccentricity with respect to the center of gravity of the anchors in tension. Equation (B-7) is limited to cases where $e_N' \leq s/2$ to alert the designer that all anchors may not be in tension.

RB.5.2.5 If anchors are located close to an edge so that there is not enough space for a complete breakout prism to develop, the load-bearing capacity of the anchor is further reduced beyond that reflected in A_N/A_{No}. If the smallest side cover distance is greater than $1.5h_{ef}$, a complete prism can form, and there is no reduction ($\Psi_2 = 1$). If the side cover is less than $1.5\,h_{ef}$, the factor Ψ_2 is required to adjust for the edge effect.[B.1]

RB.5.2.6 The analyses for cracking should consider all specified load combinations using unfactored loads, including the effects of restrained shrinkage. Anchors that perform well in a crack that is 0.012 in. wide are considered suitable for use in cracked concrete. If wider cracks are expected, confining reinforcement to control the crack width to about 0.012 in. should be provided.

RB.5.2.8 In the future, there are expected to be more expansion and undercut anchors that are to be calculated with the k-value for headed studs. Tests with one special undercut anchor have shown that this is possible.

RB.5.3—Pullout strength of anchor in tension
RB.5.3.3 The pullout strength in tension of headed studs or headed bolts can be increased by providing confining reinforcement, such as closely spaced spirals, throughout the head region. This increase can be demonstrated by tests.

RB.5.3.4 Equation (B-10) corresponds to the load at which the concrete under the anchor head begins to crush.[B.17] It is not the load required to pull the anchor completely out of the concrete, so the equation contains no term relating to embedment depth. The designer should be aware that local crushing under the head will greatly reduce the stiffness of the connection and generally will be the beginning of a pullout failure.

RB.5.4—Concrete side-face blowout strength of anchor in tension
The design requirements for side-face blowout are based on the recommendations of Reference B.26. Side-face blowout may control when the anchor is close to an edge ($c < 0.4\,h_{ef}$). These requirements are applicable to headed anchors that usually are cast-in anchors. Splitting during installation, rather than side-face blowout, generally governs post-installed anchors. When a group of anchors is close to an edge, side-face blowout will be controlled by the row of anchors closest to the edge. The anchors away from the edge will have greater strength than those closest to the edge. The side-face blowout of the group is conservatively calculated using the strength of the anchors closest to the edge.

RB.6—Design requirements for shear loading
RB.6.1—Steel strength of anchor in shear
RB.6.1.2 The nominal shear strength of anchors is best represented by $A_{se}f_{ut}$ for welded headed stud anchors, and $0.6A_{se}f_{ut}$ for other anchors rather than a function of $A_{se}f_y$ because typical anchor materials do not exhibit a well-defined yield point. The use of Eq. (B-13) and (B-14) with the load factors of Section 9.2 and the ϕ factors of B.4.4 gives results consistent with the AISC Load and Resistance Factor Design Specifications.
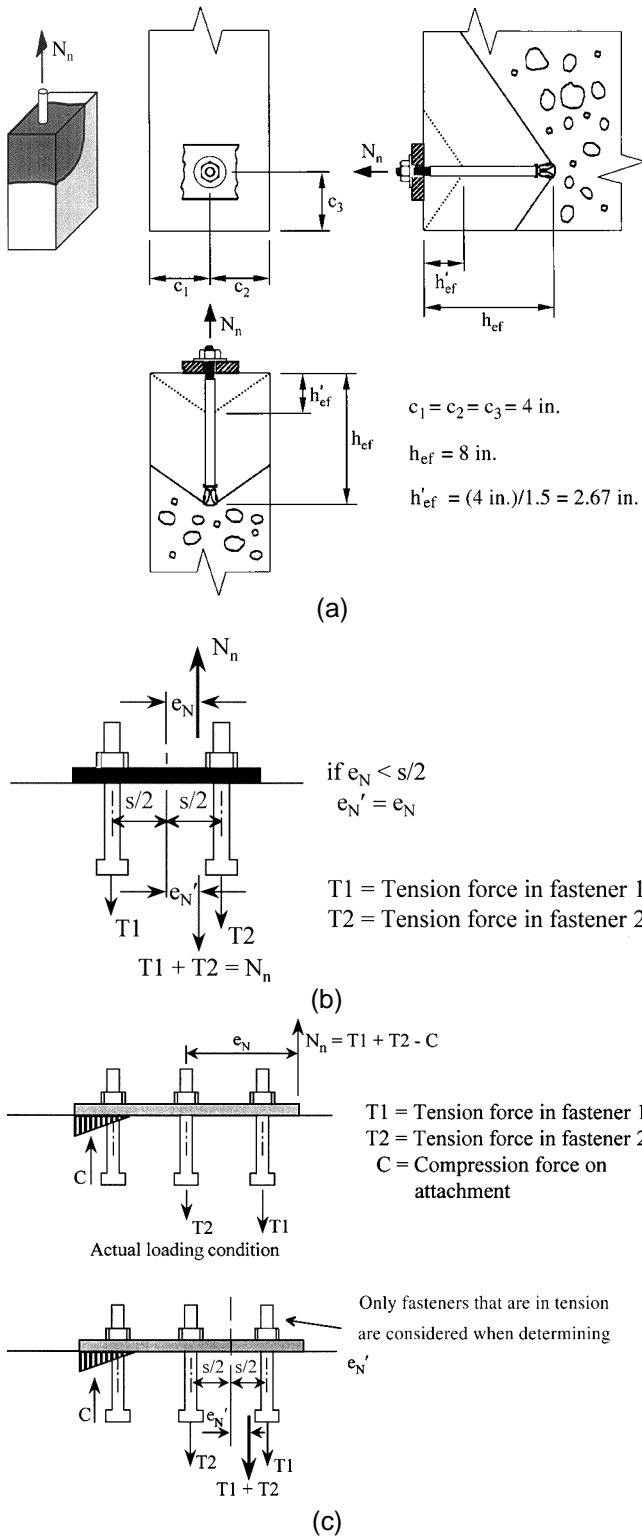
Fig. RB.5.2—(a) Failure surfaces in narrow members for different embedment depths; (b) definition of dimension e_N' when all fasteners in a group are in tension; and (c) determination of e_N' for fastener group with only some fasteners in tension.

The limitation of $1.9f_y$ on f_{ut} is to ensure that, under service load conditions, the anchor does not exceed f_y. The limit on f_{ut} of $1.9f_y$ was determined by converting the LRFD provisions to corresponding service level conditions as discussed in B.5.1.2.

RB.6.1.3 The shear strength of a grouted base plate is based on limited testing. It is recommended that the height of the grout pad not exceed 2 in.

RB.6.1.4 The friction force that develops between the base plate and concrete due to the compressive resultant from moment and/or axial load contributes to the shear strength of the connection. For as-rolled base plates installed against hardened concrete, the coefficient of friction is approximately 0.40.[B.11]

If the frictional strength is larger than the applied shear load, the base plate will not slip. When the frictional strength is less than the applied shear, the shear resistance will be a combination of both frictional strength and shear strength provided by the anchors. It must be assured that the compressive resultant used in determining the frictional resistance acts concurrent with the shear load. The presence or absence of loads should satisfy Section 9.2.3. Compressive resultants due to secondary loads should not be considered.

RB.6.2—Concrete breakout strength of anchor in shear

RB.6.2.1 The shear-strength equations were developed from the CCD method. They assume a breakout cone angle of approximately 35 degrees (Fig. RB.4.2(b)) and consider fracture mechanics theory. The effects of multiple anchors, spacing of anchors, edge distance, and thickness of the concrete member on nominal concrete breakout strength in shear are included by applying the reduction factor A_V/A_{Vo} and ψ_5 in Eq. (B-16). For anchors far from the edge, B.6.2 usually will not govern. For these cases, B.6.1 and B.6.3 often govern.

Figure RB.6.2(a) shows A_{Vo} and the development of Eq. (B-17). A_{Vo} is the maximum projected area for a single anchor that approximates the surface area of the full breakout prism or cone for an anchor unaffected by edge distance, spacing, or depth of member. Figure RB.6.2(b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. A_V approximates the full surface area of the breakout cone for the particular arrangement of anchors. Because A_V is the total projected area for a group of anchors, and A_{Vo} is the area for a single anchor, there is no need to include the number of anchors in the equation.

The assumption shown in Fig. RB.6.2(b) with the case for two anchors perpendicular to the edge is a conservative interpretation of the distribution of the shear force on an elastic basis. If the anchors are welded to a common plate when the anchor nearest the front edge begins to form a failure cone, shear load would be transferred to the stiffer and stronger rear anchor. For cases where nominal strength is not controlled by ductile steel elements, B.3.1 specifies that load effects be determined by elastic analysis. It has been suggested in the PCI Design Handbook approach[B.27] that the increased capacity of the anchors away from the edge be considered. Because this is a reasonable approach, assuming that the anchors are spaced far enough apart so that the shear failure surfaces do not intersect,[B.18] B.6.2 allows such a procedure. If the failure surfaces do not intersect, as would generally occur if the

The critical edge distance for headed studs, headed bolts, expansion fasteners, and undercut fasteners is $1.5c_1$

$A_{Vo} = 2 \times 1.5c_1 \times 1.5c_1$
$= 3c_1 \times 1.5c_1 = 4.5c_1^2$

(a)

if $h < 1.5c_1$
$A_V = 2 \times 1.5c_1 \times h$

if $h < 1.5c_1$
$A_V = 2 \times 1.5c_1 \times h$

Note: One assumption of the distribution of forces indicates that half the shear would be critical on front fastener and its projected area

if $c_2 < 1.5c_1$
$A_V = 1.5c_1 (1.5c_1 + c_2)$

if $h < 1.5c_1$
$A_V = 2 \times 1.5c_1 \times h$

if $h < 1.5c_1$ and $s_1 < 3c_1$
$A_V = (2 \times 1.5c_1 + s_1) \times h$

Note: Another assumption of the distribution of forces that applies only where fasteners are rigidly connected to the attachment indicates that the total shear would be critical on the rear fastener and its projected area.

(b)

$V_{n \text{ (parallel to edge)}}$

$V_{n \text{ (parallel to edge)}} = 2V_{n \text{ (normal to edge)}}$
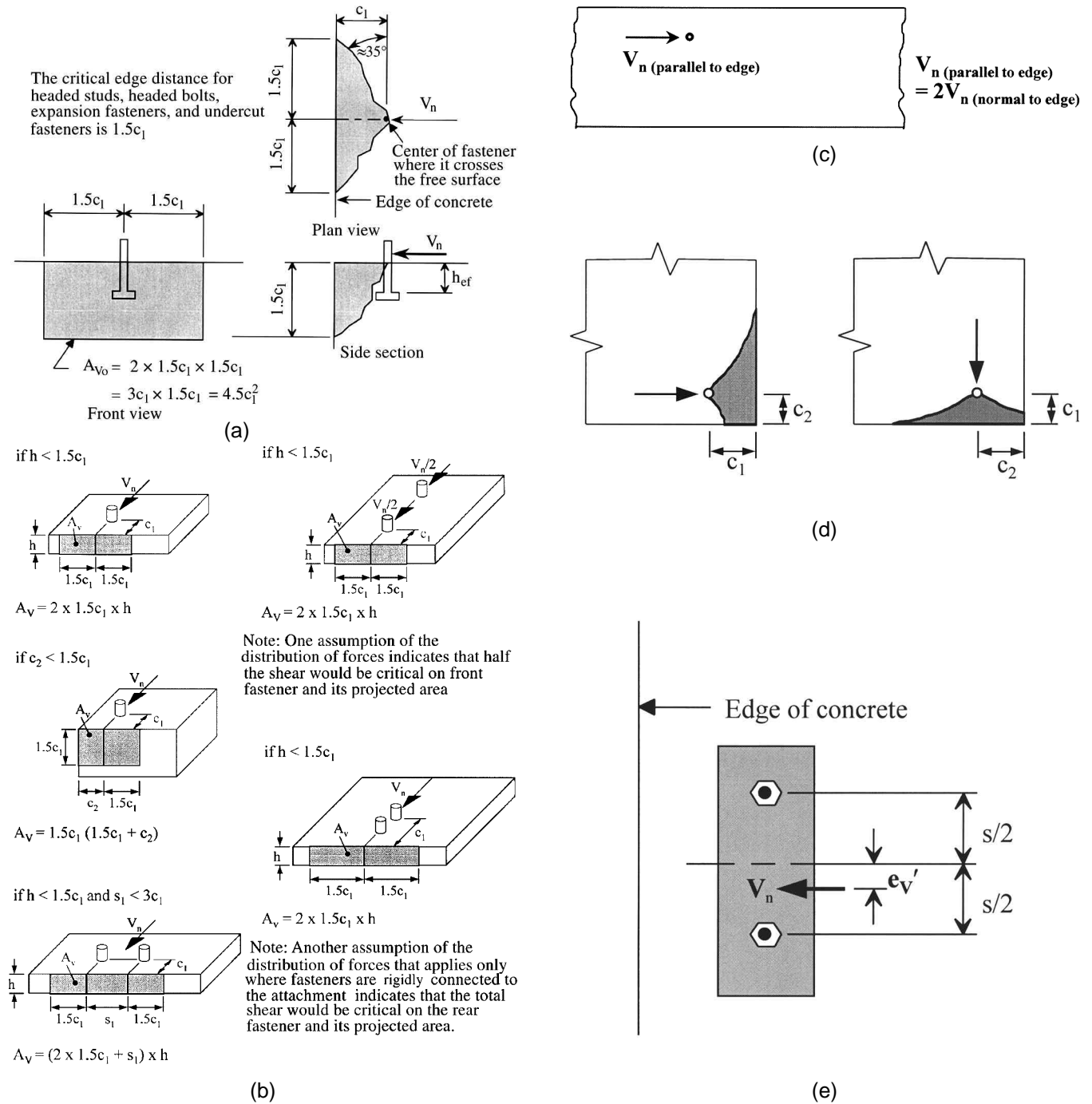
(c)

(d)

Edge of concrete

(e)

Fig.RB.6.2—(a) Calculation of A_{Vo}; (b) projected areas for single fasteners and groups of fasteners; (c) shear force parallel to an edge; (d) fasteners near a corner; and (e) definition of dimension e_V'.

anchor spacing s is equal to or greater than $1.5c_1$, then after formation of the near-edge failure surface, the higher capacity of the farther anchor would resist most of the load. As shown in the bottom example in Fig. RB.6.2(b), considering the full shear capacity to be provided by this anchor with its much larger resisting failure surface is appropriate. No contribution of the anchor near the edge is then considered. Checking the near-edge anchor condition to preclude undesirable cracking at service load conditions is advisable. Further discussion of design for multiple anchors is given in Reference B.17.

For the case of anchors near a corner subjected to a shear force with components normal to each edge, a satis-

factory solution is to check independently the connection for each component of the shear force. Other specialized cases, such as the shear resistance of anchor groups where all anchors do not have the same edge distance, are treated in Reference B.18.

The detailed provisions of B.6.2.1(a) apply to the case of shear force directed towards an edge. When the shear force is directed away from the edge, the strength will usually be governed by B.6.1 or B.6.3.

The case of shear force parallel to an edge (B.6.2.1(c)) is shown in Fig. RB.6.2(c). A special case can arise with shear force parallel to the edge near a corner. Take the example of a single anchor near a corner (Fig. RB.6.2(d)).

If the edge distance to the side c_2 is 40% or more of the distance c_1 in the direction of the load, the shear strength parallel to that edge can be computed directly from Eq. (B-16) using c_1 in the direction of the load.

RB.6.2.2 Like the concrete breakout tensile capacity, the concrete breakout shear strength does not increase with the failure surface, which is proportional to c_1^2. Instead, the strength increases proportionally to $c_1^{1.5}$ due to the size effect. The capacity is also influenced by the anchor stiffness and the anchor diameter.[B.1, B.2, B.18, B.21]

The constant 7 in the shear strength equation was determined from test data reported in Reference B.1 at the 5% fractile adjusted for cracking.

RB.6.2.3 For the special case of cast-in headed bolts rigidly welded to an attachment, test data[B.28, B.29] show that somewhat higher shear capacity exists, possibly due to the stiff welding connection clamping the bolt more effectively than an attachment with an anchor gap. Because of this, the basic shear value for such anchors is increased. Limits are imposed to ensure sufficient rigidity. The design of supplementary reinforcement is discussed in References B.17 to B.19.

RB.6.2.4 For anchors influenced by three or more edges where any edge distance is less than $1.5c_1$, the shear breakout strength computed by the basic CCD Method, which is the basis for Eq. (B-17), gives safe but misleading results. These special cases were studied for the **k** Method[B.21] and the problem was pointed out by Lutz. [B.25] Similar to the approach used for tensile breakouts in B.5.2.3, a correct evaluation of the capacity is determined if the value of c_1 in Eq. (B-17) to (B-20) is limited to $h/1.5$.

RB.6.2.5 This section provides a modification factor for an eccentric shear force towards an edge on a group of anchors. If the shear load originates above the plane of the concrete surface, the shear should first be resolved as a shear in the plane of the concrete surface, with a moment that can or cannot also cause tension in the anchors, depending on the normal force. Figure RB.6.2(e) defines the term e_v' for calculating the Ψ_5 modification factor that accounts for the fact that more shear is applied on one anchor than the other, tending to split the concrete near an edge. If $e_v' > s/2$, the CCD procedure is not applicable.

RB.6.2.7 Torque-controlled and displacement-controlled expansion anchors are permitted in cracked concrete under pure shear loads.

RB.6.3—Concrete pryout strength
Reference RB.1 indicates that the pryout shear resistance can be approximated as 1 to 2 times the anchor tensile resistance with the lower value appropriate for h_{ef} less than 2.5 in.

RB.7—Interaction of tensile and shear forces
The shear-tension interaction expression has traditionally been expressed as

$$\left(\frac{N}{N_n}\right)^\alpha + \left(\frac{V}{V_n}\right)^\alpha \le 1.0$$

where **a** varies from 1 to 2.

The current trilinear recommendation is a simplification of the expression where **a** = 5/3 (Fig. RB.7). The limits were chosen to eliminate the requirement for computation of interaction effects where very small values of the second force are present. Any other interaction expression that is verified by test data, however, can be used under B.4.3.

RB.8—Required edge distances, spacings, and thicknesses to preclude splitting failure
The minimum spacings, edge distances, and thicknesses are very dependent on the anchor characteristics. Installation forces and torques in post-installed anchors can cause splitting of the surrounding concrete. Such splitting can also be produced in subsequent torquing during connection of attachments to anchors including cast-in anchors. The primary source of values for minimum spacings, edge distances, and thicknesses of post-installed anchors should be the product-specific tests. In some cases, however, specific products are not known in the design stage. Approximate values are provided for use in design.

RB.8.2 In the absence of product-specific test information, at the design stage the minimum center-to-center spacing for post-installed anchors may be taken as $6d_o$.

RB.8.3 The edge cover over a deep embedment close to the edge can have a significant effect on the side-face blowout strength of B.5.4. The engineer can use cover larger than the normal concrete cover requirements to increase the side-face blowout strength.

RB.8.4 In the absence of product-specific test information, at the design stage the minimum edge distance may be taken as not less than:

Undercut anchors	$6d_o$
Torque-controlled expansion anchors	$8d_o$
Deformation-controlled expansion anchors	$10d_o$

If these values are used in design, the project drawings and project specifications should specify use of anchors with minimum center-to-center spacing and edge distance as assumed in design.

Headed anchors close to an edge are permitted to be torqued to 60% of the design strength.

Drilling holes for post-installed anchors can cause microcracking. The requirement for a minimum edge distance 2 times the maximum aggregate size is to minimize the effects of such microcracking.

RB.11—Shear capacity of embedded plates and shear lugs
RB.11.1—Shear lugs
The code requirements for the design of shear lugs are based on testing reported in Reference B.22. This testing confirmed that shear lugs are effective with axial compression and tension loads on the embedment, and that the strength is increased due to the confinement afforded by the tension anchors in combination with external loads. The shear strength of the embedment is the sum of the bearing strength and the strength due to confinement.

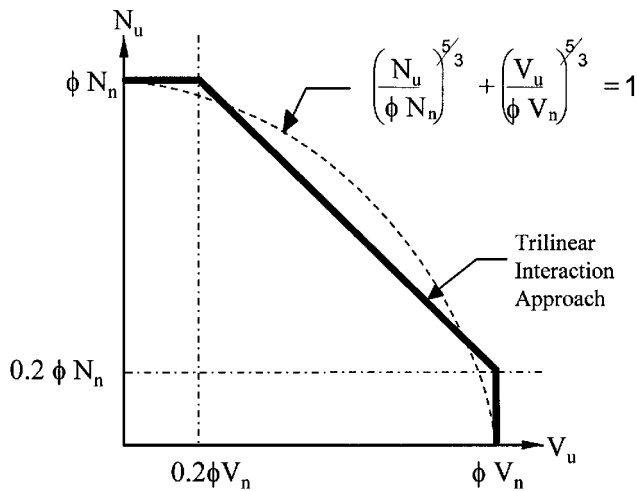The tests also revealed two distinct response modes:

$$\left(\frac{N_u}{\phi N_n}\right)^{5/3} + \left(\frac{V_u}{\phi V_n}\right)^{5/3} = 1$$

Fig. RB.7—Shear and tensile load interaction equation.

1. A bearing mode characterized by shear resistance from direct bearing of shear lugs and inset faceplate edges on concrete or grout augmented by shear resistance from confinement effects associated with tension anchors and external concurrent axial loads; and

2. A shear-friction mode such as defined in 11.7 of the Code.

The embedments first respond in the bearing mode and then progress into the shear-friction mode subsequent to formation of final fracture planes in the concrete in front of the shear lugs or base plate edge.

The bearing strength of single shear lugs bearing on concrete is defined in B.4.5. For multiple lugs, the shear strength should not exceed the shear strength between shear lugs as defined by a shear plane between the shear lugs as shown in Fig. RB.11.1 and a shear stress limited to $10\phi\sqrt{f_c'}$, with ϕ equal to 0.85.

The anchorage shear strength due to confinement can be taken as $\phi K_c(N_y - P_a)$, with ϕ equal to 0.85, where N_y is the yield strength of the tension anchors equal to $nA_{se}f_y$, and P_a is the factored external axial load on the anchorage. (P_a is positive for tension and negative for compression). This considers the effect of the tension anchors and external loads acting across the initial shear fracture planes (see Fig. RB.11.1). When P_a is negative, the provisions of Section 9.2.3 regarding use of load factors of 0.9 or zero must also be considered. The confinement coefficient K_c, given in Reference B.22, is as follows:

K_c = 1.6 for inset base plates without shear lugs, or for anchorage with multiple shear lugs of height h and spacing s (clear distance face-to-face between shear lugs) less than or equal to $0.13h\sqrt{f_c'}$; and

K_c = 1.8 for anchorage with a single shear lug located a distance h or greater from the front edge of the base plate, or with multiple shear lugs and a shear lug spacing s greater than $0.13h\sqrt{f_c'}$.

These values of confinement factor K_c are based on the analysis of test data. The different K_c values for plates with and without shear lugs primarily reflect the difference in initial shear-fracture location with respect to the
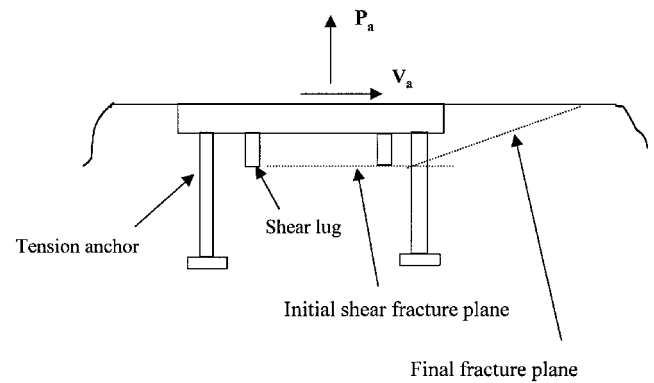

Fig. RB.11.1—Fracture planes for embedments with shear lugs.

tension anchors. The tests also show that the shear strength due to confinement is directly additive to the shear strength determined by bearing or by shear stress. The tension anchor steel area required to resist applied moments can also be utilized for determining N_s, providing that the compressive reaction from the applied moment acts across the potential shear plane in front of the shear lug.

For inset base plates, the area of the base plate edge in contact with the concrete can be used as an additional shear-lug-bearing area providing displacement compatibility with shear lugs can be demonstrated. This requirement can be satisfied by designing the shear lug to remain elastic under factored design loads with a displacement (shear plus flexure) less than 0.01 in.

For cases such as in grouted installations where the bottom of the base plate is above the surface of the concrete, the shear-lug-bearing area should be limited to the contact area below the plane defined by the concrete surface. This accounts for the potential extension of the initial shear fracture plane (formed by the shear lugs) beyond the perimeter of the base plate, that could diminish the effective bearing area.

Multiple shear lugs should be proportioned by considering relative shear stiffness. When multiple shear lugs are used near an edge, the effective stress area for the concrete design shear strength should be evaluated for the embedment shear at each shear lug.

RB.11.3—Shear strength of embedments with embedded base plates

The coefficient of 1.4 for embedments with shear lugs reflects concrete-to-concrete friction afforded by confinement of concrete between the shear lug(s) and the base plate (postbearing mode behavior). This value corresponds to the friction coefficient of 1.4 recommended in 11.7 of the Code for concrete-to-concrete friction, and is confirmed by tests discussed in Reference B.22.

RB.13—Comparison of Concrete Capacity Design Method and ACI 349-97

The following sections provide comparisons of the capacities of anchors in accordance with the Concrete Capacity Design Method (included in this edition of ACI
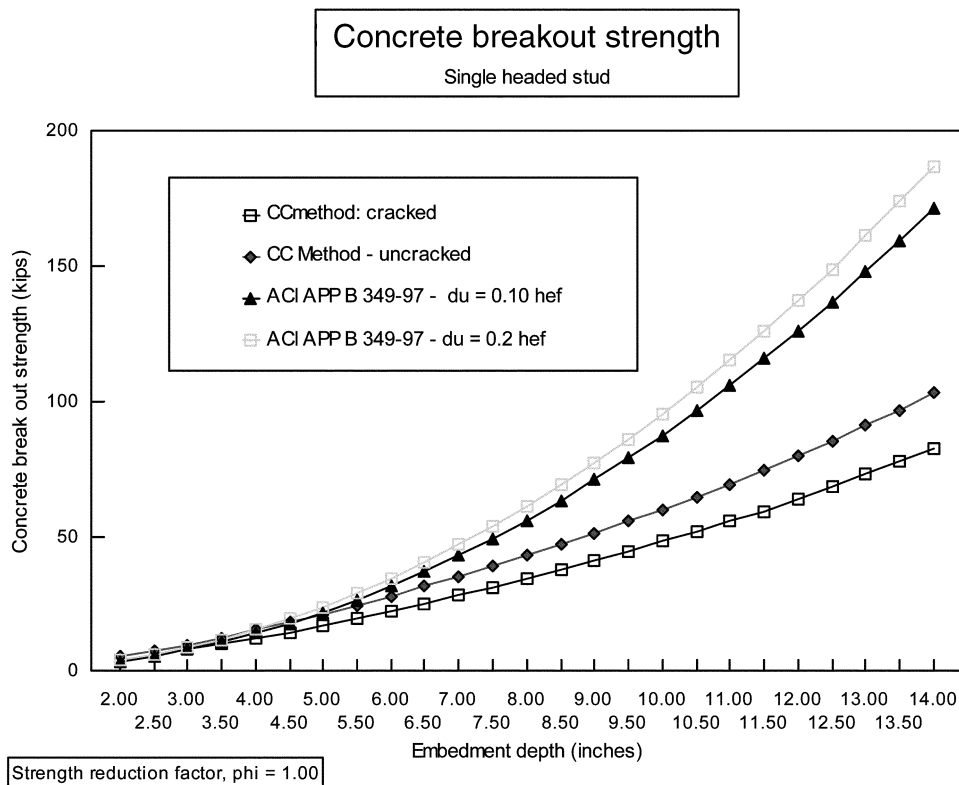
Concrete breakout strength
Single headed stud

Strength reduction factor, phi = 1.00

Fig. RB.13.1—Concrete breakout strength for single stud in tension.

Table RB.13.1—Concrete breakout strength of a single headed stud*

Concrete breakout nominal strength, kips			
Embedment depth	ACI 349-01		ACI 349-97
	Cracked $k = 24/16^\dagger$ $\psi_3 = 1.0$	Uncracked $k = 24/16^\dagger$ $\psi_3 = 1.25$	$d_u = 0.1h_{ef}$
4 in.	12.1	15.2	14.0
8 in.	34.3	42.9	56.0
12 in.	$63.6^\dagger$	$80.0^\dagger$	125.9
Concrete breakout design strength, kips			
Embedment depth	ACI 349-01		ACI 349-97
	$\phi = 0.85$	$\phi = 0.85$	$\phi = 0.65$	$\phi = 0.85$
4 in.	10.3	12.9	9.1	11.9
8 in.	29.2	36.5	36.4	47.6
12 in.	$54.1^\dagger$	$67.6^\dagger$	81.8	107.0

*Concrete strength = 4000 psi
†Strength for embedment depths of 4 and 8 in. is calculated using Eq. (B-6a); the strength for the embedment depth of 12 in. is calculated using Eq. (B-6b).

349) against those calculated in accordance with the previous provision of ACI 349 Appendix B (ACI 349-97).

RB.13.1—Concrete breakout strength of a single headed stud in tension

Figure RB.13.1 shows the concrete breakout strength of a single anchor in tension ($\Psi_3 N_b$) in concrete with a compressive strength of 4000 psi. The CCD Method in cracked concrete is from Eq. (B-6a) of the Code with $k = 24$ for a headed stud. This is increased by $\Psi_3 = 1.25$ for the strength of uncracked concrete. The ACI 349-97 strength is dependent on the head diameter and is shown for head diameters of the stud equal to 10 and 20% of the embedment depth.

Table RB.13.1 shows values from Fig. RB.13.1 for embedment depths of 4, 8, and 12 in. The table also shows the design strengths. For the CCD Method, the cracked and uncracked breakout strengths are multiplied by the strength reduction factor of 0.85 for cases where the potential concrete failure surfaces are crossed by supplementary reinforcement. The factor of 0.85 is also specified in ACI 349, paragraph B.4.4.1, when determining if an anchor is ductile. For ACI 349-97, design strengths are shown for strength reduction factors of 0.65 and 0.85 based on the requirements of paragraph B.4.2. The strength reduction factor of 0.85 is only applicable in areas of compression or low tension, and may be considered as uncracked. The strength reduction factor of 0.65 may be considered as applicable to cracked concrete.

The comparisons in Fig. RB.13.1 and Table RB.13.1 show a significant reduction in strength for larger embedment depths. This is due to the exponent on embedment depth and is discussed in Reference B.1. Committee 349 reviewed the test data and concluded that the exponent of 2 was unconservative. An exponent of 1.6 or 1.7 would be consistent with the test data. It was decided to use 1.5 for depths less than 11 in., and 1.67 for greater depths.

ACI 349-97 gives lower strengths for shallow embedments (up to a depth of about 5 in.) than the CCD Method. ACI 349-97 becomes progressively less conservative than the CCD Method as the embedment depth increases.

RB.13.2—Concrete breakout strength of a single expansion anchor in tension

The concrete breakout strength of a single expansion anchor in tension in uncracked concrete is about 20% lower than that of a headed stud ($k\Psi_3 = 17 \times 1.4 = 24$ versus
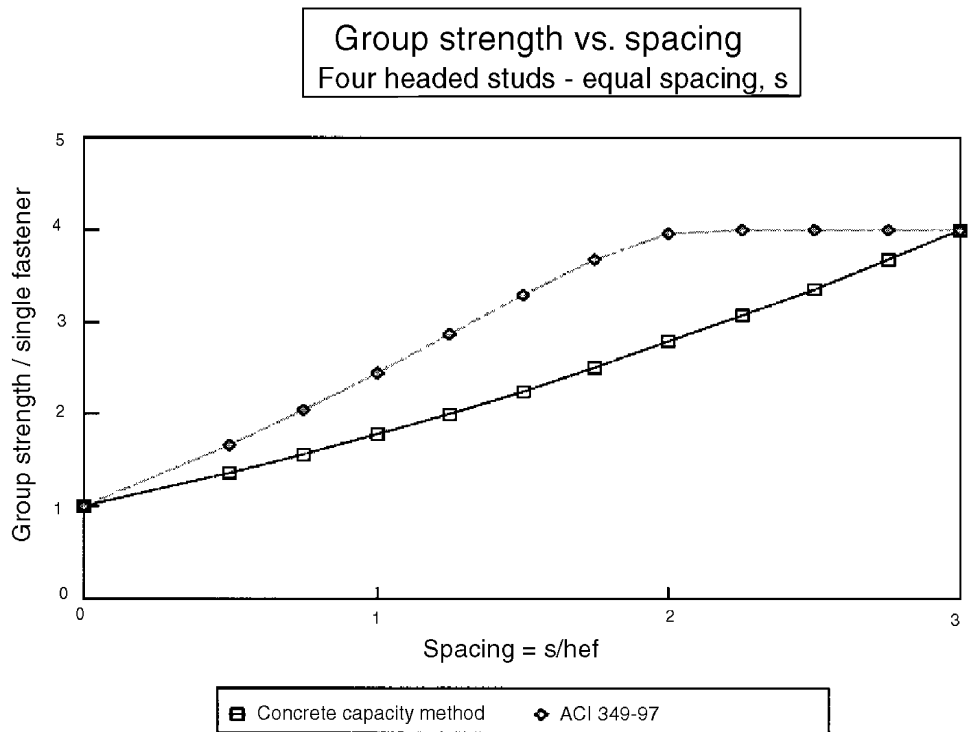
Group strength vs. spacing
Four headed studs - equal spacing, s

Fig. RB.13.2—Effect of group spacing on concrete breakout strength.

Effect of edge and corner distance
Single headed stud

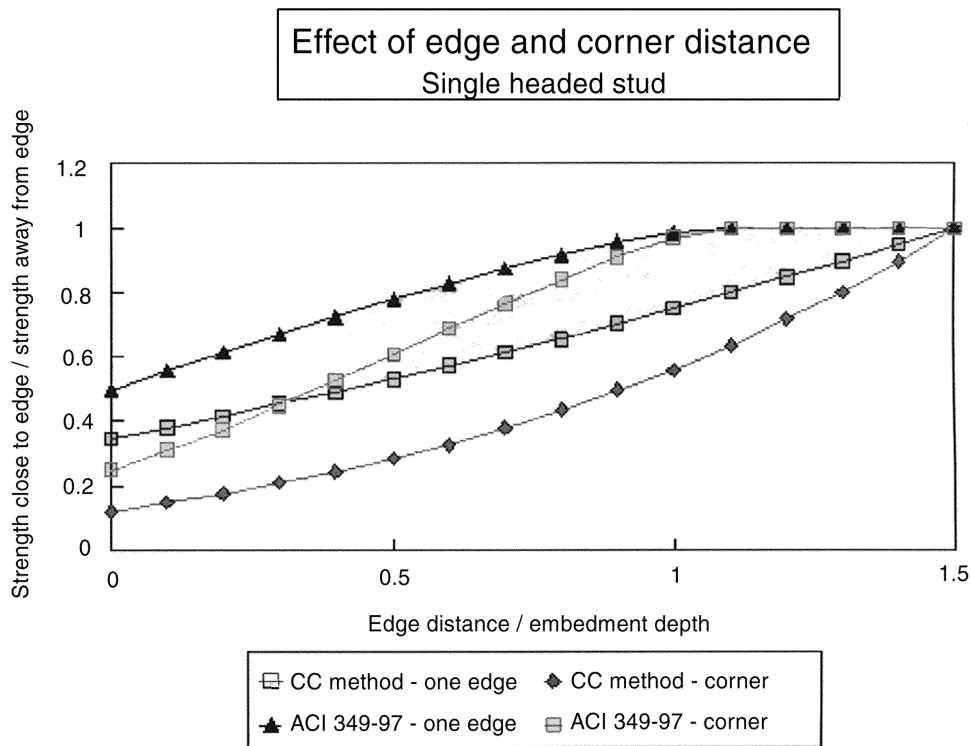*Fig. RB.13.3—Effect of edge and corner distance for single stud.*

24 × 1.25 = 30). In ACI 349-97, the difference was about 10% because the strength of headed studs included the diameter of the head. Test data show a larger reduction in strength for expansion anchors than for headed studs in cracked concrete.

The concrete breakout strength should be verified by the qualification tests for post-installed anchors. Undercut anchors generally perform better than other expansion anchors and may have the same concrete breakout strength as headed studs in both uncracked and cracked concrete.

RB.13.3—Concrete breakout strength of an anchor group

The breakout strength calculations in the CCD Method are based on a breakout prism angle of 35 degrees instead
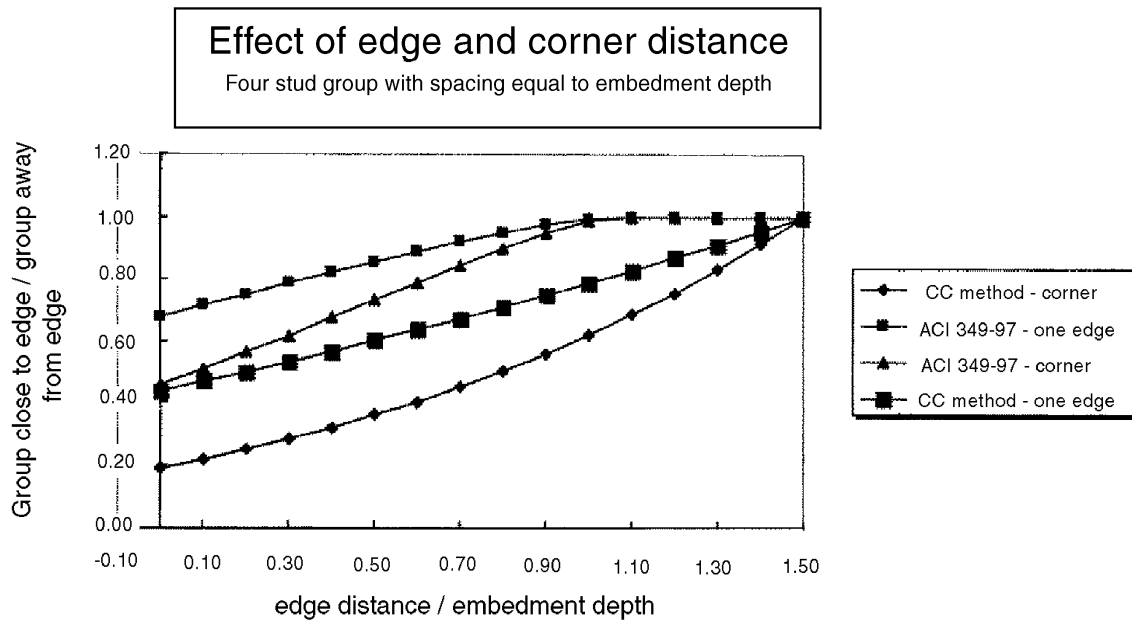
Fig. RB.13.4—Effect of edge and corner distance for four stud group.

of the 45 degree cone in ACI 349-97. Figure RB.13.2 shows the ratio of the concrete breakout strength of a group of four single headed studs at equal spacing in each direction to that of a single headed stud as a function of the anchor spacing (s/h_{ef}). For the CCD Method, the strength is affected when the spacing is less than 3 times the embedment depth; for ACI 349-97, the strength is affected when the spacing is less than twice the embedment depth plus head radius. The CCD Method reduces the strength by a maximum of about 30%.

RB.13.4—Concrete breakout strength of a single headed stud in tension close to an edge

Figure RB.13.3 shows the ratio of the concrete breakout strength of a headed stud close to an edge to that of a single headed stud away from the edge ($\mathbf{Y}_2 A_n/A_{no}$) as a function of the edge distance. This calculation uses the projected area of the 35 degree prism for the CCD Method, and of a 45 degree cone for ACI 349-97. The CCD Method has an additional reduction factor $\mathbf{Y}_2$ to adjust for the edge effect. Both methods require a separate evaluation for side blow-out for small edge distances. Figure RB.13.3 also shows similar ratios for the anchor close to a corner with edge distance C_{min} to two edges.

RB.13.5—Concrete breakout strength of an anchor group in tension close to an edge

Figure RB.13.4 shows the ratio of the concrete breakout strength of a group of four single headed studs close to an edge to that of the same anchor group away from the edge as a function of the edge distance C_{min}. The ratio is influenced by the spacing of the anchors, and this figure applies to four single headed studs with embedment depth of 6 in., spacing of 6 in., and head diameter of 0.6 in. The figure also shows similar ratios for the anchor group close to a corner with edge distance C_{min} to two edges.

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APPENDIX C—SPECIAL PROVISIONS FOR IMPULSIVE AND IMPACTIVE EFFECTS

RC.1—Scope

RC.1.2 While the provisions of this appendix apply to those structural elements directly affected by the impactive and impulsive loadings, vibratory effects at points away from the location of impact should also be considered.

RC.2—Dynamic strength increase

Because of the rapid strain rates that occur in structural elements under impactive or impulsive loading, both the concrete and reinforcing steel will exhibit strengths that are higher than those under static loading conditions.

The Dynamic Increase Factors (DIF) represent the ratio of dynamic to static yield stresses, or strengths, and are a direct function of the strain rates involved, as indicated in Table RC.1 and References C.1 and C.2.

DIF given above are based on tests with specified concrete strengths f_c' of 4000 to 6000 psi and may not be used for high-strength concrete.

RC.3—Deformation

RC.3.1 The ductility ratio is used in conjunction with total deformation consisting of both shear and flexural displacements.

RC.3.2 This section specifies a minimum structural strength for resisting certain impulsive loads whose time-dependence curve contains an interval, equal to or greater than the fundamental period of the structural element, during which the load is approximately constant. For example, referring to Fig. RC.1, the impulsive loading, which attains a maximum value F, has the approximately constant value F_2 during a time Δt, where Δt is equal to or greater than the fundamental period of the structural element. Let R_{m1} denote the resistance required by the impulsive loading with peak value F_1 that acts before the time interval Δt.

Section C.3.2 requires that the minimum available resistance for the impulsive load be that larger of the values R_{m1} and $R_{m2} = 1.2F_2$, and stipulates that this value is applicable to the load combinations which include impulsive loads in Chapter 9.

This section emphasizes by referencing Section C.8 that the calculation of available resistance or margin in a particular structural element should consider the strength required for other loads which may be acting concurrently with the impulsive load.

RC.3.3 This section defines the permissible ductility ratio of a concrete member in terms of the tension and compression reinforcement or as a function of the rotational capacity as defined in C.3.4. It should be noted that the compression reinforcement contributes to the ductility of a structural element, by enabling a large angle-change to take place before general crushing failure of the concrete occurs, thereby increasing the deflection which the structural element can undergo before collapse. The compression reinforcement is most effective in contributing to the ductility of beams when it is tied by stirrups to the tension reinforcement. However, in certain cases, the position of the neutral axis of a structural element may result in the so-called compression reinforcement being actually in tension when the section reaches its ultimate capacity. In such cases, the section should be evaluated to determine the effectiveness of the compressions reinforcement contribution to the ductility of the structural element.

The equation for ductility, $\mu_d = 0.05/(\rho - \rho')$ is based upon test data given in References C.3 and C.4 and is widely accepted in engineering practice. The coefficient of 0.05 was chosen instead of 0.1 given in Reference C.4 to provide an additional margin of safety against overestimating ductility. However, available data indicate that the 0.05 factor may be too conservative.[C.1,C.4,C.22]

When the permissible ductility ratio is defined as a function of the rotational capacity, the maximum acceptable displacement is established by calculating the displacement at ultimate, with an upper limit based on the rotational capacity specified in Section C.3.4.

Reference C.24 presents a rational method for obtaining a conservative estimate of the displacement at ultimate of a reinforced concrete slab subjected to a concentrated load.

Table RC-1—Dynamic increase factors

Material	Dynamic increase factor (DIF)[*]	But not more than
Reinforcing steel		
Grade 40	$1.1 + 0.0723 \, (\log SR + 3.3)$	1.20
Grade 50	$1.05 + 0.08 \quad (\log SR + 3.0)$	1.15
Grade 60	$1.0 + 0.02625 \, (\log SR + 5.9)$	1.10
Prestressing steel	1.0	—
Concrete		
Axial and flexural compression	$0.90 + 0.10 \quad (\log SR + 5.0)$	1.25
Shear	$[0.90 + 0.10 \quad (\log SR + 5.0)]^{1/2}$	1.10

[*]Where SR = strain rate, in./in./sec, and DIF ≥ 1.0.

Fig. RC.1—Typical impulsive transient force.

e_b = Eccentricity corresponding to the balanced strain conditions specified in Section 10.3.2.

P_b = Nominal axial load strength at balanced strain conditions specified in Section 10.3.2.

$\phi P_{n\,(max)}$ = Design axial load strength specified in Section 10.3.5.

μ_d = Ductility ratio for flexure specified in Section C.3.3 or C.3.4.

Fig. RC.2—Interaction diagram and ductility ratio variation.
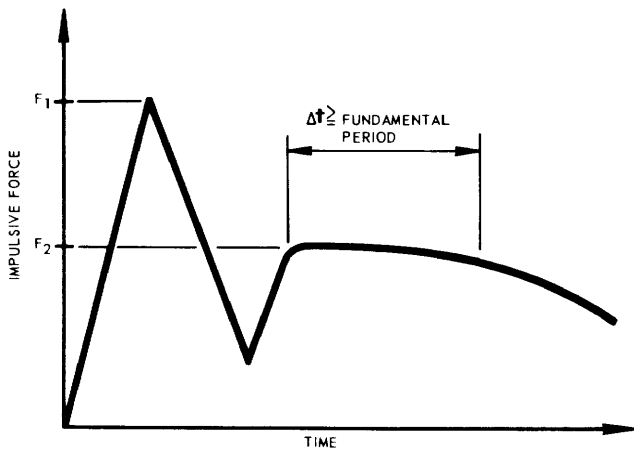
It is likely that the upper limit of 10 specified for the case when the permissible ductility ratios are established using the $\mu_d = 0.05/(r - r')$ equation is too restrictive for two-way slabs. Therefore, the Code permits the designer, in accordance with Section C.1.3, to use higher limits if sufficient justification can be provided.
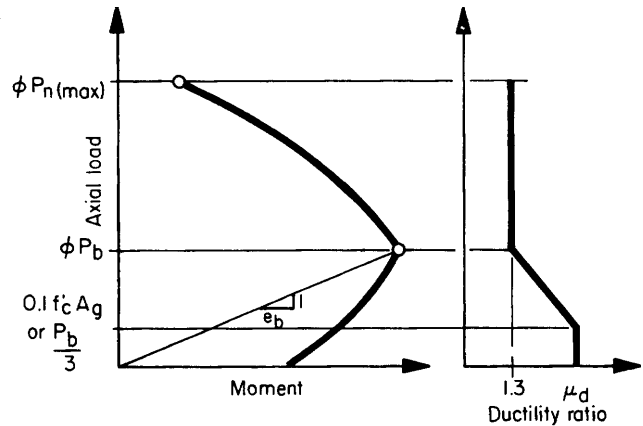
RC.3.4 The rotational capacity r_u of any yield hinge can be expressed by

$$r_u = \psi_u D_h \tag{R3.4.1}$$

in which the ultimate curvature ψ_u is given by

$$\psi_u = \frac{\varepsilon_u}{c} \tag{R3.4.2}$$

where ε_u is the ultimate compressive strain capacity of the concrete; c is the distance from the extreme compressive fiber to the neutral axis at ultimate strength; and D_h is the effective dimension of the plastic hinge zone. Reference C.5, based upon testing simply supported beams with concentrated loads, suggests that the ultimate concrete compressive strain be given by

$$\varepsilon_u = 0.003 + \frac{0.5}{z} \tag{R3.4.3}$$

and the effective plastic hinge zone dimension be given by

$$D_h = \frac{d}{2}\left[1 + \left(1.14\sqrt{\frac{z}{d}} - 1\right)\left(1 - \frac{q - q'}{q_b}\sqrt{\frac{d}{16.2}}\right)\right] \tag{R3.4.4}$$

where z is the span distance in inches from the point of maximum moment to zero moment; and d is the effective beam depth in inches. The steel reinforcement indexes are

$$q = \frac{\rho f_y}{f_c'}$$

$$q' = \frac{\rho' f_y}{f_c'}$$

and q_b = tensile reinforcement index for balanced ultimate strength conditions.

All the test data from which Eq. (3.4.3) and (3.4.4) were developed were obtained from beams with widths of 6 in., and depths of 10 and 20 in. Excessive conservatism may result from extrapolating these equations to beams with depths substantially greater that 20 in. since the terms in these equations are not all dimensionless.

For members designed in accordance with the provisions of this Code for impulsive or impactive loads, the reinforcement indexes are limited to

$$\frac{q - q'}{q_b} \le 0.5 \tag{3.4.5}$$

In this case, it can be shown[C.5] that within practical limits for z and d, the rotations obtained from Eq. (3.4.1) through (R3.4.4) can be conservatively estimated by

$$r_u = (0.0065)\frac{d}{c} \tag{3.4.6}$$

The ultimate rotation results reported in Reference C.5 for beams which satisfy Eq. (3.4.5) are conservatively estimated by Eq. (3.4.6). The ratio of test results to calculated results has a mean 0f 1.47 and a standard deviation of 0.49. Equation (3.4.6) generally yields rotations in the range from 0.025 to 0.075 radians (1.4 to 4.3 degrees) when applied to beams which satisfy the requirements of Eq. (3.4.5). Because of the lack of sufficient test data showing beam rotational capacities in excess of 0.07 radians (4 degrees) it is desirable to limit maximum rotations to this amount even under those circumstances where Eq. (3.4.6) may yield greater rotations.

RC.3.5 This section covers the special case of impulsive or impactive loads due to blast and compartment pressurization that could affect the integrity of the structure as a whole. Such loads may have a more significant overall effect than other impactive or impulsive loads defined in Sections C.1.4 and C.1.5. Therefore, the upper limit of ductility has been conservatively limited to 3.0 to minimize the permanent deformation due to these loads.

RC.3.6 The Code specifies that the load capacity in shear shall be at least 20% greater than the load capacity in flexure, to assure that flexure will control the behavior of the structural element subjected to impulsive or impactive loading. This requirement is based on the fact that the increase in strength under rapid strain exhibited by reinforcing bars is better established than that for shear strength of concrete.[C.1,C.2,C.4,C.23]

When considering the conservative limitations placed on the dynamic increase factors, the load capacity in flexure might be underestimated to a greater degree than the load capacity in shear.

Careful consideration should be given to special cases where the flexural behavior goes significantly past yield into the strain hardening range. In such cases, the margin

for load capacity in shear over the load capacity in flexure should preferably be higher than 20%.

RC.3.7 This section specifies the ductility ratios for reinforced concrete structures where diagonal or punching shear, rather than flexure, controls the design. A ductility ratio of 1.3 is specified for cases in which the shear is carried only by the concrete. The fact that a ductility ratio greater than 1.0 is permitted is based on the fact that even brittle structures[C.1] have some inelastic deformation capabilities.

This section allows the ductility ratio to be increased from 1.3 to 1.6, provided at least 20% of the shear load is carried by stirrups or bent bars, with the rest of the shear load being resisted by concrete.

RC.3.8 and **C.3.9** The ductility of a member at failure is more dependent on the mode of failure than on the type of loading. A compressive type of failure may occur in members such as columns which are subjected to either an axial load or axial load and bending moment. Under these conditions the mode of failure will be brittle. This is the case when failure is controlled by the compression region on the interaction diagram for columns (see Fig. RC.2). In this situation the provisions of Section 10.3.3 that limit the amount of flexural reinforcement are not applicable and the member can be over-reinforced. In such cases the permissible ductility ratio has been specified as 1.3 in accordance with Reference C.1.

When flexure controls the design, the ductility ratio is to be as specified in Sections C.3.3 or C.3.4. Section C.3.8b defines that a design with axial load less than or equal to $0.1f_c'A_g$, or 1/3 of that which produces balanced conditions can be considered a flexural failure. The limits of $0.1f_c'A_g$, or one-third that which produces balanced conditions, whichever is smaller, represent a magnitude of load below which axial effects on ductility are negligible.

RC.4—Requirements to assure ductility

The provisions to assure ductility are parallel to appropriate sections of Chapter 21 of ACI 318.

RC.5—Shear capacity

The shear capacity for concrete beams and columns is taken in accordance with Sections 11.1 and 11.5 of the main body of this Code, which were evaluated by ACI-ASCE Committee 326 on shear and diagonal tension[C.6] against an extensive body of test data and found to be satisfactory. These criteria are also invoked for walls and slabs where two-way action is not effective, also in accordance with ACI 318 practice. Examples would be checking of reaction shear at supported edges for slabs under local or distributed loads.

The shear capacity criteria for slabs and walls imply that potential failure could only occur either adjacent to the load or at the supported edge. The reference to Section 11.10 for punching shear criteria invokes the standard $4\sqrt{f_c'}$ limit taken from ACI 318. The $4\sqrt{f_c'}$ limit considers beneficial effects of two-way action and concurrent flexural stress to some extent. The punching shear criteria reference to Section 11.11 takes advantage of beneficial effect of net compression in walls in reducing principal
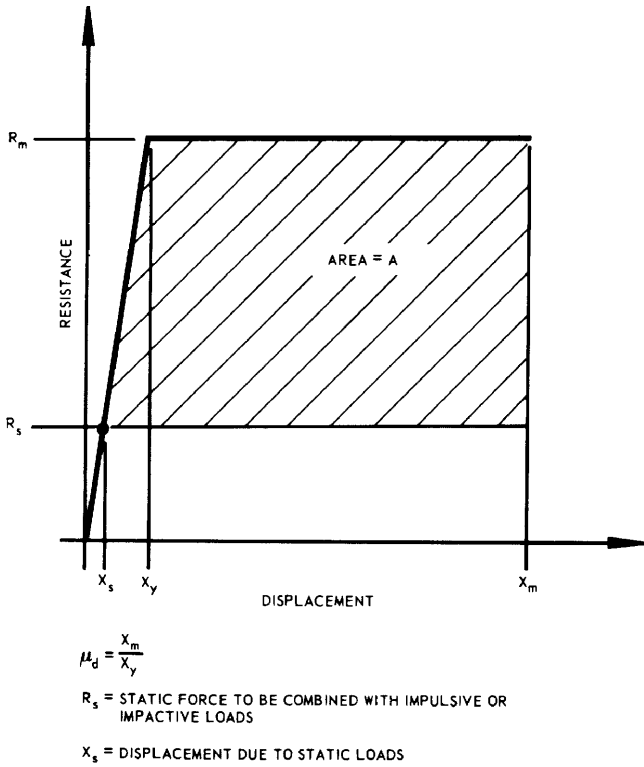
$$\mu_d = \frac{X_m}{X_y}$$

R_s = STATIC FORCE TO BE COMBINED WITH IMPULSIVE OR IMPACTIVE LOADS

X_s = DISPLACEMENT DUE TO STATIC LOADS

Fig. RC.3—Available resistance: idealized resistance-displacement curve.

(diagonal) tension. This criterion was taken from the nuclear containment code.[C.7]

The Code recognizes the possible conservatism of the punching shear equation contained in Section 11.11. Therefore, the provisions of Section C.1.3 allow substitution of alternate punching shear relationships for those specific situations where these alternate relationships can be shown to be applicable. For instance, a number of papers have been published recently (such as References C.8 and C.9) suggesting alternate punching shear relationships for two-way slabs based on percentage flexural reinforcement. In particular, Reference C.9 suggests that the punching shear capacity P_v be taken as the lesser of

$$P_v = \frac{\rho f_y d^2 \left(1 - 0.59 \frac{\rho f_y}{f_c'}\right)}{\left(0.2 - 0.9\frac{c}{L}\right)} \qquad (5.1)$$

or

$$P_v = \frac{20(c + d)d(100\rho)^{0.25}\sqrt{f_c'}}{0.75 + 4\frac{c}{L}} \qquad (5.2)$$

where r is the ratio of tensile steel reinforcement; d is the effective depth of tensile reinforcement from the compression face; c is the effective side dimension of the loaded zone as given by $c = \sqrt{A_p}$ where A_p is the area over which the P_v is applied; and L is the distance be-

tween local loads (may be taken as infinity in most impulse and impact cases).[C.9] For impactive and impulsive loads, the Dynamic Increase Factors (DIF) of Section C.2 should be used with Eq. (C.5.1) and (C.5.2) and the results of these equations should be reduced by the appropriate f factor. With these modifications, Eq. (C.5.1) and (C.5.2) can be substituted for the shear provisions of this Code for those specific situations where these relationships can be shown to be applicable.

RC.6—Impulsive effects

Three methods are identified as being acceptable for determination of structural response to impulsive loads. For the majority of cases encountered in design, application of these methods can be based on a single-degree-of-freedom (SDOF) representation of the structure. In the SDOF model, the distributed properties of the affected structural element are idealized in terms of an equivalent concentrated mass, load, and resistance-displacement function. Formulation and application of the SDOF methods is given in a number of references, such as References C.4, C.10, and C.11, and summarized briefly below.

The equivalent mass M_e, load F_e, and elastic stiffness K_e are determined on the basis of an assumed deformed shape function $\mathbf{f}(x, y)$ for the structure as follows

$$M_e = \iint m\phi^2(x,y)dxdy$$

$$F_e = \iint p\phi^2(x,y)dxdy$$

$$K_e = \frac{F_e}{F_t}K$$

where
m = mass per unit area
p = $p(x, y)$ = pressure
F_t = resultant force
K = value of F_t to cause unit deflection at point of application of resultant force

The $\mathbf{f}(x, y)$ function can generally be taken either as the fundamental mode shape or as the deformed shape had the load been applied statically. Exceptions may occur for very rapid transient or nonsymmetric loads, in which case higher mode response might predominate. The resistance-displacement function is an idealized bilinear curve characterized by elastic stiffness up to the static limit load and constant resistance thereafter (see Code Fig. C.3.1). Limit load may be determined by methods such as virtual work[C.12] or yield line theory.[C.13,C.14]

If significant deformation beyond the elastic limit is predicted, it is appropriate to assume $\mathbf{f}(x, y)$ as the shape of the collapse mechanism.

Given the parameters of the equivalent SDOF system, response can be predicted using one of the specified methods:

(a) Chart solutions such as those given in References C.4, C.10, and C.11 can be used to determine a dynamic load

factor, based Code ductility criteria, for common forms of transient load functions;

(b) For finite duration loads (as represented by the F_1 part of the loading as discussed in Section C.3.2 and as shown in Fig. RC.3.1) impulse can be equated with change in momentum to find the velocity of the structure, then velocity used to find kinetic energy, and finally kinetic energy equated to strain energy capacity required. Available strain energy capacity is that area under the resistance-displacement curve and within the Code ductility criteria; and

(c) For complex transient load functions, time history integration may be performed to predict response. Maximum permissible response is limited by Code ductility criteria.

In situations where the impulsive loads act on highly irregular structure configurations or nonuniform strength sections, the SDOF representation may not produce accurate results. For these cases, the time history dynamic analysis method is generally used with a multi-degree-of-freedom mathematical model of the structure. where the Code ductility criteria are used to permit deformation beyond elastic limits, nonlinear effects must be appropriately accounted for in the material models.

Impulsive loads must be combined with other loads in accordance with the load combinations and factors in Section 9.1. Strain energy capacity available to resist impulsive loads must be reduced by the amount of work done by other (factored) loads during deformation to maximum response.

RC.7—Impactive effects

Missile impactive loads cause both local effects and overall structural response of the impacted structure. Local effects consist of:

Penetration—Displacement of a missile into an impacted structural element. It is a measure of the depth of the crater formed at the zone of impact.

Perforation—The passing of a missile completely through the impacted structural element with or without exit velocity (that is, "full penetration").

Scabbing—Ejection of material from the back face of the impacted structural element opposite to the face of impact.

Spalling—Ejection of material from the front face of the impacted structural element (that is, the face on which the missile impacts).

Punching shear—Local shear failure occurring in the immediate vicinity of the impacted zone. A punching shear failure occurs as part of perforation.

These definitions are not universally used (for instance, *back face spalling* is sometimes used in lieu of *scabbing* to define the ejection of materials from the back face). However, the above definitions are consistently used in this Code.

If a structural element must act as a missile barrier then it is necessary that the element be sufficiently thick so as to prevent perforation and the provisions of Section C.7.2.1 must be met. However, if the structural element is not required to stop the missile and local perforation is permissible and does not impair the required function of the structural element, then the provisions of Section C.7.2.1 are not mandatory.

The provisions of Section C.7.2.1 do not preclude scabbing of concrete off the rear face of the structural element. these fragments of scabbed concrete become secondary missiles. With estimates of a spectrum of values for the masses of the fragments, the exit velocities can be calculated.[C.15] Although these concrete fragments will have exit velocities very much lower than the striking velocity of the impacting missile (so long as the wall thickness is greater than the perforation thickness), they might be damaging to fragile systems or equipment. In such a case, it is necessary to prevent scabbing by either: (1) attaching an adequately designed scab plate to the rear surface of the structural element, or (2) use of a wall thickness greater than that necessary to prevent scabbing.

A large number of empirical formulas exist for predicting the required concrete thicknesses to prevent perforation or scabbing. None of these formulas have yet been sufficiently verified or accepted to enable the Code Committee to specify a single formula and require its usage. At this time, the requirement is placed upon the designer to ensure that he is using an applicable formula or pertinent test data. Some tentative guidance concerning applicable formulas can be provided by the Code Committee. The Modified National Defense Research Council formulas,[C.16] the Bechtel formulas,[C.17] and the Stone and Webster formulas[C.18] appear to be in reasonable agreement with the available published pertinent test data[C.19-C.21] for perforation and scabbing thicknesses. Any of these formulas are tentatively recommended for usage for relatively nondeformable missiles. Other previously used formulas such as the Modified Petry, and the Modified Ballistic Research Laboratory Formulas (see Reference C.16 for discussion of these formulas) are not recommended for usage. For highly deformable missiles, usage of nondeformable missile impact formulas for calculating the required perforation or scabbing thicknesses may result in excessive conservatism and techniques have been suggested[C.15-C.17] for accounting for missile deformability.

Test data in the range of interest is rapidly becoming available.[C.18-C.21] However, sufficient data is not available to adequately define the degree of scatter on perforation or scabbing thickness. However, for higher missile velocities, the one standard deviation bounds are on the order of ±15 to 20%. Because of potential scatter of test data, and the degree of uncertainty that exists for currently available applicable formulas or pertinent test data, the Code requires that wall thicknesses be at least 20% greater than determined by an appropriate mean-centered formula or the mean of test data to prevent perforation or scabbing. This 20% factor is to account for uncertainty and is not considered to be an additional factor of safety. The factor of safety is contained in the selection of the impacting missile properties and velocity. The intent of the Code is to ensure that the concrete thickness be at least one standard deviation greater than the mean perforation or scabbing thickness. In those cases where the designer can show that he has met the intent of the Code with less than a 20% increase in thickness, then this Code provision for a 20% increase in thickness can be reduced. Inasmuch as missile test data are rapidly becoming available, values of minimum thickness are being established and receiving acceptance by industry and responsible regulatory

agencies. There would be no need to add 20% to such established thickness values determined for specifically defined impact conditions.

It should be noted that most of the test data were developed for missiles with relatively low mass and high impact velocity. In assessing the applicability of empirical formulae, the range of parameters used in the tests should be considered.

RC.8—Impactive and impulsive loads

In cases of impulsive and impactive loading where a structural element is expected to deform beyond its elastic limits, the usefulness of load combination equations presented in Chapter 9 is rather limited. These load combination equations do not provide any means of accounting for the additional work done by the static loads such as dead load, live load, etc., which may be present as the structural element deforms beyond its effective yield point (corresponding to X_y, Fig. RC.3).

If the energy balance method is used, only the energy represented by Area A. in Fig. RC.3 which is available to resist the impulsive and impactive loads should be used. Alternatively, if an elastoplastic analysis is performed, the effective ductility ratio to be used in the analysis for impactive and impulsive loading is given by

$$\mu' = \frac{X_m - X_s}{X_y - X_s} = \frac{\mu_d X_y - X_s}{X_y - X_s}$$

where μ_d is the permissible ductility ratio for the case being considered.

This effective ductility ratio is to be used in conjunction with effective available resistance equal to $R_m - R_s$.

In lieu of a more rigorous analysis, seismic forces can be conservatively treated as equivalent static loads in the analysis for determining the adequacy of the element for the impactive and impulsive loading.

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