Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06) and Commentary

Reported by ACI Committee 349

American Concrete Institute®
Code Requirements for Nuclear Safety-Related Concrete Structures
and Commentary

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American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
U.S.A.
Phone: 248-848-3700
Fax: 248-848-3701

www.concrete.org

Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06) and Commentary

An ACI Standard

Reported by ACI Committee 349

Ronald J. Janowiak
Chair

Hansraj G. Ashar
Ranjit L. Bandyopadhyay
Peter J. Carrato
Ronald A. Cook
Rolf Eligehausen
Werner Fuchs
Branko Galunic

Partha S. Ghosal
Herman L. Graves III
Orhan Gurbuz
James A. Hammell
Gunmar A. Harstead
Christopher Heinz
Charles J. Hookham

Jagadish R. Joshi
Richard E. Klingner
Nam-Ho Lee
Dan J. Naus
Dragos A. Nuta
Richard S. Orr

Bozidar Stojadinovic
Barendra K. Talukdar
Donald T. Ward
Andrew S. Whittaker
Albert Y. C. Wong
Charles A. Zalesiak

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

All notation sections have been removed from the beginning of each chapter and consolidated into one list in Chapter 2.

The format of this Code is based on the “Building Code Requirements for Structural Concrete (ACI 318-05)” and incorporates recent revisions of that standard.

The commentary, which is presented after the Code, discusses some of the considerations of ACI Committee 349 in developing “Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06).” This information is provided in the commentary because the Code is written as a legal document and therefore cannot present background details or suggestions for carrying out its requirements.

CODE

Chapter 1—General requirements, p. 349-6

1.1—Scope

1.2—Drawings and specifications

1.3—Inspection

1.4—Approval of special systems of design or construction

1.5—Quality assurance program

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11.11—Transfer of moments to columns
11.12—Special provisions for slabs and footings

Chapter 12—Development and splices of reinforcement, p. 349-51
12.1—Development of reinforcement—general
12.2—Development of deformed bars and deformed wire in tension
12.3—Development of deformed bars and deformed wire in compression
12.4—Development of bundled bars
12.5—Development of standard hooks in tension
12.6—Mechanical anchorage
12.7—Development of welded deformed wire reinforcement in tension
12.8—Development of welded plain wire reinforcement in tension
12.9—Development of prestressing strand
12.10—Development of flexural reinforcement—general
12.11—Development of positive moment reinforcement
12.12—Development of negative moment reinforcement
12.13—Development of web reinforcement
12.14—Splices of reinforcement—general
12.15—Splices of deformed bars and deformed wire in tension
12.16—Splices of deformed bars in compression
12.17—Special splice requirements for columns
12.18—Splices of welded deformed wire reinforcement in tension
12.19—Splices of welded plain wire reinforcement in tension

Chapter 13—Two-way slab systems, p. 349-56
13.1—Scope
13.2—Definitions
13.3—Slab reinforcement
13.4—Openings in slab systems
13.5—Design procedures
13.6—Direct design method
13.7—Equivalent frame method

Chapter 14—Walls, p. 349-62
14.1—Scope
14.2—General
14.3—Minimum reinforcement
14.4—Walls designed as compression members
14.5—Empirical design method
14.6—Nonbearing walls
14.7—Walls as grade beams
14.8—Alternative design of slender walls

Chapter 15—Footings, p. 349-63
15.1—Scope
15.2—Loads and reactions
15.3—Footings supporting circular or regular polygon-shaped columns or pedestals
15.4—Moment in footings
15.5—Shear in footings
15.6—Development of reinforcement in footings
15.7—Minimum footing depth
15.8—Transfer of force at base of column, wall, or reinforced pedestal
15.9—Sloped or stepped footings
15.10—Combined footings and mats

Chapter 16—Precast concrete, p. 349-65
16.1—Scope
16.2—General
16.3—Distribution of forces among members
16.4—Member design
16.5—Structural integrity
16.6—Connection and bearing design
16.7—Items embedded after concrete placement
16.8—Marking and identification
16.9—Handling
16.10—Strength evaluation of precast construction

Chapter 17—Composite concrete flexural members, p. 349-67
17.1—Scope
17.2—General
17.3—Shoring
17.4—Vertical shear strength
17.5—Horizontal shear strength
17.6—Ties for horizontal shear

Chapter 18—Prestressed concrete, p. 349-68
18.1—Scope
18.2—General
18.3—Design assumptions
18.4—Serviceability requirements—flexural members
18.5—Permissible stresses in prestressing steel
18.6—Loss of prestress
18.7—Flexural strength
18.8—Limits for reinforcement of flexural members
18.9—Minimum bonded reinforcement
18.10—Statically indeterminate structures
18.11—Compression members—combined flexure and axial loads
18.12—Slab systems
18.13—Post-tensioned tendon anchorage zones
18.14—Intentionally left blank
18.15—Intentionally left blank
18.16—Corrosion protection for unbonded tendons
18.17—Post-tensioning ducts
18.18—Grout for bonded tendons
18.19—Protection for prestressing steel
18.20—Application and measurement of prestressing force
18.21—Post-tensioning anchorages and couplers
18.22—External post-tensioning

Chapter 19—Shells, p. 349-73
19.1—Scope
19.2—General
19.3—Design strength of materials
19.4—Section design and reinforcement requirements
19.5—Construction
Chapter 20—Strength evaluation of existing structures, p. 349-74

20.1—Strength evaluation—general
20.2—Analytical investigations—general
20.3—Load tests—general
20.4—Load test procedure
20.5—Loading criteria
20.6—Acceptance criteria
20.7—Safety

Chapter 21—Provisions for seismic design, p. 349-75

21.1—Definitions
21.2—General requirements
21.3—Flexural members of moment frames
21.4—Moment frame members subjected to bending and axial load
21.5—Joints of moment frames
21.6—Intentionally left blank
21.7—Reinforced concrete structural walls and coupling beams
21.8—Intentionally left blank
21.9—Structural diaphragms and trusses
21.10—Foundations

APPENDIXES

Appendix A—Strut-and-tie models, p. 349-82
A.1—Definitions
A.2—Strut-and-tie model design procedure
A.3—Strength of struts
A.4—Strength of ties
A.5—Strength of nodal zones

Appendix B—Intentionally left blank, p. 349-84

Appendix C—Alternative load and strength-reduction factors, p. 349-84
C.1—General
C.2—Required strength
C.3—Design strength

Appendix D—Anchoring to concrete, p. 349-85
D.1—Definitions
D.2—Scope
D.3—General requirements
D.4—General requirements for strength of anchors
D.5—Design requirements for tensile loading
D.6—Design requirements for shear loading
D.7—Interaction of tensile and shear forces
D.8—Required edge distances, spacings, and thicknesses to preclude splitting failure
D.9—Installation of anchors
D.10—Structural plates, shapes, and specialty inserts
D.11—Shear strength of embedded plates and shear lugs
D.12—Grouted embedments

Appendix E—Thermal considerations, p. 349-92
E.1—Scope
E.2—Definitions
E.3—General design requirements
E.4—Concrete temperatures

Appendix F—Special provisions for impulsive and impactive effects, p. 349-93
F.1—Scope
F.2—Dynamic strength increase
F.3—Deformation
F.4—Requirements to assure ductility
F.5—Shear strength
F.6—Impulsive effects
F.7—Impactive effects
F.8—Impactive and impulsive loads

Appendix G—SI metric equivalents of U.S. Customary Units, p. 349-96

COMMENTARY

Introduction, p. 349-101

Chapter R1—General requirements, p. 349-101
R1.1—Scope
R1.2—Drawings and specifications
R1.3—Inspection
R1.4—Approval of special systems of design or construction
R1.5—Quality assurance program

Chapter R2—Notation and definitions, p. 349-102
R2.1—Commentary notation
R2.2—Definitions

Chapter R3—Materials, p. 349-102
R3.1—Tests of materials
R3.2—Cements
R3.3—Aggregates
R3.4—Water
R3.5—Steel reinforcement
R3.6—Admixtures
R3.7—Storage and identification of materials

Chapter R4—Durability requirements, p. 349-104
R4.2—Freezing and thawing exposures
R4.3—Sulfate exposures
R4.4—Corrosion protection of reinforcement

Chapter R5—Concrete quality, mixing, and placing, p. 349-105
R5.1—General
R5.3—Proportioning on the basis of field experience, or trial mixtures, or both
R5.4—Proportioning without field experience or trial mixtures
R5.6—Evaluation and acceptance of concrete
R5.7—Preparation of equipment and place of deposit
R5.9—Conveying
R5.10—Depositing
R5.11—Curing
R5.12—Cold weather requirements
R5.13—Hot weather requirements
Chapter R6—Formwork, embedded pipes, and construction joints, p. 349-108
  R6.1—Design of formwork
  R6.2—Removal of forms, shores, and reshoring
  R6.3—Conduits and pipes embedded in concrete
  R6.4—Construction joints

Chapter R7—Details of reinforcement, p. 349-108
  R7.4—Surface conditions of reinforcement
  R7.12—Minimum reinforcement
  R7.13—Requirements for structural integrity

Chapter R8—Analysis and design—general considerations, p. 349-109
  R8.2—Loading
  R8.3—Methods of analysis
  R8.5—Modulus of elasticity
  R8.11—Joist construction

Chapter R9—Strength and serviceability requirements, p. 349-109
  R9.1—General
  R9.2—Required strength
  R9.3—Design strength
  R9.4—Design strength for reinforcement
  R9.5—Control of deflections

Chapter R10—Flexure and axial loads, p. 349-113
  R10.6—Distribution of flexural reinforcement in beams and one-way slabs

Chapter R11—Shear and torsion, p. 349-113
  R11.12—Special provisions for slabs and footings

Chapter R12—Development and splices of reinforcement, p. 349-113
  R12.6—Mechanical anchorage
  R12.14—Splices of reinforcement—general
  R12.15—Splices of deformed bars and deformed wire in tension

Chapter R13—Two-way slab systems, p. 349-114

Chapter R14—Walls, p. 349-114
  R14.3—Minimum reinforcement

Chapter R15—Footings, p. 349-114

Chapter R16—Precast concrete, p. 349-114

Chapter R17—Composite concrete flexural members, p. 349-114

Chapter R18—Prestressed concrete, p. 349-114

Chapter R19—Shells, p. 349-114
  R19.1—Scope
  R19.2—General
  R19.4—Section design and reinforcement requirements

Chapter 20—Strength evaluation of existing structures, p. 349-115
  R20.1—Strength evaluation—general
  R20.2—Analytical investigations—general
  R20.3—Load tests—general
  R20.4—Load test procedure
  R20.5—Loading criteria
  R20.6—Acceptance criteria

Chapter R21—Provisions for seismic design, p. 349-116
  R21.1—Definitions
  R21.2—General requirements
  R21.3—Flexural members of moment frames
  R21.4—Moment frame members subjected to bending and axial load
  R21.5—Joints of moment frames
  R21.6—Intentionally left blank
  R21.7—Reinforced concrete structural walls and coupling beams
  R21.8—Intentionally left blank
  R21.9—Structural diaphragms and trusses
  R21.10—Foundations

APPENDIXES
Appendix RA—Strut-and-tie models, p. 349-127

Appendix RB—Intentionally left blank, p. 349-127

Appendix RC—Alternative load and design strength-reduction factors, p. 349-127
  RC.1—General
  RC.2—Required strength
  RC.3—Design strength

Appendix RD—Anchoring to concrete, p. 349-128
  RD.1—Definitions
  RD.2—Scope
  RD.3—General requirements
  RD.4—General requirements for strength of anchors
  RD.5—Design requirements for tensile loading
  RD.6—Design requirements for shear loading
  RD.7—Interaction of tensile and shear forces
  RD.8—Required edge distances, spacings, and thicknesses to preclude splitting failure
  RD.9—Installation of anchors
  RD.10—Structural plates, shapes, and specialty inserts
  RD.11—Shear strength of embedded plates and shear lugs

Appendix RE—Thermal considerations, p. 349-141
  RE.1—Scope
  RE.2—Definitions
  RE.3—General design requirements
  RE.4—Concrete temperatures

Appendix RF—Special provisions for impulsive and impactive effects, p. 349-144
  RF.1—Scope
  RF.2—Dynamic strength increase
  RF.3—Deformation
CODE

CHAPTER 1—GENERAL REQUIREMENTS

1.1—Scope

1.1.1 This Code provides minimum requirements for design and construction of nuclear safety-related concrete structures and structural members for nuclear power generating stations. Safety-related structures and structural members subject to this standard are those concrete structures that 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 359).

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.

For structural concrete, $f'_{c}$ shall not be less than 2500 psi, unless otherwise specified.

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 wherever this Code is in conflict with the specific provisions of the authority having jurisdiction (AHJ).

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 Intentionally left blank.

1.1.6 Intentionally left blank.

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 Chapters 1 through 7 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 and specifications

1.2.1 Copies of design drawings, typical details, and specifications for all structural concrete construction shall bear the seal of 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 design conforms;

(b) Live load and other loads used in design;

(c) Specified compressive strength of concrete at stated ages or stages of construction for which each part of structure is designed;

(d) Specified strength or grade of reinforcement;

(e) Size and location of all structural members, reinforcement, and anchors;

(f) Provision for dimensional changes resulting from creep, shrinkage, and temperature;

(g) Magnitude and location of prestressing forces;

(h) Anchorage length of reinforcement and location and length of lap splices;

(i) Type and location of mechanical and welded splices of reinforcement;

(j) Details and location of all contraction or isolation joints;

(k) Minimum concrete compressive strength at time of post tensioning;

(l) Stressing sequence for post-tensioning tendons;

(m) Statement if slab-on-ground is designed as a structural diaphragm, see 21.10.3.4.

1.2.2 Calculations pertinent to design and the basis of design (including the results of model analysis, if any) shall be retained by the owner or his 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 instead of calculations. Model analysis shall be permitted to supplement calculations.

1.3—Inspection

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, reinforcing steel, and anchor installation;

(b) Concrete cylinder test reports and charts;
(c) Concrete design mixture 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 splice 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 tendons;
(m) Quality and proportions of concrete materials; and
(n) Any significant construction loadings on completed floors, members, or walls.

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 AHJ for review and approval. The AHJ may investigate the data so submitted, and may require tests and formulate rules governing 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 before 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, and Title 10 of the Code of Federal Regulations, Part 830, Subpart A.

CHAPTER 2—NOTATION AND DEFINITIONS
2.1—Code notation
The terms in this list are used in the Code and as needed in the commentary.

\[ A_{brg} = \text{bearing area of the head of stud or anchor bolt, in.}^2, \text{ Appendix D} \]
\[ A_c = \text{area of concrete section resisting shear transfer, in.}^2, \text{ Chapter 11} \]
\[ A_{cf} = \text{larger gross cross-sectional area of the slab-beam strips of the two orthogonal equivalent frames intersecting at a column of a two-way slab, in.}^2, \text{ Chapter 18} \]
\[ A_{ch} = \text{cross-sectional area of a structural member measured out-to-out of transverse reinforcement, in.}^2, \text{ Chapters 10, 21} \]
\[ A_{cp} = \text{area enclosed by outside perimeter of concrete cross section, in.}^2, \text{ see 11.6.1, Chapter 11} \]
\[ A_{cs} = \text{cross-sectional area at one end of a strut in a strut-and-tie model, taken perpendicular to the axis of the strut, in.}^2, \text{ Appendix A} \]
\[ A_{ct} = \text{area of that part of cross section between the flexural tension face and center of gravity of gross section, in.}^2, \text{ Chapter 18} \]
\[ A_{cv} = \text{gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, in.}^2, \text{ Chapter 21} \]
\[ A_{cw} = \text{area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear, in.}^2, \text{ Chapter 21} \]
\[ A_f = \text{area of reinforcement in bracket or corbel resisting factored moment, in.}^2, \text{ see 11.9, Chapter 11} \]
\[ A_g = \text{gross area of concrete section, in.}^2, \text{ for a hollow section, } A_g \text{ is the area of the concrete only and does not include the area of the void(s), see 11.6.1, Chapters 9-11, 14-16, 21, Appendix F} \]
\[ A_h = \text{total area of shear reinforcement parallel to primary tension reinforcement in a corbel or bracket, in.}^2, \text{ see 11.9, Chapter 11} \]
\[ A_j = \text{effective cross-sectional area within a joint in a plane parallel to plane of reinforcement generating shear in the joint, in.}^2, \text{ see 21.5.3.1, Chapter 21} \]
\[ A_k = \text{total area of longitudinal reinforcement to resist torsion, in.}^2, \text{ Chapter 11} \]
\[ A_{k,min} = \text{minimum area of longitudinal reinforcement to resist torsion, in.}^2, \text{ see 11.6.5.3, Chapter 11} \]
\[ A_{Nc} = \text{projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension, in.}^2, \text{ see D.5.2.1, Appendix D} \]
\[ A_{Nco} = \text{projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing, in.}^2, \text{ see D.5.2.1, Appendix D} \]
\[ A_p = \text{area of reinforcement in bracket or corbel resisting tensile force } N_{uc}, \text{ in.}^2, \text{ see 11.9, Chapter 11} \]
\[ A_{pz} = \text{area of a face of a nodal zone or a section through a nodal zone, in.}^2, \text{ Appendix A} \]
\[ A_{ph} = \text{gross area enclosed by shear flow path, in.}^2, \text{ Chapter 11} \]
\[ A_{oh} = \text{area enclosed by centerline of the outermost closed transverse torsional reinforcement, in.}^2, \text{ Chapter 11} \]
\[ A_{ps} = \text{area of prestressing steel in flexural tension zone, in.}^2, \text{ Chapter 18} \]
\[ A_s = \text{area of nonprestressed longitudinal tension reinforcement, in.}^2, \text{Chapters 10-12, 14, 15, 18} \]
\[ A'_s = \text{area of longitudinal compression reinforcement, in.}^2, \text{Appendix A} \]
\[ A_{sc} = \text{area of primary tension reinforcement in a corbel or bracket, in.}^2, \text{see 11.9.3.5, Chapter 11} \]
\[ A_{se} = \text{effective cross-sectional area of anchor, in.}^2, \text{Appendix A} \]
\[ A_{sh} = \text{total cross-sectional area of transverse reinforcement (including crossstresses) within spacing s and perpendicular to dimension } b_c, \text{in.}^2, \text{Chapter 21} \]
\[ A_{si} = \text{total area of surface reinforcement at spacing } S_i \text{ in the } i\text{-th layer crossing a strut, with reinforcement at an angle } \alpha_i \text{ to the axis of the strut, in.}^2, \text{Appendix A} \]
\[ A_{s,\text{min}} = \text{minimum area of flexural reinforcement, in.}^2, \text{see 10.5, Chapter 10} \]
\[ A_{s,\text{min}}' = \text{minimum reinforcement for concrete section having a thickness of 48 in. or more, see 7.12.3, Chapter 7} \]
\[ A_{sl} = \text{total area of nonprestressed longitudinal reinforcement (bars or steel shapes), in.}^2, \text{Chapters 10, 21} \]
\[ A_{sr} = \text{area of structural steel shape, pipe, or tubing in a composite section, in.}^2, \text{Chapter 10} \]
\[ A_t = \text{area of one leg of a closed stirrup resisting torsion within spacing } s, \text{in.}^2, \text{Chapter 11} \]
\[ A_{tp} = \text{total cross-sectional area of all transverse reinforcement within spacing } s \text{ that crosses the potential plane of splitting through the reinforcement being developed, in.}^2, \text{Chapter 12} \]
\[ A_{ts} = \text{area of nonprestressed reinforcement in a tie, in.}^2, \text{Appendix A} \]
\[ A_{VC} = \text{projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear, in.}^2, \text{see D.6.2.1, Appendix D} \]
\[ A_{Vco} = \text{projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness, in.}^2, \text{see D.6.2.1, Appendix D} \]
\[ A_v = \text{area of shear reinforcement spacing } s, \text{in.}^2, \text{Chapters 11, 17} \]
\[ A_{vd} = \text{total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam, in.}^2, \text{Chapter 21} \]
\[ A_{vf} = \text{area of shear-friction reinforcement, in.}^2, \text{Chapter 11} \]
\[ A_{vh} = \text{area of shear reinforcement parallel to flexural tension reinforcement within spacing } s_2, \text{in.}^2, \text{Chapter 11} \]
\[ A_{\text{v, min}} = \text{minimum area of shear reinforcement within spacing } s, \text{in.}^2, \text{see 11.5.6.3 and 11.5.6.4, Chapter 11} \]
\[ a = \text{depth of equivalent rectangular stress block as defined in 10.2.7.1, in., Chapter 10} \]
\[ a_v = \text{shear span, equal to distance from center of concentrated load to either } (\text{a}) \text{ face of support for continuous or cantilevered members, or } (\text{b}) \text{ center of support for simply supported members, in.,} \]
\[ b = \text{width of compression face of member, in., Chapter 10} \]
\[ b_1 = \text{dimension of the critical section } b_o \text{ measured in the direction of the span for which moments are determined, in., Chapter 13} \]
\[ b_1' = \text{total length of that portion of perimeter } b_o \text{ for which } V_{c1} \text{ is computed, in., Chapter 11} \]
\[ b_2 = \text{dimension of the critical section } b_o \text{ measured in the direction perpendicular to } b_1, \text{in., Chapter 13} \]
\[ b_2' = \text{total length of that portion of perimeter } b_o \text{ for which } V_{c2} \text{ is computed, in., Chapter 11} \]
\[ b_c = \text{cross-sectional dimension of column core measured center-to-center of outer legs of the transverse reinforcement comprising area } A_{sh}, \text{in., Chapter 21} \]
\[ b_o = \text{perimeter of critical section for shear in slabs and footings, in., see 11.12.1.2, Chapter 11} \]
\[ b_s = \text{width of strut, in., Appendix A} \]
\[ b_t = \text{width of that part of cross section containing the closed stirrups resisting torsion, in., Chapter 11} \]
\[ b_v = \text{width of cross section at contact surface being investigated for horizontal shear, in., Chapter 17} \]
\[ b_w = \text{web width, or diameter of circular section, in., Chapters 10-12, 21} \]
\[ C = \text{cross-sectional constant to define torsional properties of slab and beam, see 13.6.4.2, Chapter 13} \]
\[ C_{cr} = \text{rated capacity of crane including the maximum wheel loads of the crane and the vertical, lateral, and longitudinal forces induced by the moving crane, including impact factor, where appropriate, Chapter 9, Appendix C} \]
\[ C_F = \text{the compressive resultant force between the embedment and the concrete resulting from factored moment and factored axial load applied to the embedment, lb, Appendix D} \]
\[ C_m = \text{factor relating actual moment diagram to an equivalent uniform moment diagram, Chapter 10} \]
\[ c = \text{distance from extreme compression fiber to neutral axis, in., Chapters 9, 10, 11, 14, 21, Appendix F} \]
\[ c_1 = \text{dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, in., Chapters 11, 13, 21} \]
\[ c_2 = \text{dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to } c_1, \text{in., Chapter 13} \]
\[ c_{a1} = \text{distance from the center of an anchor shaft to the edge of concrete in one direction, in. If shear is applied to anchor, } c_{a1} \text{ is taken in the direction of the applied shear. If the tension is applied to the anchor, } c_{a1} \text{ is the minimum edge distance, Appendix D} \]
\[ c_{a2} = \text{distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to } c_{a1}, \text{in., Appendix D} \]
\[ c_{ac} = \text{critical edge distance required to develop the basic concrete breakout strength of a post-} \]
installed anchor in uncracked concrete without supplementary reinforcement to control splitting, in., see D.8.6, Appendix D

c_{a,max} = maximum distance from center of an anchor shaft to the edge of concrete, in., Appendix D

c_{a,min} = minimum distance from center of an anchor shaft to the edge of concrete, in., Appendix D

c_b = smaller of (a) the distance from center of a bar or wire to nearest concrete surface, and (b) one-half the center-to-center spacing of bars or wires being developed, in., Chapter 12

c_c = clear cover of reinforcement, in., see 10.6.4, Chapter 10

c_t = distance from the interior face of the column to the slab edge measured parallel to c_t, but not exceeding c_t, in., Chapter 21

D = dead loads, or related internal moments and forces, including piping, equipment, partitions, and dead loads of the crane components, Chapters 9, 20, Appendix C

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in., Chapters 7, 9-12, 14, 17, 18, 21, Appendix F

d' = distance from extreme compression fiber to centroid of longitudinal compression reinforcement, in., Chapters 9, 18

d_b = nominal diameter of bar, wire, or prestressing strand, in., Chapters 7, 12, 21

d_o = outside diameter of anchor or shaft diameter of headed stud or headed bolt, in., see D.8.4, Appendix D

d_p = distance from extreme compression fiber to centroid of prestressing steel, in., Chapters 11,18

d_pile = diameter of pile at footing base, in., Chapter 15

d_t = distance from extreme compression fiber to centroid of extreme layer of longitudinal tension fiber, in., Chapter 9

E = load effects of earthquake, or related internal moments and forces, Chapter 21

E_c = modulus of elasticity of concrete, psi, see 8.5.1, Chapters 8-10, 14

E_{cb} = modulus of elasticity of beam concrete, psi, Chapter 13

E_{cs} = modulus of elasticity of slab concrete, psi, Chapter 13

EI = flexural stiffness of compression member, in.²-lb, see 10.12.3, Chapter 10

E_o = load effects of operating basis earthquake (see Chapter 2) or related internal moments and forces, including OBE-induced piping and equipment reactions, Chapter 9, Appendix C

E_p = modulus of elasticity of prestressing steel, psi, see 8.5.3, Chapter 8

E_s = modulus of elasticity of reinforcement and structural steel, psi, see 8.5.2, Chapters 8, 10, 14

E_{ss} = load effects of safe shutdown earthquake (see Chapter 2) or related internal moments and forces, including SSE-induced piping and equipment reactions, Chapter 9, Appendix C

\( e \) = base of Napierian logarithms, Chapter 18

\( e_{Ny} \) = distance between resultant tension load on a group of anchors loaded in tension and the centroid of the group of anchors loaded in tension, in.; \( e_{Ny} \) is always positive, Appendix D

\( e'_y \) = distance between resultant shear load on a group of anchors loaded in shear in the same direction, and the centroid of the group of anchors loaded in shear in the same direction, in.; \( e'_y \) is always positive, Appendix D

F = loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces, Chapter 9, Appendix C

\( F_n \) = nominal strength of a strut, tie, or nodal zone, lb, Appendix A

\( F_{nn} \) = nominal strength at face of a nodal zone, lb, Appendix A

\( F_{ns} \) = nominal strength of a strut, lb, Appendix A

\( F_{nt} \) = nominal strength of a tie, lb, Appendix A

\( F_u \) = factored force acting in a strut, tie, bearing area, or nodal zone in a strut-and-tie model, lb, Appendix A

\( f'_c \) = specified compressive strength of concrete, psi, Chapters 4, 5, 8-12, 14, 18, 19, 21, Appendixes A, D, F

\( \sqrt{f'_c} \) = square root of specified compressive strength of concrete, psi, Chapters 8, 9, 11, 12, 18, 21, Appendix D

\( f_{ce} \) = effective compressive strength of the concrete in a strut or a nodal zone, psi, Appendix A

\( f'_{ci} \) = specified compressive strength of concrete at time of initial prestress, psi, Chapters 7, 18

\( \sqrt{f'_{ci}} \) = square root of specified compressive strength of concrete at time of initial prestress, psi, Chapter 18

\( f'_{cr} \) = required average compressive strength of concrete used as the basis for selection of concrete proportions, psi, Chapter 5

\( f_d \) = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, psi, Chapter 11

\( f_{dc} \) = decompression stress; stress in the prestressing steel when stress is zero in the concrete at the same level as the centroid of the prestressing steel, psi, Chapter 18

\( f_{m1} \) = concrete membrane stress acting along length \( b'_1 \), to be taken as positive for compression and negative for tension, psi, Chapter 11

\( f_{m2} \) = concrete membrane stress acting along length \( b'_2 \), to be taken as positive for compression and negative for tension, psi, Chapter 11

\( f_{pc} \) = 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 the resultant compressive stress at centroid of composite section, or at junction of web and
flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone), Chapter 11

\( f_{ps} \) = 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, Chapter 11

\( f_{ps} \) = stress in pre stressing steel at nominal flexural strength, psi, Chapters 12, 18

\( f_{pu} \) = specified tensile strength of pre stressing steel, psi, Chapters 11, 18

\( f_{py} \) = specified yield strength of pre stressing steel, psi, Chapter 18

\( f_r \) = modulus of rupture of concrete, psi, see 9.5.2.3, Chapters 9, 14, 18

\( f_s \) = calculated tensile stress in reinforcement at sustained loads, psi, Chapters 10, 18

\( f_s' \) = stress in compression reinforcement under factored loads, psi, Appendix A

\( f_s'' \) = stress in reinforcing steel, psi, Chapter 7

\( f_{se} \) = effective stress in pre stressing steel (after allowance for all prestress losses), psi, Chapters 12, 18, Appendix A

\( f_t \) = extreme fiber stress in tension in the precompressed tensile zone calculated at sustained loads using gross section properties, psi, see 18.3.3, Chapter 18

\( f_{tsa} \) = specified tensile strength of anchor steel, psi, Appendix D

\( f_y \) = specified yield strength of reinforcement, psi, Chapters 3, 7, 9-12, 14, 17, 18, 21, Appendixes A-C, F

\( f_{ya} \) = specified yield strength of anchor steel, psi, Appendix D

\( f_{yt} \) = specified yield strength of transverse reinforcement, psi, Chapters 9, 12, 21

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

\( h \) = overall thickness or height of member, in., Chapters 9-14, 17, 18, 20, 21, Appendixes A, F

\( h_a \) = thickness of member in which an anchor is located, measured parallel to anchor axis, in., Appendix D

\( h_{ef} \) = effective embedment depth of anchor, in., see D.8.5, Appendix D

\( h_v \) = depth of shearhead cross section, in., Chapter 11

\( h_w \) = height of entire wall from base to top or height of the segment of wall considered, in., Chapters 11, 21

\( h_x \) = maximum center-to-center horizontal spacing of crossties or hoop legs on all faces of the column, in., Chapter 21

\( I \) = moment of inertia of section about centroidal axis, in.\(^4\), Chapters 10, 11

\( I_e \) = moment of inertia of gross section of beam about centroidal axis, in.\(^4\), see 13.2.4, Chapter 13

\( I_{cp} \) = coefficient for pryout strength, Appendix D

\( I_{e} \) = load-bearing length of anchor for shear, in., see D.6.2.2, Appendix D

\( I_{cr} \) = moment of inertia of cracked section transformed to concrete, in.\(^4\), Chapters 9, 14, Appendix D

\( I_{e} \) = effective moment of inertia for computation of deflection, in.\(^4\), see 9.5.2.3, Chapters 9, 14

\( I_{g} \) = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.\(^4\), Chapters 9, 10, Appendix F

\( I_{s} \) = moment of inertia of gross section of slab about centroidal axis defined for calculating \( \alpha_f \) and \( \beta_f \), in.\(^4\), Chapter 13

\( I_{se} \) = moment of inertia of reinforcement about centroidal axis of member cross section, in.\(^4\), Chapter 10

\( I_{sx} \) = moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross section, in.\(^4\), Chapter 10

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

\( K_{hp} \) = transverse reinforcement index, see 12.2.3, Chapter 12

\( k \) = effective length factor for compression members, Chapters 10, 14

\( k_c \) = coefficient for basic concrete breakout strength in tension, Appendix D

\( L \) = live loads, or related internal moments and forces, due to occupancy and moveable equipment, Chapters 9, 20, Appendix C

\( L_r \) = roof live load, or related internal moments and forces, Chapter 9, Appendix C

\( l \) = span length of beam or one-way slab; clear projection of cantilever, in., see 8.7, Chapter 9

\( l_1 \) = length of span in direction that moments are being determined, measured center-to-center of supports, in., Chapter 13

\( l_2 \) = length of span in direction perpendicular to \( l_1 \), measured center-to-center of supports, in., see 13.6.2.3 and 13.6.2.4, Chapter 13

\( l_a \) = additional embedment length beyond centerline of support or point of inflection, in., Chapter 12

\( l_c \) = length of compression member in a frame, measured center-to-center of the joints in the frame, in., Chapters 10, 14

\( l_d \) = development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, or pretensioned strand, in., Chapters 7, 12, 19, 21

\( l_{dc} \) = development length in compression of deformed bars and deformed wire, in., Chapter 12

\( l_{dh} \) = development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter), in., see 12.5 and 21.5.4, Chapters 12, 21

\( l_e \) = development length of beam or one-way slab; clear projection of cantilever, in., see 8.7, Chapter 9
\( \ell_h \) = maximum unsupported length of rectangular hoop measured between perpendicular legs of the hoop or supplementary crossties, in., Appendix F

\( \ell_n \) = length of clear span measured face-to-face of supports, in., Chapters 8-11, 13, 16, 18, 21

\( \ell_o \) = length, measured from joint face along axis of structural member, over which special transverse reinforcement must be provided, in., Chapter 21

\( \ell_{px} \) = distance from jacking end of prestressing steel element to point under consideration, ft, see 18.6.2, Chapter 18

\( \ell_t \) = span of member under load test, taken as the shorter span for two-way slab systems, in. Span is the smaller of (a) distance between centers of supports, and (b) clear distance between supports plus thickness \( h \) of member. Span for a cantilever shall be taken as twice the distance from face of support to cantilever end, Chapter 20

\( \ell_u \) = unsupported length of compression member, in., see 10.11.3.1, Chapter 10

\( \ell_v \) = length of shearhead arm from centroid of concentrated load or reaction, in., Chapter 11

\( \ell_w \) = length of entire wall or length of segment of wall considered in direction of shear force, in., Chapters 11, 14, 21

\( M \) = maximum unfactored moment due to service loads, including \( P-\Delta \) effects, in.-lb, Chapter 14

\( M_1 \) = smaller factored end moment on a compression member, to be taken as positive if member is bent in single curvature, and negative if bent in double curvature, in.-lb, Chapter 10

\( M_{1ns} \) = factored end moment on a compression member at the end at which \( M_1 \) acts, due to loads that cause no appreciable sideways, calculated using a first-order elastic frame analysis, in.-lb, Chapter 10

\( M_{1s} \) = factored end moment on compression member at the end at which \( M_1 \) acts, due to loads that cause appreciable sideways, calculated using a first-order elastic frame analysis, in.-lb, Chapter 10

\( M_2 \) = larger factored end moment on compression member, always positive, in.-lb, Chapter 10

\( M_{2min} \) = minimum value of \( M_2 \), in.-lb, Chapter 10

\( M_{2ns} \) = factored end moment on compression member at the end at which \( M_2 \) acts, due to loads that cause no appreciable sideways, calculated using a first-order elastic frame analysis, in.-lb, Chapter 10

\( M_{2s} \) = factored end moment on compression member at the end at which \( M_2 \) acts, due to loads that cause appreciable sideways, calculated using a first-order elastic frame analysis, in.-lb, Chapter 10

\( M_a \) = maximum unfactored moment in member at stage deflection is computed, in.-lb, Chapters 9, 14

\( M_c \) = factored moment amplified for the effects of member curvature used for design of compression member, in.-lb, see 10.12.3, Chapter 10

\( M_{cr} \) = cracking moment, in.-lb, see 9.5.2.3, Chapters 9, 14

\( M_{cre} \) = moment causing flexural cracking at section due to externally applied loads, in.-lb, Chapter 11

\( M_m \) = factored moment modified to account for effect of axial compression, in.-lb, see 11.3.2.2, Chapter 11

\( M_{max} \) = maximum factored moment at section due to externally applied loads, in.-lb, Chapter 11

\( M_n \) = nominal flexural strength at section, in.-lb, Chapters 11, 12, 14, 18, 21

\( M_{nb} \) = nominal flexural strength of beam including slab where in tension, framing into joint, in.-lb, see 21.4.2.2, Chapter 21

\( M_{nc} \) = nominal flexural strength of column framing into joint, calculated for factored axial force, consistent with the direction of lateral forces considered, resulting in lowest flexural strength, in.-lb, see 21.4.2.2, Chapter 21

\( M_o \) = total factored static moment, in.-lb, Chapter 13

\( M_p \) = required plastic moment strength of shearhead cross section, in.-lb, Chapter 11

\( M_{pr} \) = probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile stress in the longitudinal bars of at least \( 1.25f'_t \), and a strength-reduction factor \( \phi \) of 1.0, in.-lb, Chapter 21

\( M_s \) = factored moment due to loads causing appreciable sway, in.-lb, Chapter 10

\( M_{sa} \) = maximum unfactored applied moment due to service loads, not including \( P-\Delta \) effects, in.-lb, Chapter 14

\( M_{slab} \) = portion of slab factored moment balanced by support moment, in.-lb, Chapter 21

\( M_u \) = factored moment at section, in.-lb, Chapters 10, 11, 13, 14, 21

\( M_{ua} \) = moment at the midheight section of the wall due to factored lateral and eccentric vertical loads, in.-lb, Chapter 14

\( M_v \) = moment resistance contributed by shearhead reinforcement, in.-lb, Chapter 11

\( N_c \) = tension force in concrete due to factored dead load plus live load, lb, Chapter 18

\( N_{cb} \) = nominal concrete breakout strength in tension of a single anchor in cracked concrete, lb, see D.5.2.2, Appendix D

\( N_{cbg} \) = nominal concrete breakout strength in tension of a single anchor in cracked concrete, lb, see D.5.2.2, Appendix D

\( N_{c} \) = nominal strength of column framing into joint, calculated for factored axial force, consistent with the direction of lateral forces considered, resulting in lowest flexural strength, in.-lb, see 21.4.2.2, Chapter 21

\( N_{p} \) = nominal strength in tension, lb, Appendix D

\( N_{pn} \) = nominal pullout strength in tension of a single anchor, lb, see D.5.3.4 and D.5.3.5, Appendix D

\( N_{pnm} \) = nominal pullout strength in tension of a single anchor, lb, see D.5.3.3.1, Appendix D

\( N_{sa} \) = nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, lb, see D.5.1.1 and D.5.1.2, Appendix D

\( N_{sb} \) = side-face blowout strength of a single anchor, lb, Appendix D

\( N_{sbg} \) = side-face blowout strength of a group of anchors, lb, Appendix D

\( N_{u} \) = factored axial force normal to cross section
occurring simultaneously with $V_u$ or $T_u$; to be taken as positive for compression and negative for tension, lb, Chapter 11

$N_{ua}$ = factored tensile force applied to anchor or group of anchors, lb, Appendix D

$N_{uc}$ = factored horizontal tensile force applied at top of bracket or corbel acting simultaneously with $V_u$, to be taken as positive for tension, lb, Chapter 11

$n$ = number of items, such as strength tests, bars, wires, monostrand anchorage devices, anchors, or shearhead arms, Chapters 5, 11, 12, 18, Appendix D

$P_a$ = differential pressure load, or related internal moments and forces, generated by a postulated pipe break accident, Chapter 9, Appendix C

$P_b$ = nominal axial strength at balanced strain conditions, lb, see 10.3.2, Chapters 9, 10

$P_c$ = critical buckling load, lb, see 10.12.3, Chapter 10

$P_n$ = nominal axial strength of cross section, lb, Chapters 9, 10, 14

$P_{n,max}$ = maximum allowable value of $P_n$, lb, see 10.3.6, Chapter 10

$P_o$ = nominal axial strength at zero eccentricity, lb, Chapter 10

$P_{pj}$ = prestressing force at anchorage device, lb, Chapter 18

$P_{pu}$ = factored prestressing force at anchorage device, lb, Chapter 18

$P_{px}$ = prestressing force evaluated at distance $l_{px}$ from the anchoring end, lb, Chapter 18

$P_s$ = unfactored axial load at the design (midheight) section including effects of self-weight, lb, Chapter 14

$P_u$ = factored axial force; to be taken as positive for compression and negative for tension, lb, Chapters 10, 14, 21

$P_{cp}$ = outside perimeter of concrete cross section, in., see 11.6.1, Chapter 11

$P_h$ = perimeter of centerline of outermost closed transverse torsional reinforcement, in., Chapter 11

$Q$ = stability index for a story, see 10.11.4, Chapter 10

$q_{Du}$ = factored dead load per unit area, Chapter 13

$q_{Lu}$ = factored live load per unit area, Chapter 13

$q_u$ = factored load per unit area, Chapter 13

$R$ = rain load, or related internal moments and forces, Chapter 9, Appendix C

$R_R$ = resistance (that is, load capacity), Appendix F

$R_a$ = piping and equipment reactions, or related internal moments and forces, under thermal conditions generated by a postulated pipe break and including $R_o$, Chapter 9, Appendix C

$R_m$ = maximum resistance, Appendix F

$R_o$ = piping and equipment reactions, or related internal moments and forces, that occur under normal operating and shutdown conditions, excluding dead load and earthquake reactions, Chapter 9, Appendix C

$r$ = radius of gyration of cross section of a compression member, in., Chapter 10

$r_0$ = rotational capacity, radians, Appendix F

$S$ = snow load, or related internal moments and forces, Chapters 9, Appendix C

$s$ = center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, prestressing tendons, wires, or anchors, in., Chapters 10-12, 17, 21, Appendix D

$s_2$ = center-to-center spacing of longitudinal shear or torsion reinforcement, in., Chapter 11

$s_h$ = center-to-center spacing of hoops, in., Appendix F

$s_i$ = center-to-center spacing of reinforcement in the $i$-th layer adjacent to the surface of the member, in., Appendix A

$s_o$ = center-to-center spacing of transverse reinforcement within the length $l_o$, in., Chapter 21

$s_s$ = sample standard deviation, psi, Chapter 5

$T_a$ = 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 $T_o$, Chapter 9, Appendix C

$T_n$ = nominal torsional moment strength, in.-lb, Chapter 11

$T_o$ = internal moments and forces caused by temperature distributions within the concrete structure and other temperature-induced loads occurring as a result of normal operating or shutdown conditions, Chapters 9, Appendix C

$T_u$ = factored torsional moment at section, in.-lb, Chapter 11

$t$ = wall thickness of hollow section, in., Chapter 11

$U$ = required strength to resist factored loads or related internal moments and forces, Chapter 9, Appendix C

$V_b$ = basic concrete breakout strength in shear of a single anchor in cracked concrete, lb, see D.6.2.2 and D.6.2.3, Appendix D

$V_c$ = nominal shear strength provided by concrete, lb, Chapters 8, 11, 13, 21

$V_{c1}$ = punching shear strength provided by a concrete plane of length $b_1$, lb, Chapter 11

$V_{c2}$ = punching shear strength provided by a concrete plane of length $b_2$, lb, Chapter 11

$V_{cb}$ = nominal concrete breakout strength in shear of a single anchor, lb, see D.6.2.1, Appendix D

$V_{cbg}$ = nominal concrete breakout strength in shear of a group of anchors, lb, see D.6.2.1, Appendix D

$V_{ci}$ = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, lb, Chapter 11

$V_{cp}$ = nominal concrete pryout strength of a single anchor, lb, see D.6.3, Appendix D

$V_{cpg}$ = nominal concrete pryout strength of a group of anchors, lb, see D.6.3, Appendix D

$V_{cw}$ = nominal shear strength provided by concrete when diagonal cracking results from high principal tensile stress in web, lb, Chapter 11

$V_d$ = shear force at section due to unfactored dead load, lb, Chapter 11
\( V_e \) = design shear force corresponding to the development of the probable moment strength of the member, lb, see 21.3.4.1 and 21.4.5.1, Chapter 21

\( V_l \) = factored shear force at section due to externally applied loads occurring simultaneously with \( M_{\text{max}} \), lb, Chapter 11

\( V_n \) = nominal shear strength, lb, Chapters 8, 10, 11, 21, Appendix D

\( V_{nh} \) = nominal horizontal shear strength, lb, Chapter 17

\( V_p \) = vertical component of effective prestress force at section, lb, Chapter 11

\( V_s \) = nominal shear strength provided by shear reinforcement, lb, Chapter 11

\( V_{sa} \) = nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, lb, see D.6.1.1 and D.6.1.2, Appendix D

\( V_u \) = factored shear force at section, lb, Chapters 11-13, 17, 21

\( V_{ua} \) = factored shear force applied to a single anchor or group of anchors, lb, Appendix D

\( V_{us} \) = factored horizontal shear in a story, lb, Chapter 10

\( V_{v} \) = nominal shear stress, psi, see 11.12.6.2, Chapter 11

\( W \) = operating basis wind load (OBW), or related internal moments and forces, Chapter 11

\( W_c \) = unit weight of concrete, lb/ft\(^3\), Chapters 8, 9

\( W_u \) = factored load per unit length of beam or one-way slab, Chapter 8

\( X_m \) = maximum acceptable displacement, Appendix F

\( X_p \) = displacement at effective yield point, Appendix F

\( x \) = shorter overall dimension of rectangular part of cross section, in., Chapter 13

\( Y_f \) = jet impingement load, or related internal moments and forces, on the structure generated by a postulated pipe break, Chapter 9, Appendix C

\( Y_m \) = missile impact load, or related internal moments and forces, on the structure generated by a postulated pipe break, such as pipe whip, Chapter 9, Appendix C

\( Y_r \) = loads, or related internal moments and forces, on the structure generated by the reaction of the broken pipe during a postulated break, Chapter 9, Appendix C

\( y \) = longer overall dimension of rectangular part of cross section, in., Chapter 13

\( y_t \) = distance from centroidal axis of gross section, neglecting reinforcement, to tension face, in., Chapters 9, 11

\( \alpha \) = angle defining the orientation of reinforcement, Chapters 11, 21, Appendix A

\( \alpha_c \) = coefficient defining the relative contribution of concrete strength to nominal wall shear strength, see 21.7.4.1, Chapter 21

\( \alpha_f \) = 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, see 13.6.1.6, Chapters 9, 13

\( \alpha_{f1} \) = \( \alpha_f \) in direction of \( t_1 \), Chapter 13

\( \alpha_{f2} \) = \( \alpha_f \) in direction of \( t_2 \), Chapter 13

\( \alpha_{fm} \) = average value of \( \alpha_f \) for all beams on edges of a panel, Chapter 9

\( \alpha_i \) = angle between the axis of a strut and the bars in the \( i \)-th layer of reinforcement crossing that strut, Appendix A

\( \alpha_{px} \) = total angular change of tendon profile from tendon jacking end to point under consideration, radians, Chapter 18

\( \alpha_s \) = constant used to compute \( V_e \) in slabs and footings, Chapter 11

\( \alpha_p \) = ratio of flexural stiffness of shearhead arm to that of the surrounding composite slab section, see 11.12.4.5, Chapter 11

\( \beta \) = ratio of long to short dimensions: clear spans for two-way slabs, see 9.5.3.3; sides of column, concentrated load or reaction area, see 11.12.2.1; or sides of a footing, see 15.4.4.2, Chapters 9, 11, 15

\( \beta_1 \) = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, see 10.2.7.3, Chapters 10, 18

\( \beta_p \) = ratio of area of reinforcement cut off to total area of tension reinforcement at section, Chapter 12

\( \beta_d \) = ratio used to compute magnified moments in columns due to sustained loads, see 10.11.1 and 10.13.6, Chapter 10

\( \beta_e \) = ratio of length of continuous edges to total perimeter of a slab panel, Chapter 9

\( \beta_n \) = factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone, Appendix A

\( \beta_p \) = factor used to compute \( V_e \) in prestressed slabs, Chapter 11

\( \beta_s \) = factor to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete in a strut, Appendix A

\( \beta_t \) = 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, see 13.6.4.2, Chapter 13

\( \Delta_1 \) = measured maximum deflection during first load test, in., see 20.5.2, Chapter 20

\( \Delta_2 \) = maximum deflection measured during second load test relative to the position of the structure at the beginning of second load test, in., see 20.5.2, Chapter 20

\( \Delta_f \) = increase in stress in prestressing steel due to factored loads, psi, Appendix A

\( \Delta_{fs} \) = stress in prestressing steel at sustained loads less decompression stress, psi, Chapter 18

\( \Delta_o \) = relative lateral deflection between the top and bottom of a story due to lateral forces computed
using a first-order elastic frame analysis and stiffness values satisfying 10.11.1, in., Chapter 10

\[ \Delta_r = \text{difference between initial and final (after load removal) deflections for load test or repeat load test, in., Chapter 20} \]

\[ \Delta_s = \text{maximum deflection at or near midheight due to service loads, in., Chapter 14} \]

\[ \Delta_u = \text{deflection at midheight of wall due to factored loads, in., Chapter 14} \]

\[ \delta_{ns} = \text{moment magnification factor for frames braced against sidesway, to reflect effects of member curvature between ends of compression member, Chapter 10} \]

\[ \delta_s = \text{moment magnification factor for frames not braced against sidesway, to reflect lateral drift resulting from lateral and gravity loads, Chapter 10} \]

\[ \delta_u = \text{design displacement, in., Chapter 21} \]

\[ \varepsilon_t = \text{net tensile strain in extreme layer of longitudinal tension steel at nominal strength, excluding strains due to effective prestress, creep, shrinkage, and temperature, Chapters 8-10} \]

\[ \phi = \text{ratio of the bending moments of factored loads to strength-reduction factor, see 9.3, Chapters 8-10, Appendixes A, C, D} \]

\[ \gamma = \text{ratio of the bending moments of factored loads to strength-reduction factor, see 9.3, Chapters 8-10, Appendixes A, C, D} \]

\[ \gamma_f = \text{factor used to determine the unbalanced moment transferred by flexure at slab-column connections, see 13.5.3.2, Chapters 11, 13, 21} \]

\[ \gamma_p = \text{factor for type of prestressing steel, see 18.7.2, Chapter 18} \]

\[ \gamma_s = \text{factor used to determine the portion of reinforcement located in center band of footing, see 15.4.4.2, Chapter 15} \]

\[ \gamma_v = \text{factor used to determine the unbalanced moment transferred by eccentricity of shear at slab-column connections, see 11.12.6.1, Chapter 11} \]

\[ \theta = \text{angle between axis of strut, compression diagonal, or compression field and the tension chord of the member, Chapter 11, Appendix A} \]

\[ \lambda_{\Delta} = \text{multiplier for additional deflection due to long-term effects, see 9.5.2.5, Chapter 9} \]

\[ \mu = \text{coefficient of friction, see 11.7.4.3, Chapter 11} \]

\[ \mu_d = \text{ductility ratio (dimensionless), Appendix F} \]

\[ \mu_p = \text{post-tensioning curvature friction coefficient, Chapter 18} \]

\[ \rho = \text{ratio of } A_s \text{ to } bd, \text{ Chapters 11, 13, 21, Appendix F} \]

\[ \rho' = \text{ratio of } A_{s'} \text{ to } bd, \text{ Chapter 9, Appendix F} \]

\[ \rho_1' = \text{reinforcement ratio in direction “1” based on section thickness } h. \text{ See 11.12.2.3, Chapter 11} \]

\[ \rho_2' = \text{reinforcement ratio in direction “2” based on section thickness } h. \text{ See 11.12.2.3, Chapter 11} \]

\[ \rho_b = \text{ratio of } A_s \text{ to } bd \text{ producing balanced strain conditions, see 10.3.2, Chapters 10, 13, 14} \]

\[ \rho_e = \text{ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement, Chapters 11,14, 21} \]

\[ \rho_p = \text{ratio of } A_{ps} \text{ to } bd_p, \text{ Chapter 18} \]

\[ \rho_s = \text{ratio of volume of spiral reinforcement to total volume of core confined by the spiral (measured out-to-out of spirals), Chapters 10, 21, Appendix F} \]

\[ \rho_t = \text{ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement, Chapters 11, 14, 21} \]

\[ \rho_v = \text{ratio of tie reinforcement area to area of contact surface, see 17.5.3.3, Chapter 17} \]

\[ \rho_w = \text{ratio of } A_s \text{ to } b_w d, \text{ Chapter 11} \]

\[ \xi = \text{time-dependent factor for sustained load, see 9.5.2.5, Chapter 9} \]

\[ \omega = \text{tension reinforcement index, see 18.7.2, Chapter 18} \]

\[ \omega' = \text{compression reinforcement index, see 18.7.2, Chapter 18} \]

\[ \psi_{c,N} = \text{factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete, see D.5.2.6, Appendix D} \]

\[ \psi_{c,P} = \text{factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete, see D.5.3.5, Appendix D} \]

\[ \psi_{c,V} = \text{factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement, see D.6.2.7 for anchors in shear, Appendix D} \]

\[ \psi_{cp,N} = \text{factor used to modify tensile strength of post-installed anchors intended for use in uncracked concrete without supplementary reinforcement, see D.5.2.7, Appendix D} \]

\[ \psi_e = \text{factor used to modify development length based on reinforcement coating, see 12.2.4, Chapter 12} \]

\[ \psi_{ec,N} = \text{factor used to modify tensile strength of anchors based on eccentricity of applied loads, see D.5.2.4, Appendix D} \]

\[ \psi_{ec,V} = \text{factor used to modify shear strength of anchors based on eccentricity of applied loads, see D.6.2.5, Appendix D} \]

\[ \psi_{ed,N} = \text{factor used to modify tensile strength of anchors based on proximity to edges of concrete member, see D.5.2.5, Appendix D} \]

\[ \psi_{ed,V} = \text{factor used to modify shear strength of anchors based on proximity to edges of concrete member, see D.6.2.6, Appendix D} \]

\[ \psi_s = \text{factor used to modify development length based on reinforcement size, see 12.2.4, Chapter 12} \]

\[ \psi_l = \text{factor used to modify development length based on reinforcement location, see 12.2.4, Chapter 12} \]

2.2—Definitions

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 device—In post-tensioning, the hardware used for transferring a post-tensioning force from the prestressing steel to the concrete.

authority having jurisdiction (AHJ)—A federal government agency (or agencies), such as the Nuclear Regulatory Commission, the Department of Energy, that is empowered to enforce regulations affecting the design, construction, and operation of nuclear facilities.

bonded tendon—Tendon in which prestressing steel is bonded to concrete either directly or through grouting.

cementitious materials—Materials as specified in Chapter 3, which have cementing value when used in concrete either by themselves, such as portland cement, blended hydraulic cements, and expansive cement, or such materials in combination with fly ash, other raw or calcined natural pozzolans, silica fume, and/or ground-granulated blast-furnace slag.

column—Member with a ratio of height-to-least lateral dimension exceeding 3 used primarily to support axial compressive load.

composite concrete flexural members—Concrete flexural members of precast or cast-in-place concrete elements, or both, constructed in separate placements but so interconnected that all elements respond to loads as a unit.

compression-controlled section—a cross section in which the net tensile strain in the extreme tension steel at nominal strength is less than or equal to the compression-controlled strain limit.

compression-controlled strain limit—the net tensile strain at balanced strain conditions. See 10.3.3.

concrete—Mixture of portland cement or any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixtures.

concrete, specified compressive strength of, \( f'c \)—Compressive strength of concrete used in design and evaluated 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 result has units of pounds per square inch (psi).

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

creep—Time-dependent deformation due to sustained load.

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

defomed reinforcement—Deformed reinforcing bars, bar mats, deformed wire, and welded wire reinforcement conforming to 3.5.3.

design basis tornado (DBT)—The combinations of translational speed, rotational speed, and prescribed pressure drop related to the effects of tornadoes on structures. DBT shall be as defined by AHJ at the site.

devolvement length—Length of embedded reinforcement, including pretensioned strand, required to develop the design strength of reinforcement at a critical section. See 9.3.3.

drop panel—a projection below the slab at least 1/4 of the slab thickness beyond the drop.

duct—a conduit (plain or corrugated) to accommodate prestressing steel for post-tensioned installation. Requirements for post-tensioning ducts are given in 18.17.

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

effective prestress—Stress remaining in prestressing steel after all losses have occurred.

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 such as to interfere least with performance of the structure, yet such as to allow relative movement in three directions and avoid formation of cracks elsewhere in the concrete and through which all or part of the bonded reinforcement is interrupted.

jacking force—in prestressed concrete, temporary force exerted by device that introduces tension into prestressing steel.

load, dead—Dead weight supported by a member, as defined by the engineer (without load factors).

load, factored—Load, multiplied by appropriate load factors, used to proportion members by the strength design method of this Code. See 8.1.1 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 that 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.
moment frame—a frame in which members and joints resist forces through flexure, shear, and axial force. Moment frames shall be categorized as follows:

ordinary moment frame—a cast-in-place or precast concrete frame complying with the requirements of Chapters 1 through 18.
special moment frame—a cast-in-place frame complying with the requirements of 21.2 through 21.5. In addition, the requirements for ordinary moment frames shall be satisfied.

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

operating basis earthquake (OBE)—the operating basis earthquake 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 functional. 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 (OBW)—wind velocities and forces required for the design of a structure in accordance with SEI/ASCE 7-02 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 not exceeding 3.

plain reinforcement—reinforcement that does not conform to definition of deformed reinforcement. See 3.5.4.

post-tensioning—method of prestressing in which prestressing steel is tensioned after concrete has hardened.

precast concrete—structural concrete element cast elsewhere than its final position in the structure.

precompressed tensile zone—portion of a prestressed member where flexural tension, calculated using gross section properties, would occur under unfactored dead and live loads if the prestress force was not present.

prestressed concrete—structural concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.

prestressing steel—high-strength steel element such as wire, bar, or strand, or a bundle of such elements, used to impart prestress forces to concrete.

pretensioning—method of prestressing in which prestressing steel is tensioned before concrete is placed.

reinforced concrete—structural concrete reinforced with no less than the minimum amounts of prestressing steel or nonprestressed reinforcement specified in Chapters 1 through 21 and Appendixes A through C.

reinforcement—material that conforms to 3.5, excluding prestressing steel 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 before the installation of the reshores.

safe shutdown earthquake (SSE)—the safe shutdown earthquake ground motion 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 and for more information, see Appendix S of 10CFR50 of the Federal Regulation. In the DOE Nuclear facilities and older nuclear plants, the term Design Basis Earthquake (DBE) is used, conveying the same meaning as SSE for design purposes.

sheathing—a material encasing prestressing steel to prevent bonding of the prestressing steel with the surrounding concrete, to provide corrosion protection, and to contain the corrosion-inhibiting coating.

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

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

span length—see 8.7.

specification, construction—an explicit set of requirements, typically specified by the owner and/or engineer, for the procurement of materials and for the erection of the structures.

specification, design—an explicit set of requirements, typically created by the owner, for the design of a safety-related structure.

specification, technical—the design and performance criteria and operating limits and principles of an operating license to be observed during the initial fuel loading, critical testing, start-up, power operations, refueling and maintenance operations.

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 reinforcement either single leg or bent into L, U, or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. (The term “stirrups” is usually applied to lateral reinforcement in flexural members and the term “ties” to those in compression members.) See also tie.

strength, design—nominal strength multiplied by a strength-reduction factor φ. See 9.3.

strength, nominal—strength of a member or cross section calculated in accordance with provisions and assumptions of the strength design method of 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—the time-dependent decrease in stress in a material held at a constant strain.

structural walls—walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake
motions. A shearwall is a structural wall. Structural walls shall be categorized as follows:

**ordinary reinforced concrete structural wall**—a wall complying with the requirements of Chapters 1 through 18.

**special reinforced concrete structural wall**—a cast-in-place wall complying with the requirements of 21.2 and 21.7 in addition to the requirements for ordinary reinforced concrete structural walls.

**tendon**—in pretensioned applications, the tendon is the prestressing steel. In post-tensioned applications, the tendon is a complete assembly consisting of anchorages, prestressing steel, and sheathing with coating for unbonded applications or ducts with grout for bonded applications.

**tension-controlled section**—a cross section in which the net tensile strain in the extreme tension steel at nominal strength is greater than or equal to 0.005.

**technical safety requirements**—the limits, controls, and related actions that establish the specific parameters and requisite actions for the safe operation of a nuclear facility.

**tie**—loop of reinforcing bar or wire enclosing longitudinal reinforcement. A continuously wound bar or wire in the form of a circle, rectangle, or other polygon shape without re-entrant corners is acceptable. See also **stirrup**.

**transfer**—act of transferring stress in prestressing steel from jacks or pretensioning bed to concrete member.

**transfer length**—length of embedded pretensioned strand required to transfer the effective prestress to the concrete.

**unbonded tendon**—tendon in which the prestressing steel is prevented from bonding to the concrete and is free to move relative to the concrete. The prestressing force is permanently transferred to the concrete at the tendon ends by the anchorages only.

**wall**—member, usually vertical, used to enclose or separate spaces.

**welded wire reinforcement**—reinforcing elements consisting of plain or deformed wires, conforming to ASTM A 82 or A 496, respectively, fabricated into sheets in accordance with ASTM A 185 or A 497, respectively.

**wobble friction**—in prestressed concrete, friction caused by unintended deviation of prestressing sheath or duct from its specified profile.

**yield strength**—specified minimum yield strength or yield point of reinforcement. Yield strength or yield point shall be determined in tension according to applicable ASTM standards as modified by 3.5 of this Code.

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**CHAPTER 3—MATERIALS**

**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.

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**3.2—Cements**

3.2.1 Cement shall conform to one of the following specifications:

(a) “Standard Specification for Portland Cement” (ASTM C 150);

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

(c) “Standard 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) “Standard Specification for Concrete Aggregates” (ASTM C 33);

(b) “Standard Specification for Aggregates for Radiation-Shielding Concrete” (ASTM C 637).

Exception: Aggregates failing to meet ASTM C 33 or C 637 but that have been shown by special test or actual service to produce concrete of adequate strength and durability shall be permitted to be used for normalweight concrete where authorized by the engineer.

3.3.2 Nominal maximum size of coarse aggregate shall be not larger than:

(a) 1/5 the narrowest dimension between sides of forms, nor

(b) 1/3 the depth of slabs, nor

(c) 3/4 the minimum clear spacing between individual reinforcing bars or wires, bundles of bars, individual tendons, bundled tendons, or ducts.

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

3.3.3 Testing requirements

3.3.3.1 Tests for full conformance with each of the requirements of the applicable specification, including tests for potential reactivity (ASTM C 289), shall be performed before 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 aggregate water content, gradation, fineness modulus, 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 periodically.
3.4—Water

3.4.1 Water used in mixing concrete shall be clean and free from injurious amounts of oils, acids, alkalies, salts, organic materials, or other substances 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 4.4.1.

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

3.4.3.1 Selection of concrete proportions shall be based on concrete mixtures using water from the same source.

3.4.3.2 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 “Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)” (ASTM C 109).

3.5—Steel reinforcement

3.5.1 Reinforcement shall be deformed reinforcement, except that plain reinforcement shall be permitted for spirals or prestressing steel; and reinforcement consisting of structural steel, steel pipe, or steel tubing shall be permitted as specified in 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, by a certified welder. 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 the requirements for deformed bars in one of the following specifications:

(a) “Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement” (ASTM A 615);

(b) “Standard Specification for Low-Alloy Steel Deformed and Plain 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 (ASTM A 370).

3.5.3.2 Deformed reinforcing bars shall conform to one of the ASTM specifications listed in 3.5.3.1, except that for bars with $f_y$ exceeding 60,000 psi, the yield strength shall be taken as the stress corresponding to a strain of 0.35%. See 9.4.

3.5.3.3 Bar mats for concrete reinforcement shall conform to “Standard Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement” (ASTM A 184). Reinfocing bars used in bar mats shall conform to ASTM A 615 or A 706.

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

3.5.3.5 Welded plain wire reinforcement shall conform to “Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete” (ASTM A 185). Welded intersections shall not be spaced farther apart than 12 in. in the direction of calculated stress, except for welded deformed wire reinforcement used as stirrups in accordance with 12.13.2.

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

3.5.3.7 This section number is listed to maintain consistency with ACI 318-05.

3.5.3.8 Epoxy-coated reinforcing bars shall comply with “Standard Specification for Epoxy-Coated Reinforcing Steel Bars” (ASTM A 775) or with “Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars” (ASTM A 934). Epoxy-coated wires and welded wire reinforcement shall comply with “Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement” (ASTM A 884). Epoxy-coated reinforcing steel shall also conform to one of the specifications listed in 3.5.3.1. Wires to be epoxy-coated shall conform to 3.5.3.4 and welded wire reinforcement to be epoxy-coated shall conform to 3.5.3.5 or 3.5.3.6. The engineer shall evaluate the suitability of coated reinforcing steel and epoxy-coated steel wire and welded wire reinforcement for the expected service environment in each application.

3.5.4 Plain reinforcement

3.5.4.1 Plain bars for spiral reinforcement shall conform to the specification listed in 3.5.3.1(a), or 3.5.3.1(b) including additional requirements of 3.5.3.1.1.

3.5.4.2 Plain wire for spiral reinforcement shall conform to “Standard Specification for Steel Wire, Plain, for Concrete Reinforcement” (ASTM A 82).

3.5.5 Prestressing steel

3.5.5.1 Steel for prestressing shall conform to one of the following specifications:

(a) Wire conforming to “Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete” (ASTM A 421);

(b) Low-relaxation wire conforming to “Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete” including Supplement “Low-Relaxation Wire” (ASTM A 421);
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 steel forms. See 4.3.2 and 4.4.1.


3.6.5 Water-reducing admixtures, retarding admixtures, accelerating admixtures, water-reducing and retarding admixtures, and water-reducing and accelerating admixtures shall conform to “Standard Specification for Chemical Admixtures for Concrete” (ASTM C 494) or “Standard 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 “Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete” (ASTM C 618).

3.6.7 Ground-granulated blast-furnace slag used as an admixture shall conform to “Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars” (ASTM C 989).

3.6.8 Admixtures used in concrete containing ASTM C 845 expansive cements shall be compatible with the cement and produce no deleterious effects.

3.6.9 Silica fume used as an admixture shall conform to “Standard Specification for Silica Fume Used in Cementitious Mixtures” (ASTM C 1240).

3.10 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 and periodically thereafter.

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 (ASTM E 204).

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 aggregates shall be stored in such 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 specification 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—Referenced standards

3.8.1 Standards of the ASTM 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:

(c) Strand conforming to “Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete” (ASTM A 416);

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

3.5.6 Structural steel, steel pipe, or tubing

3.5.6.1 Structural steel used with reinforcing bars in composite compression members meeting requirements of 10.16.7 or 10.16.8 shall conform to one of the following specifications:

(a) “Standard Specification for Carbon Structural Steel” (ASTM A 36);

(b) “Standard Specification for High-Strength Low-Alloy Structural Steel” (ASTM A 242);

(c) “Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel” (ASTM A 572);

(d) “Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in. (100 mm) Thick” (ASTM A 588);

(e) “Standard Specification for Structural Steel Shapes” (ASTM A 992).

3.5.6.2 Steel pipe or tubing for composite compression members composed of a steel-encased concrete core meeting requirements of 10.16.6 shall conform to one of the following specifications:

(a) Grade B of “Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless” (ASTM A 53);

(b) “Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes” (ASTM A 500);

(c) “Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing” (ASTM A 501).

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.

3.8.1 Standards of the ASTM 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/A 36M-04 Standard Specification for Carbon Structural Steel

A 53/A 53M-02 Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless

A 82-02 Standard Specification for Steel Wire, Plain, for Concrete Reinforcement

A 184/A 184M-01 Standard Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement

A 185-02 Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete

A 242/A 242M-04 Standard Specification for High-Strength Low-Alloy Structural Steel

A 307-04 Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength

A 370-05 Standard Test Methods and Definitions for Mechanical Testing of Steel Products

A 416/A 416M-02 Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete

A 421/A 421M-02 Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete

A 496-02 Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement

A 497/A 497M-02 Standard Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete

A 500-03a Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

A 501-01 Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing

A 572/A 572M-04 Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel

A 588/A 588M-04 Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4-in. [100 mm] Thick

A 615/A 615M-04b Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement

A 706/A 706M-04b Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement


A 775/A 775M-04a Standard Specification for Epoxy-Coated Steel Reinforcing Bars

A 884/A 884M-04 Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement

A 934/A 934M-04 Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars

A 992/A 992-M04 Standard Specification for Structural Steel Shapes

C 31/C 31M-03a Standard Practice for Making and Curing Concrete Test Specimens in the Field

C 33-03 Standard Specification for Concrete Aggregates

C 39/C 39M-03 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens

C 42/C 42M-04 Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

C 88-99a Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate

C 94/C 94M-04 Standard Specification for Ready-Mixed Concrete


C 144-03 Standard Specification for Aggregate for Masonry Mortar

C 150-04a Standard Specification for Portland Cement

C 172-04 Standard Practice for Sampling Freshly Mixed Concrete

C 192/C 192M-05 Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory

C 260-01 Standard Specification for Air-Entraining Admixtures for Concrete

C 289-03 Standard Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method)

C 494/C 494M-04 Standard Specification for Chemical Admixtures for Concrete

C 595-03 Standard Specification for Blended Hydraulic Cements

C 597-02 Standard Test Method for Pulse Velocity through Concrete

C 618-03 Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete


C 685/685M-01 Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing

C 845-04 Standard Specification for Expansive Hydraulic Cement
Table 4.2.1—Total air content for frost-resistant concrete

<table>
<thead>
<tr>
<th>Nominal maximum aggregate size, in.</th>
<th>Severe exposure</th>
<th>Moderate exposure</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8</td>
<td>7.5</td>
<td>6</td>
</tr>
<tr>
<td>1/2</td>
<td>7</td>
<td>5.5</td>
</tr>
<tr>
<td>3/4</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>1</td>
<td>6</td>
<td>4.5</td>
</tr>
<tr>
<td>1-1/2</td>
<td>5.5</td>
<td>4.5</td>
</tr>
<tr>
<td>2†</td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td>3†</td>
<td>4.5</td>
<td>3.5</td>
</tr>
</tbody>
</table>

*Refer to ASTM C 33 for tolerance on oversize for various nominal maximum size designations.
†These air contents apply to total mixture, as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 1-1/2 in. is removed by hand-picking or sieving, and air content is determined on the 1-1/2 in. fraction of mixture (tolerance on air content as delivered applies to this value). Air content of total mixture is computed from value determined on the 1-1/2 in. fraction.

Table 4.2.2—Requirements for special exposure conditions

<table>
<thead>
<tr>
<th>Exposure condition</th>
<th>Maximum water-cementitious material ratio, by weight, normalweight concrete, psi†</th>
<th>Minimum $f'_c$, normalweight concrete, psi†</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete intended to have low permeability when exposed to water</td>
<td>0.50</td>
<td>4000</td>
</tr>
<tr>
<td>Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals</td>
<td>0.45</td>
<td>4500</td>
</tr>
<tr>
<td>For corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources</td>
<td>0.40</td>
<td>5000</td>
</tr>
</tbody>
</table>

*When both Tables 4.3.1 and 4.2.2 are considered, the lowest applicable maximum water-cementitious material ratio and highest applicable minimum $f'_c$ shall be used.

except as noted in 5.2, slag meeting ASTM C 989, and silica fume meeting ASTM C 1240, if any, except as limited by 4.2.3.

4.2—Freezing and thawing exposures

4.2.1 Normalweight 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 $f'_c$ greater than 5000 psi, air content indicated in Table 4.2.1 shall be permitted to be reduced by 1.0 percentage points.

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 material ratios and minimum $f'_c$ 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 percentages of the total weight of cementitious materials given in Table 4.2.3.

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
Table 4.2.3—Requirements for concrete exposed to deicing chemicals

<table>
<thead>
<tr>
<th>Cementitious materials</th>
<th>Maximum percent of total cementitious materials by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fly ash or other pozzolans conforming to ASTM C 618</td>
<td>25</td>
</tr>
<tr>
<td>Slag conforming to ASTM C 989</td>
<td>50</td>
</tr>
<tr>
<td>Silica fume conforming to ASTM C 1240</td>
<td>10</td>
</tr>
<tr>
<td>Total of fly ash or other pozzolans, slag, and silica fume</td>
<td>50†</td>
</tr>
<tr>
<td>Total of fly ash or other pozzolans and silica fume</td>
<td>35†</td>
</tr>
</tbody>
</table>

†Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.
‡Seawater.
*When both Tables 4.3.1 and 4.2.2 are considered, the lowest applicable maximum water-cementitious material ratio and highest applicable minimum concrete cover requirements of 7.7 shall be satisfied.

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

<table>
<thead>
<tr>
<th>Sulfate exposure</th>
<th>Water-soluble sulfate (SO₄) in soil, % by weight</th>
<th>Sulfate (SO₄) in water, ppm</th>
<th>Cement type</th>
<th>Maximum w/cm, by weight, normal-weight concrete</th>
<th>Minimum fᶜ', normal-weight concrete, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negligible</td>
<td>0.00 ≤ SO₄ &lt; 0.10</td>
<td>0 ≤ SO₄ &lt; 150</td>
<td>II, IP(PS), IS(MS), IP(MS), IS(PS), IS(MS), IP(PS), IP(MS)</td>
<td>0.50</td>
<td>4000</td>
</tr>
<tr>
<td>Moderate</td>
<td>0.10 ≤ SO₄ &lt; 0.20</td>
<td>150 ≤ SO₄ &lt; 1500</td>
<td>V</td>
<td>0.45</td>
<td>4500</td>
</tr>
<tr>
<td>Severe</td>
<td>0.20 ≤ SO₄ &lt; 2.00</td>
<td>1500 ≤ SO₄ &lt; 10,000</td>
<td>V plus pozzolan†</td>
<td>0.45</td>
<td>4500</td>
</tr>
<tr>
<td>Very severe</td>
<td>SO₄ &gt; 2.00</td>
<td>SO₄ &gt; 10,000</td>
<td>V plus pozzolan†</td>
<td>0.45</td>
<td>4500</td>
</tr>
</tbody>
</table>

†When both Tables 4.3.1 and 4.2.2 are considered, the lowest applicable maximum water-cementitious material ratio and highest applicable minimum fᶜ' shall be used.
‡Seawater.
*Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

Table 4.4.1—Maximum chloride ion content for corrosion protection of reinforcement

<table>
<thead>
<tr>
<th>Type of member</th>
<th>Maximum water-soluble chloride ion (Cl⁻) in concrete, percent by weight of cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressed concrete</td>
<td>0.06</td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Concrete made with a cement that provides sulfate resistance and that has a maximum water-cementitious material ratio and minimum fᶜ' 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, cementitious 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.

4.4.2 When reinforced concrete will be exposed to deicing chemicals, salt, brackish water, seawater, or spray from these sources, requirements of Table 4.2.2 for maximum water-cementitious material ratio and minimum fᶜ', and the minimum concrete cover requirements of 7.7 shall be satisfied. See 18.16 for unbonded tendons.

CHAPTER 5—CONCRETE QUALITY, MIXING, AND PLACING

5.1—General

5.1.1 Concrete shall be proportioned to provide an average compressive strength fᶜ', as prescribed in 5.3.2, and shall satisfy the durability criteria of Chapter 4. Concrete shall be produced to minimize the frequency of strength tests below fᶜ', as prescribed in 5.6.3.3. For concrete designed and constructed in accordance with the Code, fᶜ' shall not be less than 2500 psi.

5.1.2 Requirements for fᶜ' shall be based on tests of cylinders made and tested as prescribed in 5.6.3.

5.1.3 Unless otherwise specified, fᶜ' shall be based on 28-day tests. If other than 28 days, test age for fᶜ' 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 fᶜ' 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;

(c) Conformance with strength test requirements of 5.6.

5.2.2 Where different materials are to be used for different portions of proposed work, each combination shall be evaluated.

5.2.3 Concrete proportions shall be established in accordance with 5.3 or, alternatively, 5.4, and shall meet applicable requirements of Chapter 4.

5.3—Proportioning on the basis of field experience or trialmixtures, or both

5.3.1 Sample standard deviation

5.3.1.1 Where a concrete production facility has test records, a sample standard deviation sₓ shall be established. Test records from which sₓ 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 restrictive than those for proposed work;

(b) Shall represent concrete produced to meet a specified concrete strength or strengths within 1000 psi of fᶜ'.


(c) Shall consist of at least 30 consecutive tests or two groups of consecutive tests totaling at least 30 tests as defined in 5.6.2.4, 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 sample standard deviation \( s_x \) shall be established as the product of the calculated sample standard deviation and modification factor of Table 5.3.1.2. To be acceptable, test records 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.

5.3.2 Required average strength

5.3.2.1 Required average compressive strength \( f_{cr} \) used as the basis for selection of concrete proportions shall be determined from Table 5.3.2.1 using the sample standard deviation \( s_x \), calculated in accordance with 5.3.1.1 or 5.3.1.2.

5.3.2.2 When a concrete production facility does not have field strength test records for calculation of \( s_x \) meeting requirements of 5.3.1.1 or 5.3.1.2, \( f_{cr} \) shall be determined from Table 5.3.2.2, and documentation of average strength shall be in accordance with requirements of 5.3.3.

5.3.3 Documentation of average compressive strength—
Documentation that proposed concrete proportions will produce an average compressive strength equal to or greater than required average compressive strength \( f_{cr} \) (see 5.3.2) shall consist of a field strength test record, several strength test records, or trial mixtures.

5.3.3.1 When test records are used to demonstrate that proposed concrete proportions will produce \( f_{cr} \) (see 5.3.2), such records shall represent materials and conditions similar to those expected. Changes in materials, conditions, and proportions within the test records shall not have been more restrictive than those for proposed work. For the purpose of documenting average strength potential, test records consisting of less than 30 but not less than 10 consecutive tests are acceptable, provided test records encompass a period of time not less than 45 days. Required concrete proportions shall be permitted to be established by interpolation between the strengths and proportions of two or more test records, each of which meets other requirements of this section.

5.3.3.2 When an acceptable record of field test results is not available, concrete proportions established from trial mixtures meeting the following restrictions shall be permitted:
(a) Combination of materials shall be those for proposed work;
(b) Trial mixtures having proportions and consistencies required for proposed work shall be made using at least three different water-cementitious material ratios or cementitious material contents that will produce a range of strengths encompassing \( f_{cr} \);
(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 material ratio or cementitious material content, at least three test cylinders for each test age shall be made and cured in accordance with “Standard Practice for 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 \( f_{cr} \);
(e) From results of cylinder tests, a curve shall be plotted showing the relationship between water-cementitious material ratio or cementitious material content and compressive strength at designated test age;
(f) Maximum water-cementitious material ratio or minimum cementitious material content for concrete to be used in proposed work shall be that shown by the curve to produce \( f_{cr} \) required by 5.3.2, unless a lower water-cementitious material ratio or higher strength is required by Chapter 4.

5.4—Proportioning without field experience or trial mixtures

5.4.1 If data required by 5.3 are not available, concrete proportions shall be based on other experience or information, if approved by the engineer. The required average compressive strength \( f_{cr} \) of concrete produced with materials similar to those proposed for use shall be at least 1200 psi greater than

<table>
<thead>
<tr>
<th>No. of tests*</th>
<th>Modification factor for sample standard deviation†</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 15</td>
<td>Use Table 5.3.2.2</td>
</tr>
<tr>
<td>15</td>
<td>1.16</td>
</tr>
<tr>
<td>20</td>
<td>1.08</td>
</tr>
<tr>
<td>25</td>
<td>1.03</td>
</tr>
<tr>
<td>30 or more</td>
<td>1.00</td>
</tr>
</tbody>
</table>

*Interpolate for immediate number of tests.
†Modified sample standard deviation \( s_x \) to be used to determine required average strength \( f_{cr} \) from 5.3.2.1.

<table>
<thead>
<tr>
<th>Compressive strength, psi</th>
<th>Required average compressive strength, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_c ) ≤ 3000</td>
<td>Use larger value computed from Eq. (5-1) and (5-2)</td>
</tr>
<tr>
<td></td>
<td>( f_{cr} = f_c + 1.34s_x ) (5-1)</td>
</tr>
<tr>
<td></td>
<td>( f_{cr} = f_c + 2.33s_x - 500 ) (5-2)</td>
</tr>
<tr>
<td>( f_c &gt; 5000 )</td>
<td>Use larger value computed from Eq. (5-1) and (5-3)</td>
</tr>
<tr>
<td></td>
<td>( f_{cr} = f_c + 1.34s_x ) (5-1)</td>
</tr>
<tr>
<td></td>
<td>( f_{cr} = 0.90f_c + 2.33s_x ) (5-3)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Compressive strength, psi</th>
<th>Required average compressive strength, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f' c ) &lt; 3000</td>
<td>( f_{cr} = f' c + 1000 )</td>
</tr>
<tr>
<td>3000 ≤ ( f' c ) ≤ 5000</td>
<td>( f_{cr} = f' c + 1200 )</td>
</tr>
<tr>
<td>( f' c &gt; 5000 )</td>
<td>( f_{cr} = 1.10f' c + 700 )</td>
</tr>
</tbody>
</table>
This alternative shall not be used if $f'_c$ is greater than 5000 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 compressive strength reduction

As data become available during construction, it shall be permitted to reduce the amount by which the required average concrete strength $f''_c$ must exceed $f'_c$, provided:

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

(b) Fifteen to 29 test results are available and average of test results exceeds that required by 5.3.2.1 using a sample 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 Concrete shall be tested in accordance with the requirements of 6.2 through 6.5. Qualified field testing technicians shall perform tests on fresh concrete at the job site, prepare specimens required for curing under field conditions, prepare specimens required for testing in the laboratory, and record the temperature of the fresh concrete when preparing specimens for strength tests. Qualified laboratory technicians shall perform all required laboratory tests.

5.6.2 Frequency of testing

5.6.2.1 Samples for strength tests of concrete should be taken at least once per day for each class of concrete placed, or at least once for each 100 yd$^3$ of concrete placed. When the standard deviation for 30 consecutive tests of a given class is less than 600 psi, the amount of concrete placed between tests may be increased by 50 yd$^3$ for each 100 psi the class is less than 600 psi, the amount of concrete placed on more than one shift per day or not less than one test for each shift 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.

5.6.2.2 On a given project, if total volume of concrete is such that frequency of testing required by 5.6.2.1 would provide less than five strength tests for a given class of concrete, tests shall be made from at least five randomly selected batches or from each batch if fewer than five batches are used.

5.6.2.3 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.

5.6.2.4 A strength test shall be the average of the strengths of a minimum of two cylinders made from the same sample of concrete and tested at 28 days or at test age designated for determination of $f'_c$.

5.6.3 Laboratory-cured specimens

5.6.3.1 Samples for strength tests shall be taken in accordance with “Standard Practice for Sampling Freshly Mixed Concrete” (ASTM C 172).

5.6.3.2 Cylinders for strength tests shall be molded and laboratory-cured in accordance with “Practice for Making and Curing Concrete Test Specimens in the Field” (ASTM C 31) and tested in accordance with “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens” (ASTM C 39).

5.6.3.3 Strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met:

(a) Every arithmetic average of any three consecutive strength tests equals or exceeds $f'_c$;

(b) No individual strength test (average of two cylinders) falls below $f'_c$ by more than 500 psi when $f'_c$ is 5000 psi or less; or by more than 0.10 $f'_c$ when $f'_c$ is more than 5000 psi.

5.6.3.4 If either of the requirements of 5.6.3.3 is not met, steps shall be taken to improve the average of subsequent strength test results. Requirements of 5.6.5 shall be observed if the requirement of 5.6.3.3(b) is not met.

5.6.4 Field-cured specimens

5.6.4.1 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.

5.6.4.2 Field-cured cylinders shall be cured under field conditions in accordance with “Standard Practice for Making and Curing Concrete Test Specimens in the Field” (ASTM C 31).

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

5.6.4.4 Procedures for protecting and curing concrete shall be improved when strength of field-cured cylinders at test age designated for determination of $f'_c$ is less than 85% 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.

5.6.5 Investigation of low-strength test results

5.6.5.1 If any strength test (see 5.6.2.4) of laboratory-cured cylinders falls below $f'_c$ by more than the values given in 5.6.3.3(b) or if tests of field-cured cylinders indicate deficiencies in protection and curing (see 5.6.4.4), steps shall be taken to assure that load-carrying capacity of the structure is not jeopardized.

5.6.5.2 If the likelihood of low-strength concrete is confirmed and calculations indicate that load-carrying capacity is significantly reduced, tests of cores drilled from the area in question in accordance with “Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete” (ASTM C 42) shall be permitted. In such cases, a minimum of three cores shall be taken for each strength test that falls below the values given in 5.6.3.3(b).

5.6.5.3 Cores shall be prepared for transport and storage by wiping drilling water from their surfaces and placing the cores in watertight bags or containers immediately after drilling. Cores shall be tested no earlier than 48 hours and not later than 7 days after coring unless approved by the engineer.

5.6.5.4 Concrete in an area represented by core tests shall be considered structurally adequate if the average of...
three cores is equal to at least 85% 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.  

5.6.5.4 If criteria of 5.6.5.4 are not met and if the structural adequacy remains in doubt, the engineer shall be permitted to 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 clean 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;
(g) All laitance and other unsound material shall be removed before additional concrete is placed against hardened concrete. The method for cleaning construction joints shall be stated in the specification.

5.8—Mixing  
5.8.1 All concrete shall be batched using automated equipment with digital recordation and 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 “Standard Specification for Ready-Mixed Concrete” (ASTM C 94) or “Standard 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 “Standard Specification for Ready-Mixed Concrete” (ASTM C 94);
(d) Materials handling, batching, and mixing shall conform to applicable provisions of “Standard Specification for Ready-Mixed Concrete” (ASTM C 94);
(e) A detailed record shall be kept to identify:
(1) number of batches produced;
(2) proportions of materials used;
(3) approximate location of final deposit in structure;
(4) time and date of mixing and placing.

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 or self-consolidating concrete may be used. 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, mixture proportioning, or alteration of this mixture.

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 5.6.4 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.

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 specifications.

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;
(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 to 6.1.7.1 and 6.1.7.2.

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

6.1.7.2 To the depth of placement 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, shores, and reshoring
6.2.1 Removal of forms—Forms shall be removed in such a manner as not to impair safety and serviceability of the structure. Concrete exposed by form removal shall have sufficient strength not to be damaged by removal operation.

6.2.2 Removal of shores and reshoring—The provisions of 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 and pipes embedded in concrete
6.3.1 Conduits, pipes, and sleeves of any material not harmful to concrete and within limitations of 6.3 shall be permitted to be embedded in concrete with approval of the engineer, provided they are not considered to replace structurally the displaced concrete, except as provided in 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 significantly impair 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 engineer, conduits and pipes embedded within a slab, wall, or beam (other than those merely passing through) shall satisfy 6.3.5.1 through 6.3.5.3.

6.3.5.1 They shall not be larger in outside dimension than 1/3 the overall thickness of slab, wall, or beam in which they are embedded.

6.3.5.2 They shall not be spaced closer than three diameters or widths on center.

6.3.5.3 They shall not significantly impair 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 in 6.3.6.1 through 6.3.6.3.

6.3.6.1 They are not exposed to rusting or other deterioration.

6.3.6.2 They are of uncoated or galvanized iron or steel not thinner than standard Schedule 40 steel pipe.

6.3.6.3 They have a nominal inside diameter not over 2 in. and are spaced not less than three diameters on centers.

6.3.7 Pipes and fittings shall be designed to resist effects of the material, pressure, and temperature to which they will be subjected.

6.3.8 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, 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 less than 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 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 proper location 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 strength 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 that are allowed to have increased temperatures not to exceed 200 °F;

(b) During an accident or for any other short-term interruption, 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 strength. Evidence shall also be provided that verifies that the increased temperatures do not cause deterioration of the concrete with or in the absence of applied loads.

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.

6.4.5 Construction joints in girders shall be offset a minimum distance of two times the width of intersecting beams.

6.4.6 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.
6.4.7 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

7.1—Standard hooks
The term “standard hook,” as used in this Code, shall mean one of the following:

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

7.1.2 90-degree bend plus 12\(d_b\) extension at free end of bar.

7.1.3 For stirrup and tie hooks
(a) No. 5 bar and smaller, 90-degree bend plus 6\(d_b\) extension at free end of bar; or
(b) No. 6, No. 7, and No. 8 bar, 90-degree bend plus 12\(d_b\) extension at free end of bar; or
(c) No. 8 bar and smaller, 135-degree bend plus 6\(d_b\) extension at free end of bar.

7.1.4 Seismic hooks as defined in 21.1.

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.

7.2.2 Inside diameter of bend for stirrups and ties shall not be less than 4\(d_b\) 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 bend in welded wire reinforcement for stirrups and ties shall not be less than 4\(d_b\) for deformed wire larger than D6 and 2\(d_b\) for all other wires. Bends with inside diameter of less than 8\(d_b\) shall not be less than 4\(d_b\) from nearest welded intersection.

7.3—Bending

7.3.1 All reinforcement shall be bent cold, unless otherwise permitted by the engineer.

7.3.2 Reinforcement partially embedded in concrete shall not be field bent, except as shown on the design drawings or permitted by the engineer.

7.4—Surface conditions of reinforcement

7.4.1 At the time concrete is placed, reinforcement shall be free from mud, oil, or other nonmetallic coatings that decrease bond. Epoxy coating of steel reinforcement in accordance with standards referenced in 3.5.3.8 shall be permitted if the coating is qualified for service conditions (that is, temperature and radiation) as well as fabrication conditions (that is, damaged epoxy coatings shall be repaired).

7.4.2 Except for prestressing steel, steel reinforcement with rust, mill scale, or a combination of both shall be considered satisfactory, provided the minimum dimensions (including height of deformations) and weight of a hand-wire-brushed test specimen comply with applicable ASTM specifications referenced in 3.5.

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

Table 7.2—Minimum diameters of bend

<table>
<thead>
<tr>
<th>Bar size</th>
<th>Minimum diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 3 through No. 8</td>
<td>6(d_b)</td>
</tr>
<tr>
<td>No. 9, No. 10, and No. 11</td>
<td>8(d_b)</td>
</tr>
<tr>
<td>No. 14 and No. 18</td>
<td>10(d_b)</td>
</tr>
</tbody>
</table>

7.5—Placing reinforcement

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

7.5.2 Unless otherwise specified by the engineer, reinforcement, including tendons and post-tensioning ducts, shall be placed within the tolerances in 7.5.2.1 and 7.5.2.2.

7.5.2.1 Tolerance for \(d\) and minimum concrete cover in flexural members, walls, and compression members shall be as follows:

<table>
<thead>
<tr>
<th>(d)</th>
<th>Tolerance on (d)</th>
<th>Tolerance on minimum concrete cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d \leq 8) in.</td>
<td>±3/8 in.</td>
<td>–3/8 in.</td>
</tr>
<tr>
<td>8 in. &lt; (d \leq 24) in.</td>
<td>±1/2 in.</td>
<td>–1/2 in.</td>
</tr>
<tr>
<td>(d &gt; 24) in.</td>
<td>±1 in.</td>
<td>–1/2 in.</td>
</tr>
</tbody>
</table>

except that tolerance that for the clear distance to formed soffits shall be –1/4 in. and tolerance for cover shall not exceed –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 ±2 in., except the tolerance shall be ±1/2 in. at the discontinuous ends of brackets and corbels, and ±1 in. at the discontinuous ends of other members. The tolerance for minimum concrete cover of 7.5.2.1 shall also apply at discontinuous ends of members.

7.5.3 Welded wire reinforcement (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 shall be permitted to 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 be \(d_b\), but not less than 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.
In spirally reinforced or tied reinforced compression members, clear distance between longitudinal bars shall be not less than \(4d_b\) nor less than 1-1/2 in. See also 3.3.2.

6.4 Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.

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 farther apart than 18 in.

6.6 Bundled bars

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.

6.6.2 Bundled bars shall be enclosed within stirrups or ties.

6.6.3 Bars larger than No. 11 shall not be bundled in beams.

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.

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.

6.7 Tendons and ducts

6.7.1 Center-to-center spacing of pretensioning tendons at each end of a member shall be not less than \(4d_b\) for strands, or \(5d_b\) for wire, except that if specified compressive strength of concrete at time of initial prestress, \(f_{ci}\), is 4000 psi or more, minimum center-to-center spacing of strands shall be 1-3/4 in. for strands of 1/2 in. nominal diameter or smaller and 2 in. for strands of 0.6 in. nominal diameter. See also 3.3.2. Closer vertical spacing and bundling of tendons shall be permitted in the middle portion of a span.

6.7.2 Bundling of post-tensioning ducts shall be permitted if shown that concrete can be satisfactorily placed and if provision is made to prevent the prestressing steel, when tensioned, from breaking through the duct.

7—Concrete protection for reinforcement

7.1 Cast-in-place concrete (nonprestressed)—The following minimum concrete cover shall be provided for reinforcement, but shall not be less than required by 7.7.5 and 7.7.7:

(a) Concrete cast against and permanently exposed to earth.........................3
(b) Concrete exposed to earth or weather:
   No. 6 through No. 18 bars.................................................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

7.2 Cast-in-place concrete (prestressed)—The following minimum concrete cover shall be provided for prestressed and nonprestressed reinforcement, ducts, and end fittings, but shall not be less than required by 7.7.5, 7.7.5.1, and 7.7.7:

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

7.3 Precast concrete (manufactured under plant control conditions)—The following minimum concrete cover shall be provided for prestressed and nonprestressed reinforcement, ducts, and end fittings, but shall not be less than required by 7.7.5, 7.7.5.1, and 7.7.7:

Minimum cover, in.

(a) Concrete exposed to earth or weather:
   Wall panels:
   No. 14 and No. 18 bars, prestressing tendons larger than 1-1/2 in. diameter....................1-1/2
   No. 11 bar and smaller, prestressing tendons 1-1/2 in. diameter and smaller, W31 and D31 wire and smaller.................3/4
   Other members:
   No. 14 and No. 18 bars, prestressing tendons larger than 1-1/2 in. diameter....................2
   No. 6 through No. 11 bars, prestressing tendons larger than 5/8 in. diameter through 1-1/2 in. diameter......................................................1-1/2
   No. 5 bar and smaller, prestressing tendons 5/8 in. diameter and smaller, W31 and 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, prestressing tendons larger than 1-1/2 in. diameter....................1-1/4
   Prestressing tendons 1-1/2 in. diameter and smaller ....3/4
   No. 11 bar and smaller, W31 or D31 wire, 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:
  Prestressing tendons ........................................ 3/4
  No. 6 bar and larger ........................................... 5/8
  No. 5 bar and smaller, W31 or D31 wire,
  and smaller .................................................. 3/8

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, where 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.

7.7.5.1 For prestressed concrete members exposed to corrosive environments or other severe exposure conditions, and that are classified as Class T or C in 18.3.3, minimum cover to the prestressed reinforcement shall be increased 50%. This requirement shall be permitted to be waived if the precompressed tensile zone is not in tension under sustained loads.

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

7.7.7 Fire protection—When necessary, the minimum cover thicknesses specified in 7.7.1 through 7.7.6 shall be increased considering the fire protection requirements.

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 shall be designed to transfer the load from the steel core only, provided ample concrete section is available for transfer of the portion of the total load carried by the reinforced concrete section to the footing by compression in the concrete and by reinforcement.

7.9—Connections

7.9.1 At connections of principal framing elements (such as beams and columns), enclosure shall be provided for splices of continuing reinforcement and for anchorage of reinforcement terminating in such connections.

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

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 Spiral reinforcement shall be spliced, if needed, by any one of the following methods:

(a) Lap splices not less than the larger of 12 in. and the length indicated in one of (1) through (5) below:

(1) deformed uncoated bar or wire ....................... 48d_b
(2) plain uncoated bar or wire ......................... 72d_b
(3) epoxy-coated deformed bar or wire ............ 72d_b
(4) plain uncoated bar or wire with a standard stirrup or tie hook in accordance with 7.1.3 at ends of lapped spiral reinforcement. The hooks shall be embedded within the core confined by the spiral reinforcement .......... 48d_b
(5) epoxy-coated deformed bar or wire with a standard stirrup or tie hook in accordance with 7.1.3 at ends of lapped spiral reinforcement. The hooks shall be embedded within the core confined by the spiral reinforcement

12.14.3.

7.10.4.6 Spirals shall extend from top of footing or slab in any story to level of lowest horizontal reinforcement in members supported above.

7.10.4.7 Where beams or brackets do not frame into all sides of a column, ties shall extend above termination of spiral to bottom of slab or drop panel.

7.10.4.8 In columns with capitals, spirals shall extend to a level at which the diameter or width of capital is two times that of the column.

7.10.4.9 Spirals shall be held firmly in place and true to line.

7.10.5 Ties—Tie reinforcement for compression members shall conform to the following:

7.10.5.1 All nonprestressed bars shall be enclosed by lateral ties, at least 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 reinforcement 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 degrees 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 1/2 a tie spacing above the top of footing or slab in any story, and shall be spaced as provided herein to not more than 1/2 a tie spacing below the lowest horizontal reinforcement in slab or drop panel above.

7.10.5.5 Where beams or brackets frame from four directions into a column, termination of ties not more than 3 in. below lowest reinforcement in shallowest of such beams or brackets shall be permitted.

7.10.5.6 Where anchor bolts are placed in the top of columns or pedestals, the bolts shall be enclosed by lateral reinforcement that also surrounds at least four vertical bars of the column or pedestal. The lateral reinforcement shall be distributed within 5 in. of the top of the column or pedestal, and shall consist of at least two No. 4 or three No. 3 bars.

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 reinforcement of equivalent area. Such ties or stirrups shall be provided throughout the distance where compression reinforcement is required.

7.11.2 Lateral reinforcement for flexural framing members subject to stress reversals or to torsion at supports shall consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement.

7.11.3 Closed ties or stirrups shall be formed in one piece by overlapping standard stirrup or tie end hooks around a longitudinal bar, or formed in one or two pieces lap spliced with a Class B splice (lap of 1.3d) 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 reinforcement 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 \( A'_{\text{min}} \) in each direction at each face given by \( A'_{\text{min}} = f'/A f''_y \) but need not exceed \( A/100 \).

The minimum reinforcement size shall be No. 6 bars. Instead of computation, \( f''_y \) may be taken as 60% of the specified yield strength of \( f_y \).

7.12.4 For concrete sections having a thickness of 72 in. or more, the minimum reinforcement requirements shall be permitted to be waived provided that members are 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 1/3 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 steel conforming to 3.5.5 used for minimum reinforcement shall be provided in accordance with 7.12.6.1 through 7.12.6.3.

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 conforming 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 with a Class A tension splice or a mechanical or welded splice satisfying 12.14.3 and at noncontinuous supports shall be terminated with a standard hook.

7.13.2.2 Beams along the perimeter of the structure shall have continuous reinforcement consisting of:

(a) at least 1/6 of the tension reinforcement required for negative moment at the support, but not less than two bars; and
(b) at least 1/4 of the tension reinforcement required for positive moment at midspan, but not less than two bars.

7.13.2.3 Where splices are needed to provide the required continuity, the top reinforcement shall be spliced at or near midspan and the bottom reinforcement shall be spliced at or near the support. Splices shall be Class A tension splices or mechanical or welded splices satisfying 12.14.3. The continuous reinforcement required in 7.13.2.2(a) and 7.13.2.2(b) shall be enclosed by the corners of U-stirrups having not less than 135-degree hooks around the continuous top bars, or by one-piece closed stirrups with not less than 135-degree hooks around one of the continuous top bars. Stirrups need not be extended through any joints.

7.13.2.4 In other than perimeter beams, when stirrups as defined in 7.13.2.3 are not provided, at least 1/4 of the positive moment reinforcement required at midspan, but not less than two bars, shall be continuous or shall be spliced over or near the support with a Class A tension splice or a mechanical or welded splice satisfying 12.14.3, and at noncontinuous supports shall be terminated with a standard hook.

7.13.2.5 For two-way slab construction, see 13.3.8.5.

7.13.3 For precast concrete construction, tension ties shall be provided in the transverse, longitudinal, and vertical directions and around the perimeter of the structure to effectively tie elements together. The provisions of 16.5 shall apply.

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

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 specified in Chapter 9.

8.1.2 Intentionally left blank.

8.1.3 Anchors within the scope of Appendix D, Anchoring to Concrete, installed in concrete to transfer loads between connected elements shall be designed using Appendix D.

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 Appendixes D, E, and F. 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 shall be permitted for buildings of usual types of construction, spans, and story heights.

8.3.3 As an alternate to frame analysis, the following approximate moments and shears shall be permitted for design of continuous beams and one-way slabs (slabs reinforced to resist flexural stresses in only one direction), provided:

(a) There are two or more spans;
(b) Spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20%;
(c) Loads are uniformly distributed;
(d) Unfactored live load $L$ does not exceed three times unfactored dead load $D$; and
(e) Members are prismatic.

Positive moment

End spans
Discontinuous end unrestrained $w_u l_u^2 / 11$
Discontinuous end integral with support $w_u l_u^2 / 14$
Interior spans $w_u l_u^2 / 16$

Negative moments at exterior face of first interior support:

Two spans $w_u l_u^2 / 9$
More than two spans $w_u l_u^2 / 10$

Negative moment at other faces of interior support $w_u l_u^2 / 11$

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 8
at each end of the span $w_u l_u^2 / 12$

Negative moment at interior face of exterior support for members built integrally with supports:
Where support is spandrel beam $w_u l_u^2 / 24$
Where support is a column $w_u l_u^2 / 16$

Shear in end members at face of first interior support $w_u l_u^2 / 2$
Shear at face of all other supports $w_u l_u^2 / 2$

8.3.4 Strut-and-tie models shall be permitted to be used in the design of structural concrete. See Appendix A.
8.4—Redistribution of negative moments in continuous flexural members

8.4.1 Except where approximate values for moments are used, it shall be permitted to increase or decrease negative moments calculated by elastic theory at supports of continuous flexural members for any assumed loading arrangement by not more than $1000\varepsilon\%$, with a maximum of 20%.

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 $\varepsilon$ is equal to or greater than 0.0075 at the section at which moment is reduced.

8.5—Modulus of elasticity

8.5.1 Modulus of elasticity $E_c$ for concrete shall be permitted to be taken as $w_c^{1+33} F_c^{1/2}$ (in psi) for values of $w_c$ between 90 and 155 lb/ft. For normalweight concrete, $E_c$ shall be permitted to be taken as $57,000 \sqrt{F_c}$.

8.5.2 Modulus of elasticity $E_p$ for nonprestressed reinforcement shall be permitted to be taken as 29,000,000 psi.

8.5.3 Modulus of elasticity $E_p$ for prestressing steel shall be determined by tests or reported by the manufacturer.

8.6—Stiffness

8.6.1 Use of any set of reasonable assumptions shall be permitted for computing relative flexural and torsional stiffnesses of columns, walls, floors, and roof systems. The assumptions adopted shall be consistent throughout analysis.

8.6.2 Effect of haunches shall be considered both in determining moments and in design of members.

8.7—Span length

8.7.1 Span length of members not built integrally with supports shall be considered as the clear span plus the depth of the member, but need not exceed distance between centers of supports.

8.7.2 In analysis of frames or continuous construction for determination of moments, span length shall be taken as the distance center-to-center of supports.

8.7.3 For beams built integrally with supports, design on the basis of moments at faces of support shall be permitted.

8.7.4 It shall be permitted to analyze solid or ribbed slabs built integrally with supports, with clear spans not more than 10 ft, 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 far ends of columns built integrally with the structure to be fixed.

8.8.4 Resistance to moments at any floor or roof level shall be provided by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffnesses and conditions of restraint.

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 1/4 of the span length of the beam, and the effective overhanging flange width on each side of the web shall not exceed:
(a) eight times the slab thickness; and
(b) 1/2 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) 1/12 the span length of the beam; and
(b) six times the slab thickness; and
(c) 1/2 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 1/2 the width of web and an effective flange width not more than four times the width of web.

8.10.5 Where primary flexural reinforcement in a slab that is considered as a T-beam flange (excluding joist construction) is parallel to the beam, reinforcement perpendicular to the beam shall be provided in the top of the slab in accordance with the following:

8.10.5.1 Transverse reinforcement shall be designed to carry the factored load on the overhanging slab width assumed to act as a cantilever. For isolated beams, the full width of overhanging flange shall be considered. For other T-beams, only the effective overhanging slab width need be considered.

8.10.5.2 Transverse reinforcement shall be spaced not farther apart than five times the slab thickness, nor farther apart than 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 be not 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 Intentionally left blank.

8.11.6 When removable forms are used:

8.11.6.1 Slab thickness shall be not less than 1/12 the clear distance between ribs, nor less than 2 in.

8.11.6.2 Reinforcement normal to the ribs shall be provided in the slab as required for flexure, considering load concentrations, if any, but not less than required by 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 significantly impair the strength of the construction.

8.11.8 For joist construction, \( V_c \) 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 nonstructural considerations.

CHAPTER 9— STRENGTH AND SERVICEABILITY REQUIREMENTS

9.1— General

9.1.1 Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated for the following loads combined in accordance with the provisions specified in 9.2.

9.1.1.1 Normal loads—Those loads that are encountered during normal plant operation and shutdown, including \( C_{cr} \).

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 that 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 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, 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 F.

9.1.6 Design of structures and structural members using the load factor combinations and strength-reduction factors of Appendix C shall be permitted. The use of load factor combinations from this chapter in conjunction with strength-reduction factors of Appendix C shall not be permitted.

9.2— Required strength

9.2.1 Required strength \( U \) shall be at least equal to the effects of factored loads in Eq. (9-1) through (9-9). The effect of one or more loads not acting simultaneously shall be investigated.

\[
U = 1.4(D + F + R_o) + T_o \quad (9-1)
\]

\[
U = 1.2(D + F + T_o + R_o) + 1.6(L + H) + 1.4C_{cr} + 0.5(L_r \text{ or } S \text{ or } R) \quad (9-2)
\]

\[
U = 1.2(D + F + R_o) + 0.8(L + H) + 1.4C_{cr} + 1.6(L_r \text{ or } S \text{ or } R) \quad (9-3)
\]

\[
U = 1.2(D + F + R_o) + 1.6(L + H + E_o) \quad (9-4)
\]

\[
U = 1.2(D + F + R_o) + 1.6(L + H + W) \quad (9-5)
\]

\[
U = D + F + 0.8L + C_{cr} + H + T_o + R_o + E_{ss} \quad (9-6)
\]

\[
U = D + F + 0.8L + H + T_o + R_o + W_t \quad (9-7)
\]

\[
U = D + F + 0.8L + C_{cr} + H + (T_a + R_a + 1.2P_a) \quad (9-8)
\]

\[
U = D + F + 0.8L + H + (T_a + R_a + P_a) \quad (9-9)
\]

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 Eq. (9-4) through (9-9). Estimations of these effects shall be based on a realistic assessment of such effects occurring in service.

9.2.3 Load combinations in 9.2.1 shall be evaluated with \( 0.9D \) to assess the adverse effects of reduced dead load. For any other load (for example, \( L \)), if the load reduces the effects of other loads, the corresponding factor for that load shall be taken as 0.9 of the assigned factor, if it can be demonstrated that the load is always present or occurs simul-
9.3.2.1 through 9.3.2.7:

(b) Other reinforced members.......................................0.65

9.2.6 Equation (9-7) shall be satisfied first without the tornado missile load, and Eq. (9-9) shall be first satisfied without \( Y_j, Y_r, \) and \( Y_m \). When considering these impactive and impulsive loads, local section strength 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 F.

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 Eq. (9-7).

9.2.8 For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum prestressing steel jacking force.

9.2.9 In Eq. (9-6), the crane load \( C_{cr} \) may be omitted if probability analysis demonstrates that the simultaneous occurrence of an SSE (DBE) with crane usage is not credible.

9.2.10 In Eq. (9-6) and (9-9), it shall be permitted to reduce the load effects of \( E_{st} \) by 10%, provided the exceedance probability of \( E_{st} \) is equal to or lower than (1) 1.0E-5 (median), for nuclear power plant structures, and (2) 1.0E-4 (mean) for other nuclear facilities structures.

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 the strength-reduction factors \( \phi \) in 9.3.2.

9.3.2 Strength-reduction factor \( \phi \) shall be as given in 9.3.2.1 through 9.3.2.7:

9.3.2.1 Tension-controlled sections as defined in 10.3.4 ........................................................................................................................................0.90

(See also 9.3.2.7)

9.3.2.2 Compression-controlled sections, as defined in 10.3.3:

(a) Members with spiral reinforcement

conforming to 10.9.3 ........................................................0.70

(b) Other reinforced members .............................................0.65

For sections in which the net tensile strain in the extreme tension steel at nominal strength, \( \varepsilon \), is between the limits for compression-controlled and tension-controlled sections, \( \phi \) shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as \( \varepsilon \) increases from the compression-controlled strain limit to 0.005.

9.3.2.3 Shear and torsion..................................................0.75

9.3.2.4 Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models) ...............0.65

9.3.2.5 Post-tensioned anchorage zones..........................0.85

9.3.2.6 Strut-and-tie models (Appendix A), and struts, ties, nodal zones, and bearing areas in such model ......0.75

9.3.2.7 Flexural sections in pretensioned members where strand embedment is less than the development length as provided in 12.9.1.1:

(a) From the end of the member to the end of the transfer length ...............................................................................0.75

(b) From the end of the transfer length to the end of the development length, \( \phi \) shall be permitted to be linearly increased from .................................................0.75 to 0.90

Where bonding of a strand does not extend to the end of the member, strand embedment shall be assumed to begin at the end of the debonded length. See also 12.9.3.

9.3.3 Development lengths specified in Chapter 12 do not require a \( \phi \) factor.

9.4—Design strength for reinforcement

The values of \( f_y \) used in design calculations shall not exceed 60,000 psi, except for prestressing steel.

9.5—Control of deflections

9.5.1 General

9.5.1.1 Deflection limits—Reinforced concrete members subjected to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that adversely affect the strength or serviceability of structural and nonstructural elements.

One-way construction, two-way construction, and shored 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.

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 (c) shall not apply and the members shall be sized such that the calculated deflections are within the required limits.

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 shall be permitted to be computed by factored load analysis and divided by a factor \( \gamma \) to obtain the deflections corresponding to unfactored loads. Unless otherwise determined by computation, the factor \( \gamma \) shall be as follows:

(a) For Eq. (9-1) to (9-5), \( \gamma = 1.3 \)

(b) For Eq. (9-6) to (9-9), \( \gamma = 1.0 \)

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 deflections due to all appropriate nonsustained 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 in accordance with 9.5.2.3, 9.5.3.5, or 9.5.4.3, 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 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 can 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.
9.5.3.2 For slabs without beams, but with drop panels extending in each direction from centerline of support a distance not less than 1/6 the span length in that direction measured center-to-center of supports, and a projection below the slab at least 1/4 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 \( \alpha_f \) 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 size and shape of the panel, conditions of support, and nature of restraints at the panel edges. The modulus of elasticity of concrete, \( E_c \), shall be as specified in 8.5.1. The effective moment of inertia shall satisfy the provisions of 9.5.2.3; other values shall be permitted to be used if they result in predictions of deflections in reasonable agreement with results of comprehensive tests.

9.5.3.5 Computation of long-time deflection—Unless values are obtained by a more comprehensive analysis or test, the additional long-time deflection for normalweight 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, \( I_g \), shall be permitted to be used for Class U flexural members, as defined in 18.3.3.

9.5.4.2 For Class C and Class T flexural members, as defined in 18.3.3, deflection calculations shall be based on a cracked transformed section analysis. It shall be permitted to base computations on a bilinear moment-deflection relationship, or an effective moment of inertia, \( I_e \), as defined by Eq. (9-10).

9.5.4.3 Additional long-term 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.

9.5.4.4 Deflection computed in accordance with 9.5.4.1 or 9.5.4.2, and 9.5.4.3 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 prestressed 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 or 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 LOAD

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 that, for deep beams as defined in 10.7.1, an analysis that considers a nonlinear distribution of strain shall be performed. Alternatively, it shall be permitted to use a strut-and-tie model. See 10.7, 11.8, and Appendix A.

10.2.3 Maximum usable strain at extreme concrete compression fiber shall be assumed equal to 0.003.

10.2.4 Stress in reinforcement below \( f_y \) shall be taken as \( E_y \) times steel strain. For strains greater than that corresponding to \( f_y \), stress in reinforcement shall be considered independent of strain and equal to \( f_y \).

10.2.5 Tensile strength of concrete shall be neglected in axial and flexural calculations of reinforced concrete, except when meeting requirements of 18.4.

10.2.6 The relationship between concrete compressive stress distribution and concrete strain shall be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

10.2.7 Requirements of 10.2.6 are satisfied by an equivalent rectangular concrete stress distribution defined by the following:
10.2.7.1 Concrete stress of 0.85\(f'_c\) shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance \(a = \beta_1 c\) from the fiber of maximum compressive strain.

10.2.7.2 Distance from the fiber of maximum strain to the neutral axis, \(c\), shall be measured in a direction perpendicular to the neutral axis.

10.2.7.3 For \(f'_c\) between 2500 and 4000 psi, \(\beta_1\) shall be taken as 0.85. For \(f'_c\) above 4000 psi, \(\beta_1\) shall be reduced linearly at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi, but \(\beta_1\) shall not be taken less than 0.65.

10.3—General principles and requirements

10.3.1 Design of cross sections subject to flexure or axial loads, or to combined flexure and axial loads, shall be based on stress and strain compatibility using assumptions in 10.2.

10.3.2 Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to the neutral axis, \(\epsilon_{t}\), equal to or less than the compression-controlled strain limit when the concrete in compression reaches its assumed ultimate strain of 0.003.

10.3.3 Sections are compression-controlled if the net tensile strain in the extreme tension steel, \(\epsilon_{t}\), is equal to or less than the compression-controlled strain limit when the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 60 reinforcement, and for all prestressed reinforcement, it shall be permitted to set the compression-controlled strain limit equal to 0.002.

10.3.4 Sections are tension-controlled if the net tensile strain in the extreme tension steel, \(\epsilon_{t}\), is equal to or greater than 0.005 when the concrete in compression reaches its assumed strain limit of 0.003. Sections with \(\epsilon_{t}\) between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.

10.3.5 For nonprestressed flexural members and nonprestressed members with factored axial compressive load less than 0.10\(f'_c\)A\(_g\), \(\epsilon_{t}\) at nominal strength shall not be less than 0.004.

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

10.3.6 Design axial strength \(\phi P_n\) of compression members shall not be taken greater than \(\phi P_{n,max}\), computed by Eq. (10-1) or (10-2).

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

\[
\phi P_{n,max} = 0.85\phi(0.85f'_c(A_g - A_{sl}) + f_yA_{st})
\]  
(10-1)

10.3.6.2 For nonprestressed members with tie reinforcement conforming to 7.10.5

\[
\phi P_{n,max} = 0.80\phi(0.85f'_c(A_g - A_{sl}) + f_yA_{st})
\]  
(10-2)

10.3.7 Members subject to compressive axial load shall be designed for the maximum moment that can accompany the axial load. The factored axial force \(P_n\) at given eccentricity shall not exceed that given in 10.3.6. The maximum factored moment \(M_{u}\) shall be magnified for slenderness effects in accordance with 10.10.

10.4—Distance between lateral supports of flexural members

10.4.1 Spacing of lateral supports for a beam shall not exceed 50 times \(b\), the least width 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, \(A_g\) provided shall not be less than that given by

\[
A_{s,min} = \frac{3f_y}{f_y} b_w d
\]  
(10-3)

and not less than 200\(b_wd\).

10.5.2 For statically determinate members with a flange in tension, \(A_{s,min}\) shall not be less than the value given by Eq. (10-3), except that \(b_w\) is replaced by either 2\(b_w\) or the width of the flange, whichever is smaller.

10.5.3 The requirements of 10.5.1 and 10.5.2 need not be applied if at every section \(A_g\) provided is at least 1/3 greater than that required by analysis.

10.5.4 For structural slabs and footings of uniform thickness, \(A_{s,min}\) in the direction of the span shall be the same as that required by 7.12. Maximum spacing of this reinforcement shall not exceed three times the thickness, nor 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 The spacing of reinforcement closest to the tension face \(s\) shall not exceed that given by

\[
s = 15\left(\frac{40,000}{f_s}\right) - 2.5\epsilon_c
\]  
(10-4)
but not greater than \(12(40,000/\sigma_s)\), where \(\sigma_s\) is the least distance from surface of reinforcement or prestressing steel to the tension face. If there is only one bar or wire nearest to the extreme tension face, \(s\) used in Eq. (10-4) is the width of the extreme tension face.

Calculated stress \(f_s\) in reinforcement closest to the tension face at sustained load shall be computed based on the unfactored moment. It shall be permitted to take \(f_s\) as 40% of \(f_y\). The sustained loads shall include those loads identified in Load Combination (9-2), 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 Where \(h\) of a beam or joist exceeds 36 in., longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member. Skin reinforcement shall extend for a distance \(h/2\) from the tension face. The spacing \(s\) shall be as provided in 10.6.4, where \(c_r\) is the least distance from the surface of the skin reinforcement or prestressing steel to the side face. 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.

10.7—Deep beams

10.7.1 Deep beams are members loaded on one face and supported on the opposite face so that compression struts can develop between the loads and the supports, and have either:
(a) clear spans \(l_n\) equal to or less than four times the overall member depth; or
(b) regions with concentrated loads within twice the member depth from the face of the support.

Deep beams shall be designed either taking into account nonlinear distribution of strain, or by Appendix A. (See also 11.8.1 and 12.10.6.) Lateral buckling shall be considered.

10.7.2 \(V_n\) of deep beams shall be in accordance with 11.8.

10.7.3 Minimum area of flexural tension reinforcement, \(A_{s,t,min}\), shall conform to 10.5.

10.7.4 Minimum horizontal and vertical reinforcement in the side faces of deep beams shall satisfy either A.3.3 or 11.8.4 and 11.8.5.

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, octagonal, or other shaped cross section, it shall be permitted to use a circular section with a diameter equal to the least lateral dimension of the actual shape. Gross area considered, required percentage of reinforcement, and design strength shall be based on that circular section.

10.8.4 Limits of section—For a compression member with a cross section larger than required by considerations of loading, it shall be permitted to base the minimum reinforcement and strength on a reduced effective area \(A_g\) not less than 1/2 the total area.

10.9—Limits for reinforcement of compression members

10.9.1 Area of longitudinal reinforcement, \(A_{s,t}\), for noncomposite compression members shall be not less than \(0.01A_g\) or more than \(0.08A_g\).

10.9.2 Minimum number of longitudinal bars in compression members shall be four for bars within rectangular or circular ties, three for bars within triangular ties, and six for bars enclosed by spirals conforming to 10.9.3.

10.9.3 Volumetric spiral reinforcement ratio \(\rho_s\) shall be not less than the value given by

\[
\rho_s = 0.45 \left( \frac{A_s}{A_{ch}} - 1 \right) \frac{f_{sy}}{f_{yt}}
\]

(10-5)

where the value of \(f_{yt}\) used in Eq. (10-5) shall not exceed 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 members on axial forces and moments from the analyses described in 10.11.
10.11—Magnified moments—general

10.11.1 Factored axial forces \( P_u \), factored moments \( M_1 \) and \( M_2 \) at the ends of the column, and, where required, relative lateral story deflections \( \Delta_o \) 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 \( I \)

<table>
<thead>
<tr>
<th>Member Type</th>
<th>( 0.35I_g )</th>
<th>( 0.70I_g )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Columns</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Walls</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uncracked</td>
<td>( 0.70I_g )</td>
<td></td>
</tr>
<tr>
<td>Cracked</td>
<td>( 0.35I_g )</td>
<td></td>
</tr>
<tr>
<td>Flat plates and flat slabs</td>
<td>( 0.25I_g )</td>
<td></td>
</tr>
</tbody>
</table>

(c) Area

The moments of inertia shall be divided by \((1 + \beta_d)\)

(a) When sustained lateral loads act; or

(b) For stability checks made in accordance with 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 \( r \) for the gross concrete section.

10.11.3 Unsupported length of compression members

10.11.3.1 The unsupported length of a compression member, \( l_u \), 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, \( l_u \) shall be measured to the lower extremity of the capital or haunch in the plane considered.

10.11.4 Columns and stories in structures shall be designated as nonsway or sway columns or stories. The design of columns in nonsway frames or stories shall be based on 10.12. The design of columns in sway frames or stories shall be based on 10.13.

10.11.4.1 It shall be permitted to assume a column in a structure is nonsway if the increase in column end moments due to second-order effects does not exceed 5% of the first-order end moments.

10.11.4.2 It also shall be permitted to assume a story within a structure is nonsway if

\[
Q = \frac{\Sigma P_u \Delta_o}{V_{us} l_c} \quad (10-6)
\]

is less than or equal to 0.05, where \( \Sigma P_u \) and \( V_{us} \) are the total factored vertical load and the horizontal story shear, respectively, in the story being evaluated, and \( \Delta_o \) is the first-order relative lateral deflection between the top and bottom of that story due to \( V_{us} \).

10.11.5 Where an individual compression member in the frame has a slenderness \( k l_u/r \) 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—nonsway frames

10.12.1 For compression members in nonsway frames, the effective length factor \( k \) shall be taken as 1.0, unless analysis shows that a lower value is justified. The calculation of \( k \) shall be based on the values of \( E_c \) and \( I \) given in 10.11.1.

10.12.2 In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy

\[
\frac{k l_u}{r} \leq 34 – 12\left(\frac{M_1}{M_2}\right) \quad (10-7)
\]

where the term \([34 – 12(M_1/M_2)]\) shall not be taken greater than 40. The term \( M_1/M_2 \) is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

10.12.3 Compression members shall be designed for factored axial force \( P_u \) and the moment amplified for the effects of member curvature \( M_c \) as follows

\[
M_c = \delta_{ns} M_2 \quad (10-8)
\]

where

\[
\delta_{ns} = \frac{C_m}{1 – \frac{P_u}{0.75 P_c}} \geq 1.0 \quad (10-9)
\]

\[
P_c = \frac{\pi^2 EI}{(k l_u)^2} \quad (10-10)
\]

\( EI \) shall be taken as

\[
EI = \frac{(0.2 E_c I_{g} + E_s I_{se})}{1 + \beta_d} \quad (10-11)
\]

or

\[
EI = \frac{0.4 E_c I_{g}}{1 + \beta_d} \quad (10-12)
\]

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

\[
C_m = 0.6 + 0.4 \frac{M_1}{M_2} \geq 0.4 \quad (10-13)
\]
where \( M_1/M_2 \) is positive if the column is bent in single curvature. For members with transverse loads between supports, \( C_m \) shall be taken as 1.0.

10.12.3.2 Factored moment \( M_2 \) in Eq. (10-8) shall not be taken less than

\[
M_{2,\text{min}} = P_u (0.6 + 0.03h) \quad (10-14)
\]

about each axis separately, where 0.6 and \( h \) are in inches. For members for which \( M_{2,\text{min}} \) exceeds \( M_2 \), the value of \( C_m \) in Eq. (10-13) shall either be taken equal to 1.0, or shall be based on the ratio of the computed end moments \( M_1 \) to \( M_2 \).

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 the values of \( E_c \) and \( I \) given in 10.11.1 and shall not be less than 1.0.

10.13.2 For compression members not braced against sidesway, it shall be permitted to neglect the effects of slenderness when \( kt_{su}/r \) is less than 22.

10.13.3 Moments \( M_1 \) and \( M_2 \) at the ends of an individual compression member shall be taken as

\[
M_1 = M_{1ns} + \delta_s M_{1s} \quad (10-15)
\]

\[
M_2 = M_{2ns} + \delta_s M_{2s} \quad (10-16)
\]

where \( \delta_s M_{1s} \) and \( \delta_s M_{2s} \) shall be computed according to 10.13.4.

10.13.4 Calculation of \( \delta_s M_s \)

10.13.4.1 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

\[
\delta_s M_s = \frac{M_s}{1 - Q} \geq M_s \quad (10-17)
\]

If \( \delta_s \) 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 \( \delta_s M_s \) as

\[
\delta_s M_s = \frac{M_s}{\Sigma P_u - 0.75 \Sigma P_c} \geq M_s \quad (10-18)
\]

where \( \Sigma P_u \) is the summation for all the factored vertical loads in a story and \( \Sigma P_c \) is the summation for all sway-resisting columns in a story. \( P_u \) is calculated using Eq. (10-10) with \( k \) from 10.13.1 and \( EI \) from Eq. (10-11) or Eq. (10-12).

10.13.5 If an individual compression member has it shall be designed for factored axial force \( P_u \) and moment \( M_c \), calculated using 10.12.3 in which \( M_1 \) and \( M_2 \) are computed in accordance with 10.13.3, \( \beta_d \) as defined for the load combination under consideration, and \( k \) as defined in 10.12.1.

10.13.6 In addition to load combinations 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 factored dead and live loads plus factored 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 \( \Sigma P_u \) for factored dead and live loads shall not exceed 0.60;

(c) When \( \delta_s M_s \) is computed from 10.13.4.3, \( \delta_s \) computed using \( \Sigma P_u \) and \( \Sigma P_c \) corresponding to the factored dead and live loads shall be positive and shall not exceed 2.5.

In (a), (b), and (c) above, \( \beta_d \) shall be taken as the ratio of the maximum factored sustained axial load to the maximum factored axial load.

10.13.7 In sway frames, flexural members shall be designed for the total magnified end moments of the compression members at the joint.

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

If \( f'c' \) of a column is greater than 1.4 times that of the floor system, transmission of load through the floor system shall be provided by 10.15.1, 10.15.2, or 10.15.3.

10.15.1 Concrete of strength specified for the column shall be placed in the floor at the column location. Top surface of the column concrete shall extend 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 6.4.6 and 6.4.7.

10.15.2 Strength of a column through a floor system shall be based on the lower value of concrete strength with vertical dowels and spirals as required.

10.15.3 For columns laterally supported on four sides by beams of approximately equal depth or by slabs, it shall be permitted to base strength of the column on an assumed concrete strength in the column joint equal to 75% of column concrete strength plus 35% of floor concrete strength. In the application of 10.15.3, the ratio of column concrete strength

\[
\frac{E_d}{E_f} \geq \frac{35}{150}
\]

(10-19)
to slab concrete strength shall not be taken greater than 2.5 for design.

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 \( r \) of a composite section shall be not greater than the value given by

\[
 r = \sqrt{\frac{(E, I_y / 5) + E_s I_{sx}}{(E, A_r / 5) + E_s A_{sx}}} \quad (10-20)
\]

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

\[
 EI = \frac{(E, I_y / 5)}{1 + \beta_d} + E_s I_{sx} \quad (10-21)
\]

10.16.6 Structural steel-encased concrete core

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

\[
b \geq \frac{f_{y}}{3 E_s} \text{ for each face of width } b
\]

nor

\[
h \geq \frac{f_{y}}{8 E_s} \text{ for circular sections of diameter } h
\]

10.16.6.2 Longitudinal bars located within the encased concrete core shall be permitted to be used in computing \( A_{sx} \) and \( I_{sx} \).

10.16.7 Spiral reinforcement around structural steel core—A composite member with spirally reinforeced concrete around a structural steel core shall conform to 10.16.7.1 through 10.16.7.5.

10.16.7.1 Specified compressive strength \( f_{c}^\prime \) shall not be less than that given in 1.1.1.

10.16.7.2 Design yield strength of structural steel core shall be the specified minimum yield strength for the grade of structural steel used but not to exceed 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 \( A_{sx} \) and \( I_{sx} \).

10.16.8 Tie reinforcement around structural steel core—A composite member with laterally tied concrete around a structural steel core shall conform to 10.16.8.1 through 10.16.8.8.

10.16.8.1 Specified compressive strength \( f_{c}^\prime \) shall not be less than that given in 1.1.1.

10.16.8.2 Design yield strength \( f_{y} \) shall be the specified minimum yield strength for the 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 0.02 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 reinforcement of equivalent area shall be permitted.

10.16.8.5 Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or 0.5 times the least side dimension of the composite member.

10.16.8.6 Longitudinal bars located within the ties shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.16.8.7 A longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not farther apart than 1/2 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 \( A_{sx} \) for strength but not in computing \( I_{sx} \) for evaluation of slenderness effects.

10.17—Bearing strength

10.17.1 Design bearing strength of concrete shall not exceed \( \phi (0.85 f_{c}^\prime A_1) \), except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area shall be permitted to be multiplied by \( \sqrt{A_2 / A_1} \) but not more than 2.

10.17.2 Section 10.17 does not apply to post-tensioning anchorages.

CHAPTER 11—SHEAR AND TORSION

11.1—Shear strength

11.1.1 Except for members designed in accordance with Appendix A, design of cross sections subject to shear shall be based on

\[
 \phi V_n \geq V_u \quad (11-1)
\]

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

\[
 V_n = V_c + V_s \quad (11-2)
\]
where \( V_c \) is nominal shear strength provided by concrete calculated in accordance with 11.3, 11.4, or 11.12, and \( V_c \) is nominal shear strength provided by shear reinforcement calculated in accordance with 11.5, 11.10.9, or 11.12.

11.1.1.1 In determining \( V_u \), the effect of any openings in members shall be considered.

11.1.1.2 In determining \( V_u \), 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 in accordance with 11.5.6.3, 11.5.6.4, or 11.6.5.2.

11.1.3 Computation of maximum \( V_u \) at supports in accordance with 11.1.3.1 or 11.1.3.2 shall be permitted if all conditions (a), (b), and (c) are satisfied:

(a) Support reaction, in direction of applied shear, introduces compression into the end regions of members;

(b) Loads are applied at or near the top of the member;

(c) 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 \( V_u \) 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 \( V_u \) computed at a distance \( h/2 \).

11.1.4 For deep beams, brackets and corbels, walls, and slabs and footings, the special provisions of 11.8 through 11.12 shall apply.

11.2—Lightweight concrete

Provisions for lightweight aggregate concrete are not provided herein.

11.3—Shear strength provided by concrete for nonprestressed members

11.3.1 \( V_c \) shall be computed by provisions of 11.3.1.1 through 11.3.1.3, 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,

\[
V_c = 2 \sqrt{f_c} b_w d \tag{11-3}
\]

11.3.1.2 For members subject to axial compression,

\[
V_c = 2 \left( 1 + \frac{N_u}{2000 A_g} \right) \sqrt{f_c} b_w d \tag{11-4}
\]

Quantity \( N_u/A_g \) shall be expressed in psi.

11.3.1.3 For members subject to significant axial tension, \( V_c \) shall be taken as zero unless a more detailed analysis is made using 11.3.2.3.

11.3.2 \( V_c \) shall be permitted to 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,

\[
V_c = \left( 1.9 \sqrt{f_c} + 2500 \rho_w \frac{V_{ud}}{M_u} \right) b_w d \tag{11-5}
\]

but not greater than \( 3.5 \sqrt{f_c} b_w d \). When computing \( V_c \) by Eq. (11-5), \( V_{id}/M_u \) shall not be taken greater than 1.0, where \( M_u \) occurs 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 \( V_{id}/M_u \) not then limited to 1.0, where

\[
M_m = M_u - N_u \left( \frac{4h - d}{8} \right) \tag{11-6}
\]

However, \( V_c \) shall not be taken greater than

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

\( N_u/A_g \) 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 zero, where \( N_u \) is negative for tension. \( N_u/A_g \) shall be expressed in psi.

11.3.3 For circular members, the area used to compute \( V_c \) shall be taken as the product of the diameter and effective depth of the concrete section. It shall be permitted to take \( d \) as 0.80 times the diameter of the concrete section.

11.4—Shear strength provided by concrete for prestressed members

11.4.1 For the provisions of 11.4, \( d \) shall be taken as the distance from extreme compression fiber to centroid of prestressed and nonprestressed longitudinal tension reinforcement, if any, but need not be taken less than \( 0.80h \).

11.4.2 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 11.4.3,

\[
V_c = \left( 0.6 \sqrt{f_c} + 700 \frac{V_{ud}}{M_u} \right) b_w d \tag{11-9}
\]

but \( V_c \) need not be taken less than \( 2 \sqrt{f_c} b_w d \). \( V_c \) shall not be taken greater than \( 5 \sqrt{f_c} b_w d \) or the value given in 11.4.4 or
11.4.5. $V_u d_p/M_u$ shall not be taken greater than 1.0, where $M_u$ occurs simultaneously with $V_u$ at the section considered.

11.4.3 $V_c$ shall be permitted to be computed in accordance with 11.4.3.1 and 11.4.3.2, where $V_c$ shall be the lesser of $V_{ci}$ and $V_{cw}$.

11.4.3.1 $V_{ci}$ shall be computed by

$$V_{ci} = 0.6 \sqrt{f_{c}’} b_w d_p + V_d + \frac{V_i M_{cre}}{M_{max}}$$  \hspace{1cm} (11-10)

where $d_p$ need not be taken less than 0.80$h$ and

$M_{cre} = (1/y_i)(6 \sqrt{f_{c}’} + f_{pe} - f_d)$  \hspace{1cm} (11-11)

and values of $M_{max}$ and $V_i$ shall be computed from the load combination causing maximum factored moment to occur at the section. $V_d$ need not be taken less than $1.7 \sqrt{f_{c}’} b_w d_p$ .

11.4.3.2 $V_{cw}$ shall be computed by

$$V_{cw} = (3.5 \sqrt{f_{c}’} + 0.3f_{pc} b_w d_p + V_p$$  \hspace{1cm} (11-12)

where $d_p$ need not be taken less than 0.80$h$.

Alternatively, $V_{cw}$ shall be permitted to 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 the intersection of flange and web when the centroidal axis is in the flange. In composite members, the principal tensile stress shall be computed using the cross section that resists live load.

11.4.4 In a pretensioned member in which the section at a distance $h/2$ from face of support is closer to the end of member than the transfer length of the prestressing steel, the reduced prestress shall be considered when computing $V_{cw}$. This value of $V_{cw}$ shall also be taken as the maximum limit for Eq. (11-9). The prestress force shall be assumed to vary linearly from zero at end of the prestressing steel to a maximum at a distance from end of the prestressing steel equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

11.4.5 In a pretensioned member where bonding of some tendons does not extend to the end of member, a reduced prestress shall be considered when computing $V_c$ in accordance with 11.4.2 or 11.4.3. The value of $V_{cw}$ calculated using the reduced prestress shall also be taken as the maximum limit for Eq. (11-9). The prestress force due to tendons for which bonding does not extend to the end of member shall be assumed to vary linearly from zero at the point at which bonding commences to a maximum at a distance from this point equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

11.5—Shear strength provided by shear reinforcement

11.5.1 Types of shear reinforcement

(a) Stirrups parallel to axis of member;
(b) Welded wire reinforcement with wires located perpendicular to axis of member;
(c) Spirals, circular ties, or hoops.

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

(a) Stirrups making an angle of 45 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.

11.5.2 The values of $f_y$ and $f_{st}$ used in design of shear reinforcement shall not exceed 60,000 psi, except the value shall not exceed 80,000 psi for welded deformed wire reinforcement.

11.5.3 Where the provisions of 11.5 are applied to prestressed members, $d$ shall be taken as the distance from extreme compression fiber to centroid of the prestressed and nonprestressed longitudinal tension reinforcement, if any, but need not be taken less than 0.80$h$.

11.5.4 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance $d$ from extreme compression fiber and shall be developed at both ends according to 12.13.

11.5.5 Spacing limits for shear reinforcement

(a) Stirrups perpendicular to axis of member;
(b) Welded wire reinforcement with wires located perpendicular to axis of member;
(c) Spirals, circular ties, or hoops.

11.5.5.1 Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/2$ in nonprestressed members or $0.75h$ in prestressed members, nor 24 in.

11.5.5.2 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.

11.5.5.3 Where $V_c$ exceeds $4 \sqrt{f_{c}’} b_w d_p$ , maximum spacings given in 11.5.5.1 and 11.5.5.2 shall be reduced by 1/2.

11.5.6 Minimum shear reinforcement

11.5.6.1 A minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where $V_u$ exceeds $0.5f_y V_c$, except:

(a) Slabs and footings;
(b) Beams with $h$ not greater than the largest of 10 in., 2.5 times thickness of flange, or 0.5 the width of web.

11.5.6.2 Minimum shear reinforcement requirements of 11.5.6.1 shall be permitted to be waived if shown by test that required $M_n$ and $V_n$ 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.

11.5.6.3 Where shear reinforcement is required by 11.5.6.1 or for strength and where 11.6.1 allows torsion to be neglected, $A_{v,min}$ for prestressed (except as provided in 11.5.6.4) and nonprestressed members shall be computed by...
where $A_v$ is the area of shear reinforcement within spacing $s$. 

11.6—Design for torsion

Design for torsion shall be in accordance with 11.6.1 through 11.6.7.

11.6.1 Threshold torsion—It shall be permitted to neglect torsion effects if the factored torsional moment $T_u$ is less than:

(a) For nonprestressed members

$$\phi_{fc} \left( \frac{A_{cp}}{p_{cp}} \right)$$

(b) For prestressed members

$$\phi_{fc} \left( \frac{A_{cp}}{p_{cp}} \right) \left( 1 + \frac{f_{pc}}{4A_{s}f_{ct}} \right)$$

(c) For nonprestressed members subjected to an axial tensile or compressive force

$$\phi_{fc} \left( \frac{A_{cp}}{p_{cp}} \right) \left( 1 + \frac{N_u}{4A_{s}f_{ct}} \right)$$

For members cast monolithically with a slab, the overhanging flange width used in computing $A_{cp}$ and $p_{cp}$ shall conform to 13.2.4. For a hollow section, $A_{cp}$ shall be used in place of $A_{cp}$ in 11.6.1, and the outer boundaries of the section shall conform to 13.2.4.

11.6.2 Calculation of factored torsional moment

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 $T_u$ 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 $T_u$ shall be permitted to be reduced to the values given in (a), (b), or (c), as applicable:

(a) For nonprestressed members, at the sections described in 11.6.2.4
\[
\phi 4 \sqrt{f'_c} \left( \frac{A^2}{P_{cp}} \right)^{1/2} \left( 1 + \frac{f_{pc}}{4 \sqrt{f'_c}} \right)
\]

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

\[
\phi 4 \sqrt{f'_c} \left( \frac{A^2}{P_{cp}} \right)^{1/2} \left( 1 + \frac{f_{pc}}{4 \sqrt{f'_c}} \right)
\]

(c) For nonprestressed members subjected to an axial tensile or compressive force

\[
\phi 4 \sqrt{f'_c} \left( \frac{A^2}{P_{cp}} \right)^{1/2} \left( 1 + \frac{N_u}{4 A_{cp} \sqrt{f'_c}} \right)
\]

In (a), (b), or (c), the correspondingly redistributed bending moments and shears in the adjoining members shall be used in the design of these members. For hollow sections, \(A_{cp}\) shall not be replaced with \(A_g\) in 11.6.2.2.

11.6.2.3 Unless determined by a more exact analysis, it shall be permitted to take the torsional loading from a slab as uniformly distributed along the member.

11.6.2.4 In nonprestressed members, sections located less than a distance \(d\) from the face of a support shall be designed for not less than \(T_u\) 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 \(T_u\) 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

\[
\sqrt{\left( \frac{V_u}{b_u d} \right)^2 + \left( \frac{T_u p_h}{1.7 A_{ph}} \right)^2} \leq \phi \left( \frac{V_u}{b_u d} + 8 \sqrt{f'_c} \right) \tag{11-18}
\]

(b) For hollow sections

\[
\sqrt{\left( \frac{V_u}{b_u d} \right)^2 + \left( \frac{T_u p_h}{1.7 A_{ph}} \right)^2} \leq \phi \left( \frac{V_c}{b_u d} + 8 \sqrt{f'_c} \right) \tag{11-19}
\]

For prestressed members, \(d\) shall be determined in accordance with 11.5.3.

11.6.3.2 If the wall thickness varies around the perimeter of a hollow section, Eq. (11-19) shall be evaluated at the location where the left-hand side of Eq. (11-19) is a maximum.

11.6.3.3 If the wall thickness is less than \(A_{oh}/p_h\), the second term in Eq. (11-19) shall be taken as

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

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 The values of \(f_p\) and \(f_{yl}\) used for design of torsional reinforcement shall not exceed 60,000 psi.

11.6.3.5 Where \(T_u\) exceeds the threshold torsion, design of the cross section shall be based on

\[
\phi T_u \geq T_u \tag{11-20}
\]

11.6.3.6 \(T_n\) shall be computed by

\[
T_n = \frac{2 A_o A_t f_{yl} \cot \theta}{s} \tag{11-21}
\]

where \(A_o\) shall be determined by analysis except that it shall be permitted to take \(A_o\) equal to \(0.85 A_{oh}\); \(\theta\) shall not be taken smaller than 30 degrees nor larger than 60 degrees. It shall be permitted to take \(\theta\) 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 area of longitudinal reinforcement to resist torsion, \(A_t\), shall not be less than

\[
A_t = \frac{A_o f_{yl}}{s} \left( \frac{f_{yl}}{f_y} \right) \cot^2 \theta \tag{11-22}
\]

where \(\theta\) 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; \(f_{yl}\) refers to closed transverse torsional reinforcement, and \(f_y\) refers to longitudinal torsional reinforcement.

11.6.3.8 Reinforcement required for torsion shall be added to that required for the shear, moment, and axial force that act in combination with the torsion. The most restrictive requirements for reinforcement spacing and placement shall be met.

11.6.3.9 It shall be permitted to reduce the area of longitudinal torsion reinforcement in the flexural compression zone by an amount equal to \(M_u/0.9 f_y l\), where \(M_u\) occurs at the section simultaneously 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 prestressing steel at each section shall resist \(M_u\) at that section plus an additional concentric longitudinal tensile force equal to \(A_t f_y\), based on \(T_u\) at that section;
(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;
(b) A closed cage of welded wire reinforcement with transverse wires perpendicular to the axis of the member;
(c) In nonprestressed beams, spiral reinforcement.

11.6.4.2 Transverse torsional reinforcement shall be anchored by one of the following:
(a) A 135-degree standard hook, or seismic hook as defined in 21.1, around a longitudinal bar;
(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 0.5Aoh/Ph.

11.6.5 Minimum torsion reinforcement

11.6.5.1 A minimum area of torsional reinforcement shall be provided in all regions where Tu exceeds the threshold torsion given in 11.6.1.

11.6.5.2 Where torsional reinforcement is required by 11.6.5.1, the minimum area of transverse closed stirrups shall be computed by

\[(A_v + 2A_t) = 0.75 \frac{f_y b_w s}{f_y'} \] \hspace{1cm} (11-23)

but shall not be less than \((500 b_w s)/f_y'\).

11.6.5.3 Where torsional reinforcement is required by 11.6.5.1, the minimum total area of longitudinal torsional reinforcement, \(A_{t,min}\), shall be computed by

\[A_{t,min} = \frac{5.5f_y' A_{cp}}{f_y} \left(\frac{A_v}{s}\right) p_h f_y' \] \hspace{1cm} (11-24)

where \(A_v/s\) shall not be taken less than \(25b_w f_y' f_y\) refers to closed transverse torsional reinforcement, and \(f_y'\) refers to longitudinal torsional reinforcement.

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. Longitudinal bars shall have a diameter at least 0.042 times the stirrup spacing, but not less than 3/8 in.

11.6.6.3 Torsional reinforcement shall be provided for a distance of at least \((b_t + d)\) beyond the point required by analysis.

11.6.7 Alternative design for torsion—For torsion design of solid sections within the scope of this Code with an aspect ratio \(h/b_t\) of three or greater, it shall be permitted to use another procedure, the adequacy of which has been shown by analysis and substantial agreement with results of comprehensive tests. Sections 11.6.4 and 11.6.6 shall apply.

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 Where shear-friction reinforcement is perpendicular to the shear plane, \(V_n\) shall be computed by

\[V_n = A_{vf} f_y' \mu \] \hspace{1cm} (11-25)

where \(\mu\) is coefficient of friction in accordance with 11.7.4.3.

11.7.4.2 Where shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in shear-friction reinforcement, \(V_n\) shall be computed by

\[V_n = A_{vf} f_y' (\mu \sin \alpha + \cos \alpha) \] \hspace{1cm} (11-26)

where \(\alpha\) is angle between shear-friction reinforcement and shear plane.

11.7.4.3 The coefficient of friction \(\mu\) in Eq. (11-25) and Eq. (11-26) shall be taken as:
- 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 \(V_n\) shall not be taken greater than the smaller of \(0.2f'_c A_c\) and \(800A_c\), where \(A_c\) is area of concrete section resisting shear transfer.
11.7.6 The value of $f_p$ used for design 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 $A_{sf} f_p$, the force in the shear-friction reinforcement, when calculating required $A_{sf}$.

11.7.8 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop $f_p$ 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 $\mu$ 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—Deep beams

11.8.1 The provisions of 11.8 shall apply to members with $t_n$ not exceeding four times the overall member depth or regions of beams with concentrated loads within twice the member depth from the support that are loaded on one face and supported on the opposite face so that compression struts can develop between the loads and supports. See also 12.10.6.

11.8.2 Deep beams shall be designed using either nonlinear analysis as permitted in 10.7.1, or Appendix A.

11.8.3 $V_u$ for deep beams shall not exceed $10 f_p b_w d$.

11.8.4 The area of shear reinforcement perpendicular to the flexural tension reinforcement, $A_v$, shall not be less than 0.0025$b_w s$, and $s$ shall not exceed the smaller of $d/\bar{S}$ and 12 in.

11.8.5 The area of shear reinforcement parallel to the flexural tension reinforcement, $A_{vh}$, shall not be less than 0.0015$b_w s^2$, and $s^2$ shall not exceed the smaller of $d/\bar{S}$ and 12 in.

11.8.6 It shall be permitted to provide reinforcement satisfying A.3.3 instead of the minimum horizontal and vertical reinforcement specified in 11.8.4 and 11.8.5.

11.9—Special provisions for brackets and corbels

11.9.1 Brackets and corbels with a shear-span-to-depth ratio $a_n/d$ less than 2 shall be permitted to be designed using Appendix A. Design shall be permitted using 11.9.3 and 11.9.4 for brackets and corbels with:

(a) $a_n/d$ not greater than 1, and

(b) subject to factored horizontal tensile force $N_{uc}$ not larger than $V_u$.

The requirements of 11.9.2, 11.9.3.2.1, 11.9.5, 11.9.6, and 11.9.7 shall apply to design of brackets and corbels. Effective depth $d$ shall be determined at the face of the support.

11.9.2 Depth at outside edge of bearing area shall not be less than 0.5$d$.

11.9.3 Section at face of support shall be designed to simultaneously resist $V_u$, a factored moment $[V_u a_v + N_{uc}(h - d)]$, and a factored horizontal tensile force $N_{uc}$.

11.9.3.1 In all design calculations in accordance with 11.9, $\phi$ shall be taken equal to 0.75.

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

11.9.3.2.1 Shear strength $V_u$ shall not be taken greater than the smaller of $0.2f' b_w d$ and $800w_d$.

11.9.3.3 Reinforcement $A_f$ to resist factored moment $\left[V_n a_v + N_{uc}(h - d)\right]$ shall be computed in accordance with 10.2 and 10.3.

11.9.3.4 Reinforcement $A_n$ to resist factored tensile force $N_{uc}$ shall be determined from $a_n f_p N_{uc}$. Factored tensile force $N_{uc}$ shall not be taken less than $0.2V_u$ unless special provisions are made to avoid tensile forces. $N_{uc}$ shall be regarded as a live load even if tension results from restraint of creep, shrinkage, or temperature change.

11.9.3.5 Area of primary tension reinforcement $A_{sc}$ shall not be less than the larger of $(A_f + A_n)$ and $(2A_{sf}/3 + A_n)$.

11.9.4 Total area $A_h$ of closed stirrups or ties parallel to primary tension of reinforcement shall not be less than 0.4($A_{sc} - A_f$). Distribute $A_h$ uniformly within $(2/3)d$ adjacent to primary tension reinforcement.

11.9.5 $A_{sc}/bd$ shall not be less than 0.04($f'/f_p$).

11.9.6 At front face of bracket or corbel, primary tension reinforcement 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 $f_p$ of primary tension reinforcement;

(b) By bending primary tension reinforcement back to form a horizontal loop; or

(c) By some other means of positive anchorage.

11.9.7 Bearing area on bracket or corbel shall not project beyond straight portion of primary tension reinforcement, 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 in-plane shear forces in a wall shall be in accordance with 11.10.2 through 11.10.9. Alternatively, it shall be permitted to design walls with a height not exceeding two times the length of the wall for horizontal shear forces in accordance with Appendix A and 11.10.9.2 through 11.10.9.5.

11.10.2 Design of horizontal section for shear in plane of wall shall be based on Eq. (11-1) and (11-2), where $V_s$ shall be in accordance with 11.10.5 or 11.10.6, and $V_s$ shall be in accordance with 11.10.9.

11.10.3 $V_n$ at any horizontal section for shear in plane of wall shall not be taken greater than $10 f' h$, where $h$ is thickness of wall, and $d$ is defined in 11.10.4.

11.10.4 For design for horizontal shear forces in plane of wall, $d$ shall be taken equal to $0.8e_w$. 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, \( V_e \) shall not be taken greater than \( 2 \sqrt{f'_c} \) \( hd \) for walls subject to axial compression, or \( V_e \) shall not be taken greater than the value given in 11.3.2.3 for walls subject to axial tension.

11.10.6 \( V_e \) shall be permitted to be the lesser of the values computed from Eq. (11-29) and (11-30).

\[
V_e = 3.3 \sqrt{f'_c} \frac{hd}{4l_w} \quad (11-29)
\]

or

\[
V_e = \left[ 0.6 \sqrt{f'_c} + \frac{l_w \left( 1.25 \sqrt{f'_c} + 0.2 \frac{N_u}{V_u} \frac{h}{l_w} \right)}{M_u - \frac{l_w}{2} \frac{V_u}{V_c}} \right] \frac{hd}{l_w} \quad (11-30)
\]

where \( l_w \) is the overall length of the wall, and \( N_u \) is positive for compression and negative for tension. If \( (M_u/N_u - \frac{l_w}{2}) \) is negative, Eq. (11-30) shall not apply.

11.10.7 Sections located closer to wall base than a distance \( l_w/2 \) or 1/2 the wall height, whichever is less, shall be permitted to be designed for the same \( V_e \) as that computed at a distance \( l_w/2 \) or 1/2 the height.

11.10.8 Where \( V_u \) is less than \( 0.5 \phi V_c \), reinforcement shall be provided in accordance with 11.10.9 or in accordance with Chapter 14. Where \( V_u \) exceeds \( 0.5 \phi V_c \), 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 \( V_u \) exceeds \( \phi V_c \), horizontal shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where \( V_s \) shall be computed by

\[
V_s = \frac{A_v f_y d}{s} \quad (11-31)
\]

where \( A_v \) is area of horizontal shear reinforcement within spacing \( s \), and \( d \) is determined in accordance with 11.10.4. Vertical shear reinforcement shall be provided in accordance with 11.10.9.4.

11.10.9.2 Ratio of horizontal shear reinforcement area to gross concrete area of vertical section, \( \rho_h \), shall not be less than 0.0025.

11.10.9.3 Spacing of horizontal shear reinforcement shall not exceed the smallest of \( l_w/5 \), \( 3h \), and 18 in., where \( l_w \) is the overall length of the wall.

11.10.9.4 Ratio of vertical shear reinforcement area to gross concrete area of horizontal section, \( \rho_v \), shall not be less than the larger of

\[
\rho_v = 0.0025 + 0.5 \left( 2.5 - \frac{h_w}{l_w} \right) (\rho_t - 0.0025) \quad (11-32)
\]

and 0.0025, but need not be greater than \( \rho_t \) required by 11.10.9.1. In Eq. (11-32), \( l_w \) is the overall length of the wall, and \( h_w \) is the overall height of the wall.

11.10.9.5 Spacing of vertical shear reinforcement shall not exceed the smallest of \( l_w/3 \), \( 3h \), and 18 in., where \( l_w \) is the overall length of the wall.

11.11—Transfer of moments to columns

11.11.1 When gravity load, wind, earthquake, or other lateral forces cause transfer of moment at connections of framing elements to columns, the shear resulting from moment transfer shall be considered in the design of lateral reinforcement in the columns.

11.11.2 Except for connections not part of a primary seismic load-resisting system that are restrained on four sides by beams or slabs of approximately equal depth, connections shall have lateral reinforcement not less than that required by Eq. (11-13) within the column for a depth not less than that of the deepest connection of framing elements to the columns. See also 7.9.

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 For two-way action, each of the critical sections to be investigated shall be located so that its perimeter \( b_o \) is a minimum but need not approach closer than \( d/2 \) to

(a) Edges or corners of columns, concentrated loads, or reaction areas; and

(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). \( V_e \) shall be computed in accordance with 11.12.2.1, 11.12.2.2, or 11.12.3.1. \( V_e \) 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, \( V_e \) shall be the smallest of (a), (b), and (c)

\[
(a) V_e = \left( 2 + \frac{2}{\beta} \right) \sqrt{f'_c} b_o d \quad (11-33)
\]

where \( \beta \) is the ratio of long side to short side of the column, concentrated load or reaction area.
where \( \alpha_s \) is 40 for interior columns, 30 for edge columns, 20 for corner columns; and

\[
(c) \quad V_c = 4\sqrt{\frac{f_c}{\beta_p}} b_o d \quad (11-35)
\]

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

\[
V_c = (\beta_p \sqrt{\frac{f_c}{\beta_p}} + 0.3 f_{pc}) b_o d + V_p \quad (11-36)
\]

where \( \beta_p \) is the smaller of 3.5 and \((\alpha_d b_o + 1.5)\), \( \alpha_s \) is 40 for interior columns, 30 for edge columns, and 20 for corner columns, \( b_o \) is perimeter of critical section defined in 11.12.1.2, \( f_{pc} \) is taken as the average value of \( f_{pc} \) for the two directions, and \( V_p \) is the vertical component of all effective prestress forces crossing the critical section. \( V_c \) shall be permitted to be computed by Eq. (11-36) if the following are satisfied; otherwise, 11.12.2.1 shall apply:

(a) No portion of the column cross section shall be closer to a discontinuous edge than four times the slab thickness; and

(b) The value of \( \sqrt{\frac{f_c}{\beta_p}} \) used in Eq. (11-36) shall not be taken greater than \( 70 \) psi; and

(c) In each direction, \( f_{pc} \) 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 \( f_{m2} \):

\[
V_c = V_{c1} + V_{c2} \quad (11-36a)
\]

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 \( 0.9 \rho_1' f_y \),

\[
V_{c1} = \left( 2 + \frac{4}{\beta_c} \right) \sqrt{\frac{f_c}{\beta_c}} b_1' h \left( 1 + \frac{0.25 f_{m1}}{\rho_1' f_y} \right) \quad (11-36b)
\]

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

\[
V_{c1} = 0.5 \sqrt{\frac{f_c}{\beta_c}} b_1' h \quad (11-36c)
\]

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. (11-36), \( f_{m1} \) and \( b_1' \) shall be used in place of \( f_{pc} \) and \( b_o \), 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 \( b_o \).

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

\[
V_{c2} = \left( 2 + \frac{4}{\beta_c} \right) \sqrt{\frac{f_c}{\beta_c}} b_2' h \left( 1 + \frac{0.25 f_{m2}}{\rho_2' f_y} \right) \quad (11-36d)
\]

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

\[
V_{c2} = 0.5 \sqrt{\frac{f_c}{\beta_c}} b_2' h \quad (11-36e)
\]

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. (11-36), \( f_{m2} \) and \( b_2' \) shall be used in place of \( f_{pc} \) and \( b_o \), 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 that \( b_2' \) shall be used in place of \( b_o \).

#### 11.12.2.3.3 When \( V_u \) does not exceed \( \Phi V_c \) and \( f_{m1} \) is a tensile stress that exceeds \( 0.6 \rho_1' f_y \), then \( \rho_1' \) provided shall not be less than

\[
\rho_1' = |f_{m1}|/(0.9 f_y) + V_u/(0.85 f_y b_1' h) \quad (11-36f)
\]

In addition, \( \rho_1' \) 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 \( \Phi V_c \) and \( f_{m2} \) is a tensile stress that exceeds \( 0.6 \rho_2' f_y \), then \( \rho_2' \) provided shall not be less than

\[
\rho_2' = |f_{m2}|/(0.9 f_y) + V_u/(0.85 f_y b_2' h) \quad (11-36g)
\]

In addition, \( \rho_2' \) 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 and single- or multiple-leg stirrups shall be permitted in slabs and footings with \( d \) greater than or equal to \( 6 \) in., but not less than \( 16 \) times the shear reinforcement bar diameter. Shear reinforcement shall be in accordance with 11.12.3.1 through 11.12.3.4.

#### 11.12.3.1 \( V_n \) shall be computed by Eq. (11-2), where \( V_c \) shall not be taken greater than \( 2 \sqrt{f_c} b_o d \), and \( V_s \) shall be calculated in accordance with 11.5. In Eq. (11-15), \( A_d \) shall be taken as the cross-sectional area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the column section.

#### 11.12.3.2 \( V_n \) shall not be taken greater than \( 6 \sqrt{f_c} b_o d \).

#### 11.12.3.3 The distance between the column face and the first line of stirrup legs that surround the column shall not exceed \( d/2 \). The spacing between adjacent stirrup legs in the first line of shear reinforcement shall not exceed \( 2d \) measured in a direction parallel to the column face. The spacing between successive lines of shear reinforcement that surround the column shall not exceed \( d/2 \) measured in a direction perpendicular to the column face.

#### 11.12.3.4 Slab shear reinforcement shall satisfy the anchorage requirements of 12.13 and shall engage the longitudinal flexural reinforcement in the direction being considered.

#### 11.12.4 Shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) shall be permitted in slabs. The provisions of 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 \( \alpha \) between the flexural stiffness of each shearhead arm and that of the surrounding composite cracked slab section of width \( c_2 + d \) shall not be less than 0.15.

11.12.4.6 Plastic moment strength \( M_p \) required for each arm of the shearhead shall be computed by

\[
M_p = \frac{V_n}{2\phi n} \left[ h_v + \alpha_v \left( \ell_v - \frac{c_1}{2} \right) \right] \quad (11-37)
\]

where \( \phi \) is for tension-controlled members, \( n \) is number of shearhead arms, and \( \ell_v \) is 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 3/4 the distance \( [2\ell_v - (c_1/2)] \) from the column face to the end of the shearhead arm. The critical section shall be located so that its perimeter \( h_o \) is a minimum, but need not be closer than the perimeter defined in 11.12.1.2(a).

11.12.4.8 \( V_n \) shall not be taken greater than \( 4\sqrt{\gamma f b_o d} \) on the critical section defined in 11.12.4.7. When shearhead reinforcement is provided, \( V_n \) shall not be taken greater than \( 7\sqrt{\gamma f b_o d} \) on the critical section defined in 11.12.1.2(a).

11.12.4.9 Moment resistance \( M_v \) contributed to each slab column strip by a shearhead shall not be taken greater than

\[
M_v = \frac{\phi n V_c^2 (\ell_v - \frac{c_1}{2})}{2n} \quad (11-38)
\]

where \( \phi \) is for tension-controlled members, \( n \) is number of shearhead arms, and \( \ell_v \) is length of each shearhead arm actually provided. However, \( M_v \) shall not be taken larger than the smallest of:

(a) 30% of the total factored moment required for each slab column strip;
(b) The change in column strip moment over the length \( \ell_v \);
(c) \( M_p \) computed by Eq. (11-37).

11.12.4.10 When unbalanced moments are considered, the shearhead must have adequate anchorage to transmit \( M_p \) to the column.

11.12.5 Openings in slabs—When openings in slabs are located at a distance less than 10 times the slab thickness from a concentrated load or reaction area, or when openings in flat slabs are located within column strips as defined in Chapter 13, the critical slab sections for shear defined in 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 1/2 of that defined in 11.12.5.1.

11.12.6 Transfer of moment in slab-column connections

11.12.6.1 Where gravity load, wind, earthquake, or other lateral forces cause transfer of unbalanced moment \( M_u \) between a slab and column, \( \gamma_f M_u \) shall be transferred by flexure in accordance with 13.5.3. The remainder of the unbalanced moment, \( \gamma_v M_u \), shall be considered to be transferred by eccentricity of shear about the centroid of the critical section defined in 11.12.1.2 where

\[
\gamma_v = (1 - \gamma) \quad (11-39)
\]

11.12.6.2 The shear stress resulting from moment transfer by eccentricity of shear shall be assumed to vary linearly about the centroid of the critical sections defined in 11.12.1.2. The maximum shear stress due to \( V_n \) and \( M_u \) shall not exceed \( \phi V_n \):

(a) For members without shear reinforcement

\[
\phi V_n = \phi V_c (b_o d) \quad (11-40)
\]

where \( V_c \) is as defined in 11.12.2.1 or 11.12.2.2.

(b) For members with shear reinforcement other than shearheads

\[
\phi V_n = \phi (V_c + V_h) (b_o d) \quad (11-41)
\]

where \( V_c \) and \( V_h \) are defined in 11.12.3.1. The design shall take into account the variation of shear stress around the column. The shear stress due to factored shear force and moment shall not exceed \( \phi (2\sqrt{\gamma f c_2'}) \) at the critical section located \( d/2 \) outside the outermost line of stirrup legs that surround the column.

11.12.6.3 When shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) is provided, the sum of the shear stresses due to vertical load acting on the critical section defined by 11.12.4.7 and the shear stresses resulting from moment transferred by eccentricity of shear about the centroid of the critical section defined in 11.12.1.2(a) and 11.12.1.3 shall not exceed \( \phi (4\sqrt{\gamma f c_2'}) \).

CHAPTER 12—DEVELOPMENT AND SPLICES OF REINFORCEMENT

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 shall not be less than 12 in.

12.2.2 For deformed bars or deformed wire, \( l_d \) shall be as follows:

\[
\begin{array}{|c|c|c|}
\hline
& \text{No. 6 and smaller bars and deformed wires} & \text{No. 7 and larger bars} \\
\hline
\text{Clear spacing of bars or wires being developed or spliced not less than} & \left( \frac{f_y \psi \psi_x}{25 \sqrt{f'_c}} \right) d_b & \left( \frac{f_y \psi \psi_x}{20 \sqrt{f'_c}} \right) d_b \\
\text{Clear spacing of bars or wires being developed or spliced not less than} & \left( \frac{f_y \psi \psi_x}{50 \sqrt{f'_c}} \right) d_b & \left( \frac{3 f_y \psi \psi_x}{40 \sqrt{f'_c}} \right) d_b \\
\text{Other cases} & \left( \frac{3 f_y \psi \psi_x}{40 \sqrt{f'_c}} \right) d_b & \left( \frac{3 f_y \psi \psi_x}{40 \sqrt{f'_c}} \right) d_b \\
\hline
\end{array}
\]

12.2.3 For deformed bars or deformed wire, \( l_d \) shall be

\[
l_d = \left( \frac{3}{40} \frac{f_y \psi \psi_x}{\sqrt{f'_c}} \left( \frac{e_b + K_{tr}}{d_b} \right) \right) d_b
\]

(12-1)

in which the term \( (e_b + K_{tr})/d_b \) shall not be taken greater than 2.5, and

\[
K_{tr} = \frac{A_{tr} f_y}{1500 s n}
\]

(12-2)

where \( n \) is the number of bars or wires being spliced or 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.

12.2.4 The factors used in the expressions for development of deformed bars and deformed wires in tension in 12.2 are as follows:

(a) Where horizontal reinforcement is placed such that more than 12 in. of fresh concrete is cast below the development length or splice, \( \psi_f = 1.3 \). For other situations, \( \psi_f = 1.0 \).

(b) For epoxy-coated bars or wires with cover less than \( 3d_b \), or clear spacing less than \( 6d_b \), \( \psi_e = 1.5 \). For all other epoxy-coated bars or wires, \( \psi_e = 1.2 \). For uncoated reinforcement, \( \psi_e = 1.0 \).

However, the product \( \psi_f \psi_e \) need not be greater than 1.7.

(c) For No. 6 and smaller bars and deformed wires, \( \psi_s = 0.8 \). For No. 7 and larger bars, \( \psi_s = 1.0 \).

12.2.5 Excess reinforcement—Reduction in \( l_d \) shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for \( f_y \) is specifically required or the reinforcement is part of the lateral load resisting system designed under provisions of Chapter 21... (\( A_s \) required)/(\( A_s \) provided)

12.3—Development of deformed bars and deformed wire in compression

12.3.1 Development length for deformed bars and deformed wire in compression, \( l_{dc} \), shall be determined from 12.3.2 and applicable modification factors of 12.3.3, but \( l_{dc} \) shall not be less than 8 in.

12.3.2 For deformed bars and deformed wire, \( l_{dc} \) shall be taken as the larger of \( (0.02 f_y / \sqrt{f'_c}) d_b \) and \( (0.0003 f_y d_b) \), where the constant 0.0003 carries the unit of in.\(^2\)/lb.

12.3.3 Length \( l_{dc} \) in 12.3.2 shall be permitted to be multiplied by the applicable factors for:

(a) Reinforcement in excess of that required by analysis..................................\((A_s \text{ required})/(A_s \text{ provided})\)

(b) 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 for deformed bars in tension terminating in a standard hook (see 7.1), \( l_{dh} \), shall be determined from 12.5.2 and the applicable modification factors of 12.5.3, but \( l_{dh} \) shall not be less than the smaller of \( 8d_b \) and 6 in.

12.5.2 For deformed bars, \( l_{dh} \) shall be \( (0.02 \psi_s f_y / \sqrt{f'_c}) d_b \) with \( \psi_s \) taken as 1.2 for epoxy-coated reinforcement. For other cases, \( \psi_s \) shall be taken as 1.0.

12.5.3 Length \( l_{dh} \) in 12.5.2 shall be permitted to be multiplied by the following applicable factors:

(a) For No. 11 bar and smaller hooks with side cover (normal to plane of hook) not less than 2-1/2 in., and for 90-degree hook with cover on bar extension beyond hook not less than 2 in................................. 0.7

(b) For 90-degree hooks of No. 11 and smaller bars that are either enclosed within ties or stirrups parallel to the bar being developed, spaced not greater than \( 3d_b \) along \( l_{dh} \); or enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than \( 3d_b \) along the length of the tail extension of the hook plus bend.................................................... 0.8
(c) For 180-degree hooks of No. 11 and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than 3\(d_b\) along \(\ell_{dh}\) ................................................................. 0.8

(d) Where anchorage or development for \(f_y\) is not specifically required, reinforcement in excess of that required by analysis......................... \((A_e \text{ required})/(A_e \text{ provided})\)

In 12.5.3(b) and 12.5.3(c), \(d_b\) is the diameter of the hooked bar, and the first tie or stirrup shall enclose the bent portion of the hook, within 2\(d_b\) of the outside of the bend.

12.5.4 For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 2-1/2 in., the hooked bar shall be enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than 3\(d_b\) along \(\ell_{dh}\). The first tie or stirrup shall enclose the bent portion of the hook, within 2\(d_b\) of the outside of the bend, where \(d_b\) is the diameter of the hooked bar. For this case, the factors of 12.5.3(b) and (c) 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 permitted to be used as anchorage.

12.6.2 Mechanical anchorages shall be designed in accordance with Appendix D.

12.7—Development of welded deformed wire reinforcement in tension

12.7.1 Development length in tension for welded deformed wire reinforcement, \(\ell_d\), measured from the point of critical section to the end of wire shall be computed as the product of \(\ell_d\), from 12.2.2 or 12.2.3, times a welded wire reinforcement factor from 12.7.2 or 12.7.3. It shall be permitted to reduce \(\ell_d\) in accordance with 12.2.5 when applicable, but \(\ell_d\) shall not be less than 8 in. except in computation of lap splices by 12.18. When using the welded wire reinforcement factor from 12.7.2, it shall be permitted to use an epoxy-coating factor \(\psi\) of 1.0 for epoxy-coated welded wire reinforcement in 12.2.2 and 12.2.3.

12.7.2 For welded deformed wire reinforcement with at least one cross wire within \(\ell_d\) and not less than 2 in. from the point of the critical section, the welded wire reinforcement factor shall be the greater of

\[
\frac{f_y - 35,000}{f_y}
\]

and

\[
\frac{5d_b}{s}
\]

but not greater than 1.0, where \(s\) is the spacing between the wires to be developed.

12.7.3 For welded deformed wire reinforcement with no cross wires within \(\ell_d\) or with a single cross wire less than 2 in. from the point of the critical section, the welded wire reinforcement factor shall be taken as 1.0, and \(\ell_d\) shall be determined as for deformed wire.

12.7.4 When any plain wires are present in the welded deformed wire reinforcement in the direction of the development length, the reinforcement shall be developed in accordance with 12.8.

12.8—Development of welded plain wire reinforcement in tension

Yield strength of welded plain wire reinforcement 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, \(\ell_d\) shall not be less than

\[
\ell_d = 0.27\frac{A_e}{s} \left(\frac{f_y}{\sqrt{f_{se}}}\right)
\]

(12-3)

where \(\ell_d\) is measured from the point of the critical section to the outermost cross wire, and \(s\) is the spacing between the wires to be developed. Where reinforcement provided is in excess of that required, \(\ell_d\) may be reduced in accordance with 12.2.5. Length \(\ell_d\) 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 Except as provided in 12.9.1.1, seven-wire strand shall be bonded beyond the critical section, a distance not less than

\[
\ell_d = \left(\frac{f_{se}}{3000}\right) d_b + \left(\frac{f_{ps} - f_{se}}{1000}\right) d_b
\]

(12-4)

The expressions in parentheses are used as constants without units.

12.9.1.1 Embedment less than \(\ell_d\) shall be permitted at a section of a member provided the design strand stress at that section does not exceed values obtained from the bilinear relationship defined by Eq. (12-4).

12.9.2 Limiting the investigation to cross sections nearest each end of the member that are required to develop full design strength under specified factored loads shall be permitted except where bonding of one or more strands does not extend to the end of the member, or where concentrated loads are applied within the strand development length.

12.9.3 Where bonding of a strand does not extend to end of member, and design includes tension at service load in precompressed tensile zone as permitted by 18.4.2, \(\ell_d\) 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 \( d \) 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 \( \ell_b \) beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

12.10.5 Flexural reinforcement shall not be terminated in a tension zone unless 12.10.5.1, 12.10.5.2, or 12.10.5.3 is satisfied.

12.10.5.1 \( V_u \) at the cutoff point does not exceed \( (2/3)\psi V_n \).

12.10.5.2 Stirrup area in excess of that required for shear and torsion is provided along each terminated bar or wire over a distance of \( 3/4d \) from the termination point. Excess stirrup area shall be not less than \( 60b_w s / f_{yt} \). Spacing \( s \) shall not exceed \( (db_b / f_{yt}) \).

12.10.5.3 For No. 11 bars and smaller, continuing reinforcement provides double the required for flexure at the cutoff point and \( V_u \) does not exceed \( (3/4)\psi V_n \).

12.10.6 Adequate anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets; deep flexural members; or members in which tension reinforcement is not parallel to compression face. See 12.11.4 and 12.12.4 for deep flexural members.

12.11—Development of positive moment reinforcement

12.11.1 At least \( 1/3 \) the positive moment reinforcement in simple members and \( 1/4 \) 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 \( f_y \) 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 \( \ell_d \) computed for \( f_y \) by 12.2 satisfies Eq. (12-5); except, Eq. (12-5) need not be satisfied for reinforcement terminating beyond centerline of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook.

\[ \ell_d \leq \frac{M_n}{V_u} + \ell_a \]  

(12-5)

where \( M_n \) is calculated assuming all reinforcement at the section to be stressed to \( f_y \); \( V_u \) is calculated at the section; \( \ell_d \) at a support shall be the embedment length beyond center of support; or \( \ell_a \) at a point of inflection shall be limited to \( d \) 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 beams, positive moment tension reinforcement shall be anchored to develop \( f_y \) in tension at the face of the support except that if design is carried out using Appendix A, the positive moment tension reinforcement shall be anchored in accordance with A.4.3. At interior supports of deep beams, positive moment tension reinforcement shall be continuous or be spliced with that of the adjacent spans.

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 \( 1/3 \) the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than \( d, 12d_b, \) or \( \ell_n/16 \), 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 as close to the compression and tension surfaces of the member as cover requirements and proximity of other reinforcement permits.

12.13.2 Ends of single leg, simple U-stirrups, or multiple U-stirrups shall be anchored as required by 12.13.2.1 through 12.13.2.5.

12.13.2.1 For No. 5 bar and D31 wire, and smaller, and for No. 6, No. 7, and No. 8 bars with \( f_{yt} \) 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 \( f_{yt} \) 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 \( 0.014d_b f_{yt} / \sqrt{\ell'} \).

12.13.2.3 For each leg of welded plain wire reinforcement forming simple U-stirrups, either:

(a) Two longitudinal wires spaced at a 2 in. spacing along the member at the top of the U; or
(b) One longitudinal wire located not more than \( d/4 \) from the compression face and a second wire closer to the compression face and spaced not less than 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 wire reinforcement, 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 \( d/2 \). 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 mid-depth \( \frac{d}{2} \) as specified for development length in 12.2 for that part of \( f_{yd} \) required to satisfy Eq. (11-17).

12.13.5 Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when length of laps are \( 1.3d \). In members at least 18 in. deep, such splices with \( A_b f_{yd} \) 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 the smaller of 1/5 the required lap splice length, and 6 in.

12.14.3 Mechanical and welded splices

12.14.3.1 Mechanical and welded splices shall be permitted.

12.14.3.2 A full mechanical splice shall develop the specified tensile strength of the spliced bar in tension and 125% of the specified yield strength \( f_y \) of the spliced bar in compression.

12.14.3.3 Except as provided in this Code, all welding shall conform to “Structural Welding Code—Reinforcing Steel” (ANSI/AWS D1.4).

12.14.3.4 A full welded splice shall develop the specified tensile strength of the spliced bar in tension and compression.

12.14.3.5 Mechanical or welded splices not meeting requirements of 12.14.3.2 or 12.14.3.4 are not permitted.

12.14.3.6 All mechanical and welded splices 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 compliance with 12.14.3.2 and 12.14.3.4.

12.14.3.7 Mechanical splices shall be staggered if the strain measured over the full length of the splice (at 0.9 yield) exceeds that of a bar that is not mechanically spliced by more than 50%. If staggered mechanical splices are required, no more than 1/2 of the bars shall be spliced in one plane normal to the bars, and the mechanical splices shall be staggered at least 30 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: \( f_{yd} \) 

Class B splice: \( 1.3f_{yd} \)

where \( f_{yd} \) is calculated in accordance with 12.2 to develop \( f_{y} \) without the modification factor of 12.2.5.

12.15.2 Lap splices of deformed bars and deformed wire in tension shall be Class B splices except that Class A splices are allowed when:

(a) the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice; and

(b) one-half or less of the total reinforcement is spliced within the required lap length.

12.15.3 Mechanical or welded splices used where area of reinforcement provided is less than twice that required by analysis shall meet requirements of 12.14.3.2 or 12.14.3.4.

12.15.4 Mechanical or welded splices not meeting the requirements of 12.14.3.2 or 12.14.3.4 are not permitted.

12.15.5 Splices in tension tie members shall be made with a full mechanical or full welded splice in accordance with 12.14.3.2 or 12.14.3.4, and splices in adjacent bars shall be staggered at least 30 in.

12.15.6 Mechanical or welded splices shall be used for connecting tension-resisting, but not crack-controlling, reinforcing bars located in a region with membrane tension normal to the mechanical or welded splice. The average strength of these mechanical or welded splices 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 \( \frac{0.0005f_{yd}}{f_y}d_b \), for \( f_y \) of 60,000 psi or less, or \( \frac{0.0009f_{yd} - 24d_b}{f_y} \) for \( f_y \) greater than 60,000 psi, but not less than 12 in. For less than 3000 psi, \( f_y' \) length of lap shall be increased by 1/3.

12.16.2 When bars of different size are lap spliced in compression, splice length shall be the larger of \( l_{dc} \) of larger bar and 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 Mechanical or welded splices used in compression shall meet requirements of 12.14.3.2 or 12.14.3.4.

12.16.4 End-bearing splices

12.16.4.1 In bars required for compression only, transmission of compressive stress by bearing of square cut ends held in concentric contact by a suitable device shall be permitted.

12.16.4.2 Bar ends shall terminate in flat surfaces within 1.5 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, mechanical splices, butt-welded splices, and 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 1/2 of the bars are spliced at any section, or Class A tension lap splices if 1/2 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 Mechanical or welded splices in columns—Mechanical or welded splices in columns shall meet the requirements of 12.14.3.2 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 $f_y$, not less than $0.25f_y$, times the area of the vertical reinforcement in that face.

12.18—Splices of welded deformed wire reinforcement in tension
12.18.1 Minimum lap splice length of welded deformed wire reinforcement measured between the ends of each reinforcement sheet shall be not less than the larger of $1.3\ell_d$ and 8 in., and the overlap measured between outermost cross wires of each reinforcement sheet shall be not less than 2 in., where $\ell_d$ is calculated in accordance with 12.7 to develop $f_y$.

12.18.2 Lap splices of welded deformed wire reinforcement, 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 welded deformed wire reinforcement in the direction of the lap splice or when welded deformed wire reinforcement is lap spliced to welded plain wire reinforcement, the reinforcement shall be lap spliced in accordance with 12.19.

12.19—Splices of welded plain wire reinforcement in tension
Minimum length of lap for lap splices of welded plain wire reinforcement shall be in accordance with 12.19.1 and 12.19.2.
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, and shall be developed at face of support according to provisions of Chapter 12.

13.3.5 Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, anchorage of reinforcement shall be permitted within the slab.

13.3.6 In slabs with beams between supports with a value of \( \alpha \) greater than 1.0, special top and bottom slab reinforcement shall be provided at exterior corners in accordance with 13.3.6.1 through 13.3.6.4.

13.3.6.1 The special reinforcement in both top and bottom of slab shall be sufficient to resist a moment per foot of width equal to the maximum positive moment in the slab.

13.3.6.2 The moment shall be assumed to be about an axis perpendicular to the diagonal from the corner in the top of the slab and about an axis parallel to the diagonal from the 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 1/5 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 When a drop panel is used to reduce the amount of negative moment reinforcement over the column of a flat slab, the dimensions of the drop panel shall be in accordance with 13.2.5. In computing required slab reinforcement, the thickness of the drop panel below the slab shall not be assumed to be greater than 1/4 the distance from the edge of drop panel to the face of column or column capital.

13.3.8 Details of reinforcement in slabs without beams

13.3.8.1 In addition to the other requirements of 13.3, reinforcement in slabs without beams shall have minimum extensions as prescribed in Fig. 13.3.8.

13.3.8.2 Where adjacent spans are unequal, extensions of negative moment reinforcement beyond the face of support as prescribed in Fig. 13.3.8 shall be based on requirements of the longer span.

13.3.8.3 Bent bars shall be permitted only when depth-span ratio permits use of bends of 45 degrees or less.

13.3.8.4 In frames where two-way slabs act as primary members resisting lateral loads, lengths of reinforcement shall be determined by analysis but shall not be less than those prescribed in Fig. 13.3.8.

13.3.8.5 All bottom bars or wires within the column strip, in each direction, shall be continuous or spliced with Class A tension splices or with mechanical or welded splices satisfying 12.14.3. Splices shall be located as shown in Fig. 13.3.8. At least two of the column strip bottom bars or wires in each direction shall pass within the column core and shall be anchored at exterior supports.

13.3.8.6 In slabs with shearheads and in lift-slab construction where it is not practical to pass the bottom bars required by 13.3.8.5 through the column, at least two bonded bottom bars or wires in each direction shall pass through the shearhead or lifting collar as close to the column as practicable and be continuous or spliced with a Class A splice. At exterior columns, the reinforcement shall be anchored at the shearhead or lifting collar.

13.4—Openings in slab systems

13.4.1 Openings of any size shall be permitted in slab systems if shown by analysis that the design strength is at least equal to the required strength set forth in 9.2 and 9.3, and that all serviceability conditions, including the limits on deflections, are met.

13.4.2 As an alternate to special analysis as required by 13.4.1, openings shall be permitted in slab systems without beams only in accordance with 13.4.2.1 through 13.4.2.4.

13.4.2.1 Openings of any size shall be permitted in the area common to intersecting middle strips, provided total amount of reinforcement required for the panel without the opening is maintained.

13.4.2.2 In the area common to intersecting column strips, not more than 1/8 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 1/4 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 set forth in 9.2 and 9.3, and that all serviceability conditions, including limits on deflections, are met.

13.5.1.1 Design of a slab system for gravity loads, including the slab and beams (if any) between supports and supporting columns or walls forming orthogonal frames, by either the direct design method of 13.6 or the equivalent frame method of 13.7 shall be permitted.

13.5.1.2 For lateral loads, analysis of frames shall take into account effects of cracking and reinforcement on stiffness of frame members.

13.5.1.3 Combining the results of the gravity load analysis with the results of the lateral load analysis shall be permitted.
Fig. 13.3.8—Minimum extensions for reinforcement in slabs without beams. (Refer to 12.11.1 for reinforcement extension into supports.)
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 11.12.6.

13.5.3.1 The fraction of unbalanced moment not transferred by flexure shall be transferred by eccentricity of shear in accordance with 11.12.6.

13.5.3.2 A fraction of the unbalanced moment given by $\gamma_f M_u$ shall be considered to be transferred by flexure within an effective slab width between lines that are 1-1/2 slab or drop panel thicknesses ($1.5\ell h$) outside opposite faces of the column or capital, where $M_u$ is the factored moment to be transferred and

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

13.5.3.3 For unbalanced moments about an axis parallel to the edge at exterior supports, the value of $\gamma_f$ by Eq. (13-1) shall be permitted to be increased up to 1.0 provided that $V_u$ at an edge support does not exceed 0.75$\phi V_c$ or at a corner support does not exceed 0.5$\phi V_c$, where $V_c$ is calculated in accordance with 11.12.2.1. For unbalanced moments at interior supports, and for unbalanced moments about an axis transverse to the edge at exterior supports, the value of $\gamma_f$ 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$\phi V_c$. Reinforcement ratio $\rho$, within the effective slab width defined in 13.5.3.2, shall not exceed 0.375$\rho_0$. No adjustments to $\gamma_f$ 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 limitations of 13.6.1.1 through 13.6.1.8 by the direct design method shall be permitted.

13.6.1.1 There shall be a minimum of three continuous spans in each direction.

13.6.1.2 Panels shall be rectangular, with a ratio of longer to shorter span center-to-center of supports within a panel not greater than 2.

13.6.1.3 Successive span lengths center-to-center of supports in each direction shall not differ by more than 1/3 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, Eq. (13-2) shall be satisfied for beams in the two perpendicular directions

$$0.2 \leq \frac{a_1\ell_2^2}{a_2\ell_1^2} \leq 5.0 \quad (13-2)$$

where $a_1$ and $a_2$ are calculated in accordance with Eq. (13-3).

$$a_f = \frac{E_{cb}I_b}{E_cI_s} \quad (13-3)$$

13.6.1.7 Moment redistribution as permitted by 8.4 shall not be applied for slab systems designed by the direct design method. See 13.6.7.

13.6.1.8 Variations from the limitations of 13.6.1 shall be permitted if demonstrated by analysis that requirements of 13.5.1 are satisfied.

13.6.2 Total factored static moment for a span

13.6.2.1 Total factored static moment $M_o$ for a span shall be determined in a strip bounded laterally by centerline of panel on each side of centerline of supports.

13.6.2.2 Absolute sum of positive and average negative factored moments in each direction shall not be less than

$$M_o = \frac{q_n\ell_2\ell_n^2}{8} \quad (13-4)$$

where $\ell_n$ is length of clear span in direction that moments are being determined.

13.6.2.3 Where the transverse span of panels on either side of the centerline of supports varies, $\ell_n$ in Eq. (13-4) 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 $\ell_n$ in Eq. (13-4).

13.6.2.5 Clear span $\ell_n$ shall extend from face to face of columns, capitals, brackets, or walls. Value of $\ell_n$ used in Eq. (13-4) 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:

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<thead>
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<th>(2)</th>
<th>(3)</th>
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<td>Slabs without beams between interior supports</td>
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<td>0.70</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>Exterior edge fully restrained</td>
<td>0</td>
<td>0.16</td>
<td>0.26</td>
<td>0.30</td>
<td>0.65</td>
</tr>
</tbody>
</table>

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 stiffness of adjoining elements.

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:

$\frac{l_2}{l_1} = 0.5, 1.0, 2.0$

$(\alpha f l_2/l_1) = 0$

<table>
<thead>
<tr>
<th>$\frac{l_2}{l_1}$</th>
<th>0.5</th>
<th>1.0</th>
<th>2.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>75</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>1</td>
<td>90</td>
<td>75</td>
<td>45</td>
</tr>
</tbody>
</table>

13.6.4.2 Column strips shall be proportioned to resist the following portions in percent of exterior negative factored moments:

$\frac{l_2}{l_1} = 0.5, 1.0, 2.0$

$(\alpha f l_2/l_1) = 0$

<table>
<thead>
<tr>
<th>$\frac{l_2}{l_1}$</th>
<th>0.5</th>
<th>1.0</th>
<th>2.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta_t = 0$</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$\beta_t = 2.5$</td>
<td>75</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>$\beta_t = 0$</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$\beta_t = 2.5$</td>
<td>90</td>
<td>75</td>
<td>45</td>
</tr>
</tbody>
</table>

The constant $C$ for T- or L-sections shall be permitted to be evaluated by dividing the section into separate rectangular parts, as defined in 13.2.4, and summing the values of $C$ for each part.

13.6.4.3 Where supports consist of columns or walls extending for a distance equal to or greater than $3(3/4)l_2$ 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:

$\frac{l_2}{l_1} = 0.5, 1.0, 2.0$

$(\alpha f l_2/l_1) = 0$

<table>
<thead>
<tr>
<th>$\frac{l_2}{l_1}$</th>
<th>0.5</th>
<th>1.0</th>
<th>2.0</th>
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<tr>
<td>0</td>
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<td>60</td>
<td>60</td>
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<tr>
<td>$\geq 1.0$</td>
<td>90</td>
<td>75</td>
<td>45</td>
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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 $\alpha f l_2/l_1$ is equal to or greater than 1.0.

13.6.5.2 For values of $\alpha f l_2/l_1$ 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 a wall-supported edge shall be proportioned to resist twice the moment assigned to the half middle strip corresponding to the first row of interior supports.

13.6.7 Modification of factored moments—Modification of negative and positive factored moments by 10% shall be permitted provided the total static moment for a panel, $M_o$, in the direction considered is not less than that required by Eq. (13-4).

13.6.8 Factored shear in slab systems with beams

13.6.8.1 Beams with $\alpha f l_2/l_1$ equal to or greater than 1.0 shall be proportioned to resist shear caused by factored loads on tributary areas that are 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 beams with \( \alpha_f \ell_2 / \ell_1 \) less than 1.0 to resist shear, linear interpolation, assuming beams carry no load at \( \alpha_f = 0 \), 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 the requirements of Chapter 11.

13.6.9 Factored moments in columns and walls

13.6.9.1 Columns and walls built integrally with a slab system shall resist moments caused by factored loads on the slab system.

13.6.9.2 At an interior support, supporting elements above and below the slab shall resist the factored moment specified by Eq. (13-7) in direct proportion to their stiffnesses unless a general analysis is made.

\[
M_u = 0.07[q_{DU} + 0.5q_{Lu} \ell_2 \ell_1^2 - q_{DU} \ell_2 (\ell_1')^2]
\]  \hspace{1cm} (13-7)

where \( q_{DU} \), \( \ell_2 \), and \( \ell_1' \) 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 It shall be permitted to neglect the change in length of columns and slabs due to direct stress, and deflections due to shear.

13.7.2 Equivalent frame

13.7.2.1 The structure shall be considered to be made up of equivalent frames on column lines taken longitudinally and transversely through the building.

13.7.2.2 Each frame shall consist of a row of columns or supports and slab-beam strips, bounded laterally by the centerline of panel on each side of the centerline of columns or supports.

13.7.2.3 Columns or supports shall be assumed to be attached to slab-beam strips by torsional members (see 13.7.5) transverse to the direction of the span for which moments are being determined and extending to bounding lateral panel centerlines on each side of a column.

13.7.2.4 Frames adjacent and parallel to an edge shall be bounded by that edge and the centerline of adjacent panel.

13.7.2.5 Analysis of each equivalent frame in its entirety shall be permitted. Alternatively, for gravity loading, a separate analysis of each floor or roof with far ends of columns considered fixed shall be permitted.

13.7.2.6 Where slab-beams are analyzed separately, determination of moment at a given support assuming that the slab-beam is fixed at any support two panels distant therefrom, shall be permitted, provided the slab continues beyond that point.

13.7.3 Slab-beams

13.7.3.1 Determination of the moment of inertia of slab-beams at any cross section outside of joints or column capitals using the gross area of concrete shall be permitted.

13.7.3.2 Variation in moment of inertia along axis of slab-beams shall be taken into account.

13.7.3.3 Moment of inertia of slab-beams from center of column to face of column, bracket, or capital shall be assumed equal to the moment of inertia of the slab-beam at face of column, bracket, or capital divided by the quantity \( 1 - c_g \ell_2^2 \), where \( c_g \) and \( \ell_2 \) are measured transverse to the direction of the span for which moments are being determined.

13.7.4 Columns

13.7.4.1 Determination of the moment of inertia of columns at any cross section outside of joints or column capitals using the gross area of concrete shall be permitted.

13.7.4.2 Variation in moment of inertia along axis of columns shall be taken into account.

13.7.4.3 Moment of inertia of columns from top to bottom of the slab-beam at a joint shall be assumed to be infinite.

13.7.5 Torsional members

13.7.5.1 Torsional members (refer to 13.7.2.3) shall be assumed to have a constant cross section throughout their length consisting of the largest of (a), (b), and (c):

(a) A portion of slab having a width equal to that of the column, bracket, or capital in the direction of the span for which moments are being determined;

(b) For monolithic or fully composite construction, the portion of slab specified in (a) plus that part of the transverse beam above and below the slab;

(c) The transverse beam as defined in 13.2.4.

13.7.5.2 Where beams frame into columns in the direction of the span for which moments are being determined, the torsional stiffness shall be multiplied by the ratio of the moment of inertia of the slab with such a beam to the moment of inertia of the slab without such a beam.

13.7.6 Arrangement of live load

13.7.6.1 When the loading pattern is known, the equivalent frame shall be analyzed for that load.

13.7.6.2 When live load is variable but does not exceed 3/4 of the dead load, or the nature of live load is such that all panels will be loaded simultaneously, it shall be permitted to assume that maximum factored moments occur at all sections with full factored live load on entire slab system.

13.7.6.3 For loading conditions other than those defined in 13.7.6.2, it shall be permitted to assume that maximum positive factored moment near midspan of a panel occurs with 3/4 of the full factored live load on the panel and on alternate panels; and it shall be permitted to assume that maximum negative factored moment in the slab at a support occurs with 3/4 of the full live load on adjacent panels only.
13.7.6.4 Factored moments shall be taken not less than those occurring with full factored live load on all panels.

13.7.7 Factored moments

13.7.7.1 At interior supports, the critical section for negative factored moment (in both column and middle strips) shall be taken at face of rectilinear supports, but not farther away than 0.175t from the center of a column.

13.7.7.2 At exterior supports with brackets or capitals, the critical section for negative factored moment in the span perpendicular to an edge shall be taken at a distance from face of supporting element not greater than 1/2 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 Where 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. (13-4).

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

14.1—Scope

14.1.1 Provisions of Chapter 14 shall apply for design of walls subjected to axial load, with or without flexure.

14.1.2 Cantilever retaining walls are designed according to flexural design provisions of Chapter 10 with minimum horizontal reinforcement according to 14.3.3, but not 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, 14.5, or 14.8.

14.2.3 Design for shear shall be in accordance with 11.10.

14.2.4 Unless otherwise demonstrated by an analysis, the horizontal length of wall considered as effective for each concentrated load shall not exceed center-to-center distance between loads, nor the bearing width plus four times the wall thickness.

14.2.5 Compression members built integrally with walls shall conform to 10.8.2.

14.2.6 Walls shall be anchored to intersecting elements, such as floors and roofs; or to columns, pilasters, buttresses, and of intersecting walls; and to footings.

14.2.7 Quantity of reinforcement and limits of thickness required by 14.3 and 14.5 shall be permitted to be waived where structural analysis shows adequate strength and stability.

14.2.8 Transfer of force to footing at base of wall shall be in accordance with 15.8.

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, ρv, shall be:

(a) 0.0012 for deformed bars not larger than No. 5 with $f_y$ not less than 60,000 psi; or

(b) 0.0015 for other deformed bars; or

(c) 0.0012 for welded wire reinforcement not larger than W31 or D31.

14.3.3 Minimum ratio of horizontal reinforcement area to gross concrete area, ρh, shall be:

(a) 0.0020 for deformed bars not larger than No. 5 with $f_y$ not less than 60,000 psi; or

(b) 0.0025 for other deformed bars; or

(c) 0.0020 for welded wire reinforcement 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 1/2 and not more than 2/3 of total reinforcement required for each direction shall be placed not less than minimum cover depth required by 7.7, nor more than 1/3 the thickness of wall from the exterior surface;

(b) The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than minimum cover depth required by 7.7, nor more than 1/3 the thickness of wall from the interior surface.

14.3.5 Vertical and horizontal reinforcement shall not be spaced farther apart than three times the wall thickness, nor farther apart than 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 area of vertical section, 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, 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 the resultant of all factored loads is located within the middle third of the overall thickness of the wall and all limits of 14.2, 14.3, and 14.5 are satisfied.
14.5.2 Design axial strength $\phi P_n$ of a wall satisfying limitations of 14.5.1 shall be computed by Eq. (14-1) unless designed in accordance with 14.4.

$$\phi P_n = 0.55 \phi f_y' A_g \left[ 1 - \left( \frac{kt}{32h} \right)^2 \right] \quad (14-1)$$

where $\phi = 0.70$ and effective length factor $k$ shall be:

(a) Restrained against rotation at one or both ends (top, bottom, or both) .............................................. 0.8

(b) Unrestrained against rotation at both ends .............. 1.0

For walls not braced against lateral translation ................ 2.0

14.5.3 Minimum thickness of walls designed by empirical design method

14.5.3.1 Thickness of bearing walls shall not be less than 1/25 the supported height or length, whichever is shorter, nor less than 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 nonbearing 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.

14.8—Alternative design of slender walls

14.8.1 When flexural tension controls the design of a wall, the requirements of 14.8 are considered to satisfy 10.10.

14.8.2 Walls designed by the provisions of 14.8 shall satisfy 14.8.2.1 through 14.8.2.6.

14.8.2.1 The wall panel shall be designed as a simply supported, axially loaded member subjected to an out-of-plane uniform lateral load, with maximum moments and deflections occurring at midspan.

14.8.2.2 The cross section shall be constant over the height of the panel.

14.8.2.3 The wall shall be tension-controlled.

14.8.2.4 Reinforcement shall provide a design strength

$$\phi M_n \geq M_{cr} \quad (14-2)$$

where $M_{cr}$ shall be obtained using the modulus of rupture $f_{cr}$, given by Eq. (9-12).

14.8.2.5 Concentrated gravity loads applied to the wall above the design flexural section shall be assumed to be distributed over a width:

(a) Equal to the bearing width, plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design section; but

(b) Not greater than the spacing of the concentrated loads; and

(c) Not extending beyond the edges of the wall panel.

14.8.2.6 Vertical stress $P_u/A_g$ at the midheight section shall not exceed $0.06 f_y'$.

14.8.3 The design moment strength $\phi M_n$ for combined flexure and axial loads at the midheight cross section shall be

$$\phi M_n \geq M_u \quad (14-3)$$

where

$$M_u = M_{ua} + P_u \Delta_u \quad (14-4)$$

$M_{ua}$ is the moment at the midheight section of the wall due to factored lateral and eccentric vertical loads, and $\Delta_u$ is

$$\Delta_u = \frac{5M_{ua} l_c^2}{(0.75)48E_c I_{cr}} \quad (14-5)$$

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

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

where

$$I_{cr} = \frac{E_c}{E_l} \left( A_s + \frac{P}{f_y'} \right) (d - c)^2 + \frac{\ell c^3}{3} \quad (14-7)$$

and the value of $E_l/E_c$ shall not be taken less than 6.

14.8.4 $\Delta_s$, maximum deflection due to service loads, including $P\Delta$ effects, shall not exceed $\ell_c/150$. At midheight, $\Delta_s$ shall be calculated by

$$\Delta_s = \frac{(5M) \ell_c^2}{48E_c l_e} \quad (14-8)$$

$$M = \frac{M_{sa}}{1 - \frac{5P_s \ell_c^2}{48E_c \ell_e}} \quad (14-9)$$

$I_e$ shall be calculated using the procedure of 9.5.2.3, substituting $M$ for $M_u$, and $I_{cr}$ shall be calculated using Eq. (14-7).

CHAPTER 15—FOOTINGS

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 determined through principles of soil mechanics.

15.2.3 For footings on piles, computations for moments and shears shall be permitted to be based on the assumption that the reaction from any pile is concentrated at pile center.

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 $M_u$ 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;
(c) Halfway between face of column and edge of steel base plate, for footings supporting a column with steel base plate.

15.4.3 In one-way footings and two-way square footings, reinforcement shall be distributed uniformly across entire width of footing.

15.4.4 In two-way rectangular footings, reinforcement shall be distributed in accordance with 15.4.4.1 and 15.4.4.2.

15.4.4.1 Reinforcement in long direction shall be distributed uniformly across entire width of footing.

15.4.4.2 For reinforcement in short direction, a portion of the total reinforcement, $\gamma_s A_s$, shall be distributed uniformly over a band width (centered on centerline of column or pedestal) equal to the length of short side of footing. Remainder of reinforcement required in short direction $(1 – \gamma_s)A_s$ shall be distributed uniformly outside center band width of footing.

$$\gamma_s = \frac{2}{(\beta + 1)} \tag{15-1}$$

where $\beta$ is the ratio of long to short sides of footing.

15.5—Shear in footings

15.5.1 Shear strength of footings supported on soil or rock shall be in accordance with 11.12.

15.5.2 Location of critical section for shear in accordance with Chapter 11 shall be measured from face of column, pedestal, or wall, for footings supporting a column, pedestal, or wall. For footings supporting a column or pedestal with steel base plates, the critical section shall be measured from location defined in 15.4.2(c).

15.5.3 Where the distance between the axis of any pile and the axis of the column is more than two times the distance between the top of the pile cap and the top of the pile, the pile cap shall satisfy 11.12 and 15.5.4. Other pile caps shall satisfy either Appendix A, or both 11.12 and 15.5.4. If Appendix A is used, the effective concrete compression strength of the struts, $f_{ce}$, shall be determined using A.3.2.2(b).

15.5.4 Computation of shear on any section through a footing supported on piles shall be in accordance with 15.5.4.1, 15.5.4.2, and 15.5.4.3.

15.5.4.1 Entire reaction from any pile with its center located $d_{pile}/2$ or more outside the section shall be considered as producing shear on that section.

15.5.4.2 Reaction from any pile with its center located $d_{pile}/2$ or more inside the section shall be considered as producing no shear on that section.

15.5.4.3 For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the section shall be based on straight-line interpolation between full value at $d_{pile}/2$ outside the section and zero value at $d_{pile}/2$ inside the section.

15.6—Development of reinforcement in footings

15.6.1 Development of reinforcement in footings shall be in accordance with Chapter 12.

15.6.2 Calculated tension or compression in reinforcement at each section shall be developed on each side of that section by embedment length, hook (tension only) or mechanical device, or a combination thereof.

15.6.3 Critical sections for development of reinforcement shall be assumed at the same locations as defined in 15.4.2 for maximum factored moment, and at all other vertical planes where changes of section or reinforcement occur. See also 12.10.6.

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 stress on concrete at contact surface between supported and supporting member shall not exceed concrete bearing strength for either surface as given by 10.17.
15.8.1.2 Reinforcement, dowels, or mechanical connectors between supported and supporting members shall be adequate to transfer:
(a) All compressive force that exceeds concrete bearing strength of either member;
(b) Any computed tensile force across interface.

In addition, reinforcement, dowels, or mechanical connectors shall satisfy 15.8.2 or 15.8.3.

15.8.1.3 If calculated moments are transferred to supporting pedestal or footing, then reinforcement, dowels, or mechanical connectors shall be adequate to satisfy 12.17.

15.8.1.4 Lateral forces shall be transferred to supporting pedestal or footing in accordance with shear-friction provisions of 11.7, or by other appropriate means.

15.8.2 In cast-in-place construction, reinforcement required to satisfy 15.8.1 shall be provided either by extending longitudinal bars into supporting pedestal or footing, or by dowels.

15.8.2.1 For cast-in-place columns and pedestals, area of reinforcement across interface shall be not less than \(0.005A_g\), where \(A_g\) is the gross area of the supported member.

15.8.2.2 For cast-in-place walls, area of reinforcement across interface shall be not less than minimum vertical reinforcement given in 14.3.2.

15.8.2.3 At footings, it shall be permitted to lap splice No. 14 and No. 18 longitudinal bars, in compression only, 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, \(l_{de}\), 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. Anchor bolts shall be designed in accordance with Appendix D.

15.8.3.1 Connection between precast columns or pedestals and supporting members shall meet the requirements of 16.5.1.3(a).

15.8.3.2 Connection between precast walls and supporting members shall meet the requirements of 16.5.1.3(b) and (c).

15.8.3.3 Anchor bolts and mechanical connections shall be designed to reach their design strength before anchorage failure or failure of surrounding concrete. Anchor bolts shall be designed in accordance with Appendix D.

15.9—Sloped or stepped footings

15.9.1 In sloped or stepped footings, angle of slope or depth and location of steps shall be such that design requirements are satisfied at every section. (See also 12.10.6.)

15.9.2 Sloped or stepped footings designed as a unit shall be constructed to ensure action as a unit.

15.10—Combined footings and mats

15.10.1 Footings supporting more than one column, pedestal, or wall (combined footings or mats) shall be proportioned to resist the factored loads and induced reactions, in accordance with appropriate design requirements of the Code.

15.10.2 The direct design method of Chapter 13 shall not be used for design of combined footings and mats.

15.10.3 Distribution of soil pressure under combined footings and mats shall be consistent with properties of the soil and the structure and with established principles of soil mechanics.

CHAPTER 16—PRECAST CONCRETE

16.1—Scope

16.1.1 All provisions of this Code, not specifically excluded and not in conflict with the 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, (a) and (b) shall be included in either the contract documents or shop drawings:
(a) Details of reinforcement, inserts and lifting devices required to resist temporary loads from handling, storage, transportation, and erection;
(b) Required concrete strength at stated ages or stages of construction.

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, 16.3.2.1 and 16.3.2.2 shall apply.

16.3.2.1 In-plane force paths shall be continuous through both connections and members.

16.3.2.2 Where tension forces occur, a continuous path of steel or steel reinforcement shall be provided.

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 each shall be not less than \(0.001A_g\), where \(A_g\) is the gross cross-sectional area of the wall panel. Spacing of reinforcement shall not exceed five times the wall thickness nor 30 in. for interior walls nor 18 in. for exterior walls.

16.5—Structural integrity

16.5.1 Except where the provisions of 16.5.2 govern, the minimum provisions of 16.5.1.1 through 16.5.1.4 for structural integrity shall apply to all precast concrete structures.

16.5.1.1 Longitudinal and transverse ties required by 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 linear ft.

16.5.1.3 Vertical tension tie requirements of 7.13.3 shall apply to all vertical structural members, except cladding, and shall be achieved by providing connections at horizontal joints in accordance with (a) through (c):

(a) Precast columns shall have a nominal strength in tension not less than \(200A_g\) in lb. 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 \(1/2\) 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;

(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-ground.

16.5.1.4 Connection details that rely solely on friction caused by gravity loads shall not be used.

16.5.2 For precast concrete bearing wall structures three or more stories in height, the minimum provisions of 16.5.2.1 through 16.5.2.5 shall apply.

16.5.2.1 Longitudinal and transverse ties shall be provided in floor and roof systems to provide a nominal strength of 1500 lb per foot of width or length. Ties shall be provided over interior wall supports and between members being laterally supported shall have a nominal tensile strength capable of resisting not less than 3000 lb per horizontal foot of wall. Not less than two ties shall be provided for each precast panel.

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.

16.5.2.6 Transverse ties perpendicular to floor or roof shall be spaced not more than 10 ft on centers. Ties shall be provided over interior wall supports and between members being laterally supported shall have a nominal tensile strength capable of resisting 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 result of imposed loading, it shall be permitted to use the provisions of 11.7 as applicable.

16.6.1.2 When designing a connection using materials with different structural properties, their relative stiffnesses, strengths, and ductilities shall be considered.

16.6.2 Bearing for precast floor and roof members on simple supports shall satisfy 16.6.2.1 and 16.6.2.2.

16.6.2.1 The allowable bearing stress at the contact surface between supported and supporting members and between any intermediate bearing elements shall not exceed the bearing strength for either surface or the bearing element, or both. Concrete bearing strength shall be as given in 10.17.

16.6.2.2 Unless shown by test or analysis that performance will not be impaired, (a) and (b) shall be met:

(a) Each member and its supporting system shall have design dimensions selected so that, after consideration of tolerances, the distance from the edge of the support to the end of the precast member in the direction of the span is at least \(l_n/180\), 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 1/3 of such reinforcement shall extend to the center of the bearing length, taking into account permitted tolerances in 7.5.2.2 and 16.2.3.

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, 16.7.1.2, and 16.7.1.3 are met.

16.7.1.1 Embedded items are not required to be hooked or tied to reinforcement within the concrete.

16.7.1.2 Embedded items are maintained in the correct position while the concrete remains plastic.

16.7.1.3 The concrete is properly consolidated around the embedded item.

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 During erection, precast members and structures shall be adequately supported and braced to ensure proper alignment and structural integrity until permanent connections are 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 16.10.1.1 and 16.10.1.2.

16.10.1.1 Test loads shall be applied only when calculations indicate the isolated precast element will not be critical in compression or buckling.

16.10.1.2 The test load shall be that load that, 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

17.1—Scope
17.1.1 Provisions of Chapter 17 shall apply for design of composite concrete flexural members defined as precast concrete, cast-in-place concrete elements, or both, constructed in separate placements but so interconnected that all elements respond to loads as a unit.

17.1.2 All provisions of the Code shall apply to composite concrete flexural members, except as specifically modified in Chapter 17.

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, density, or other properties of the various elements are different, properties of the individual elements or the most critical values shall be used in design.

17.2.4 In strength computations of composite members, no distinction shall be made between shored and unshored members.

17.2.5 All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.

17.2.6 Reinforcement shall be provided as required to minimize cracking and to prevent separation of individual elements of composite members.

17.2.7 Composite members shall meet requirements for control of deflections in accordance with 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 Where 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 ensured at contact surfaces of interconnected elements.

17.5.2 For the provisions of 17.5, \( d \) shall be taken as the distance from extreme compression fiber for entire composite section to centroid of prestressed and nonprestressed longitudinal tension reinforcement, if any, but need not be taken less than \( 0.80b \) for prestressed concrete members.

17.5.3 Unless calculated in accordance with 17.5.4, design of cross sections subject to horizontal shear shall be based on

\[
V_u \leq \phi V_{nh}
\]  

(17-1)

where \( V_{nh} \) is nominal horizontal shear strength in accordance with 17.5.3.1 through 17.5.3.4.

17.5.3.1 Where contact surfaces are clean, free of laitance, and intentionally roughened, \( V_{nh} \) shall not be taken greater than \( 80b_v d \).

17.5.3.2 Where minimum ties are provided in accordance with 17.6, and contact surfaces are clean and free of laitance, but not intentionally roughened, \( V_{nh} \) shall not be taken greater than \( 80b_v d \).

17.5.3.3 Where 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., \( V_{nh} \) shall be taken equal to \( (260 + 0.6\phi\beta f_y b_v d) \), but not greater than \( 500 b_v d \).

17.5.3.4 Where \( V_u \) at section considered exceeds \( \phi(500b_v d) \), design for horizontal shear shall be in accordance with 11.7.4.

17.5.4 As an alternative to 17.5.3, horizontal shear shall be permitted to be determined by computing the actual change in compressive or tensile force in any segment, and provisions shall be made to transfer that force as horizontal shear to the supporting element. The factored horizontal shear force \( V_u \) shall not exceed horizontal shear strength \( \phi V_{nh} \) as given in 17.5.3.1 through 17.5.3.4, where area of contact surface shall be substituted for \( b_v d \).
17.5.4.1 Where ties provided to resist horizontal shear are designed to satisfy 17.5.4, the tie area to tie spacing ratio along the member shall approximately reflect the distribution of shear forces in the member.

17.5.5 Where 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 Where ties are provided to transfer horizontal shear, tie area shall not be less than that required by 11.5.6.3, and tie spacing shall not exceed four times the least dimension of the supported element, nor exceed 24 in.

17.6.2 Ties for horizontal shear shall consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire reinforcement.

17.6.3 All ties shall be fully anchored into interconnected elements in accordance with 12.13.

CHAPTER 18—PRESTRESSED CONCRETE
18.1—Scope
18.1.1 Provisions of Chapter 18 shall apply to members prestressed with wire, strands, or bars conforming to provisions for prestressing steel in 3.5.5.

18.1.2 All provisions of 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 6.4.4, 7.6.5, 8.10.2, 8.10.3, 8.10.4, 8.11, 10.5, 10.6, 10.9.1, and 10.9.2; Chapter 13; and Sections 14.3, 14.5, and 14.6, except that certain sections of 10.6 apply as noted in 18.4.4.

18.1.4 Sustained load conditions are defined to be the load combinations in Eq. (9-1), (9-2), (9-3), (9-4), and (9-5) of 9.2.1 with the load factors taken as unity.

18.2—General
18.2.1 Prestressed members shall meet the strength requirements of this Code.

18.2.2 Design of prestressed members shall be based on strength and on behavior at sustained conditions at all stages that will be critical during the life of the structure from the time prestress is first applied.

18.2.3 Stress concentrations due to prestressing shall be considered in design.

18.2.4 Provisions shall be made for effects on adjoining construction of elastic and plastic deformations, deflections, changes in length, and rotations due to prestressing. Effects of temperature and shrinkage shall also be included.

18.2.5 The possibility of buckling in a member between points where there is intermittent contact between the prestressing steel and an oversize duct, and buckling in thin webs and flanges shall be considered.

18.2.6 In computing section properties before bonding of prestressing steel, effect of loss of area due to open ducts shall be considered.

18.3—Design assumptions
18.3.1 Strength design of prestressed members for flexure and axial loads shall be based on assumptions given in 10.2, except that 10.2.4 shall apply only to reinforcement conforming to 3.5.3.

18.3.2 For investigation of stresses at transfer of prestress, at sustained loads, and at cracking loads, elastic theory shall be permitted to be used with the assumptions of 18.3.2.1 and 18.3.2.2.

18.3.2.1 Strains vary linearly with depth through the entire load range.

18.3.2.2 At cracked sections, concrete resists no tension.

18.3.3 Prestressed flexural members shall be classified as Class U, Class T, or Class C based on \( f_i \), the computed extreme fiber stress in tension in the precompressed tensile zone calculated at sustained loads, as follows:

(a) Class U: \( f_i \leq 7.5 \sqrt{f_{ci}} \)
(b) Class T: \( 7.5 \sqrt{f_{ci}} < f_i \leq 12 \sqrt{f_{ci}} \)
(c) Class C: \( f_i > 12 \sqrt{f_{ci}} \)

Prestressed two-way slab systems shall be designed as Class U with \( f_i \leq 6 \sqrt{f_{ci}} \).

18.3.4 For Class U and Class T flexural members, stresses at sustained loads shall be permitted to be calculated using the uncracked section. For Class C flexural members, stresses at sustained loads shall be calculated using the cracked transformed section.

18.3.5 Deflections of prestressed flexural members shall be calculated in accordance with 9.5.4.

18.4—Serviceability requirements—flexural members
18.4.1 Stresses in concrete immediately after prestress transfer (before time-dependent prestress losses) shall not exceed the following:

(a) Extreme fiber stress in compression: \( 0.60 \sqrt{f_{ci}} \)
(b) Extreme fiber stress in tension: 

\[ \text{except as permitted in (c): } 3 \sqrt{f_{ci}} \]
(c) Extreme fiber stress in tension at ends of simply supported members: \( 6 \sqrt{f_{ci}} \)

Where computed tensile stresses \( f_i \) exceed the limits in (b) or (c), additional bonded reinforcement (nonprestressed or prestressed) shall be provided in the tensile zone to resist the total tensile force in concrete computed with the assumption of an uncracked section.

18.4.2 For Class U and Class T prestressed flexural members, stresses in concrete at sustained loads (based on uncracked section properties, and after allowance for all prestress losses) shall not exceed the following: (a) Extreme fiber stress in compression:

\[ \text{Load Combinations (9-1), (9-2), (9-3), (9-4), (9-5): } 0.45 \sqrt{f_{ci}} \]

18.4.3 Permissible stresses in 18.4.1 and 18.4.2 shall be permitted to be exceeded if shown by test or analysis that performance will not be impaired.

18.4.4 For Class C prestressed flexural members not subject to fatigue or to aggressive exposure, the spacing of bonded reinforcement nearest the extreme tension face shall not exceed that given by 10.6.4.
For structures subject to fatigue or exposed to corrosive environments, special investigations and precautions are required.

18.4.1 The spacing requirements shall be met by nonprestressed reinforcement and bonded tendons. The spacing of bonded tendons shall not exceed 2/3 of the maximum spacing permitted for nonprestressed reinforcement.

Where both reinforcement and bonded tendons are used to meet the spacing requirement, the spacing between a bar and a tendon shall not exceed 5/6 of that permitted by 10.6.4. See also 18.4.4.3.

18.4.4 In applying Eq. (10-4) to prestressing tendons, \( \Delta f_{ps} \) shall be substituted for \( f_y \), where \( \Delta f_{ps} \) shall be taken as the calculated stress in the prestressing steel at sustained loads based on a cracked section analysis minus the decompression stress \( f_{dc} \). It shall be permitted to take \( f_{dc} \) equal to the effective stress in the prestressing steel \( f_{se} \). See also 18.4.4.3.

18.4.4.3 In applying Eq. (10-4) to prestressing tendons, the magnitude of \( \Delta f_{ps} \) shall not exceed 36,000 psi. When \( \Delta f_{ps} \) is less than or equal to 20,000 psi, the spacing requirements of 18.4.4.1 and 18.4.4.2 shall not apply.

18.4.4.4 Where \( h \) of a beam exceeds 36 in., the area of longitudinal skin reinforcement consisting of reinforcement or bonded tendons shall be provided as required by 10.6.7.

18.5—Permissible stresses in prestressing steel

18.5.1 Tensile stress in prestressing steel shall not exceed the following:

(a) Due to prestressing steel jacking force \( 0.94 f_{py} \) but not greater than the lesser of \( 0.80 f_{pu} \) and the maximum value recommended by the manufacturer of prestressing steel or anchorage devices.

(b) Immediately after prestress transfer \( 0.82 f_{py} \) but not greater than \( 0.74 f_{pu} \).

(c) Post-tensioning tendons, at anchorage devices and couplers, immediately after force transfer \( 0.70 f_{pu} \).

18.6—Loss of prestress

18.6.1 To determine effective stress in the prestressing steel, \( f_{se} \), allowance for the following sources of loss of prestress shall be considered:

(a) Prestressing steel seating at transfer;

(b) Elastic shortening of concrete;

(c) Creep of concrete;

(d) Shrinkage of concrete;

(e) Relaxation of prestressing steel stress;

(f) Friction loss due to intended or unintended curvature in post-tensioning tendons.

18.6.2 Friction loss in post-tensioning tendons

18.6.2.1 \( P_{px} \), force in post-tensioning tendons a distance \( l_{px} \) from the jacking end shall be computed by

\[
P_{px} = P_{py} e^{-(Kf_{px} + \mu f_{px})}
\]

Where \( (Kf_{px} + \mu f_{px}) \) is not greater than 0.3, \( P_{px} \) shall be permitted to be computed by

\[
P_{px} = P_{py} (1 + Kf_{px} + \mu f_{px})^{-1}
\]

18.6.2.2 Friction loss shall be based on experimentally determined wobble \( K \) and curvature \( \mu \) friction coefficients, and shall be verified during tendon stressing operations.

18.6.2.3 Values of \( K \) and \( \mu \) used in design shall be shown on design drawings.

18.6.3 Where loss of prestress in a member occurs due to connection of the member to adjoining construction, such loss of prestress shall be allowed for in design.

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 steel, \( f_{ps} \) shall be substituted for \( f_y \) in strength computations.

18.7.2 As an alternative to a more accurate determination of \( f_{ps} \), based on strain compatibility, the following approximate values of \( f_{ps} \) shall be permitted to be used if \( f_{se} \) is not less than \( 0.5 f_{pu} \):

(a) For members with bonded tendons

\[
f_{ps} = f_{pu} \left[ 1 - \frac{\gamma e}{p_1} \left( \rho f_{pu} \frac{d}{d_p} + \frac{d}{d_p} (\omega - \omega') \right) \right] \]

(18-3)

where \( \omega \) is \( \rho f_{py} / f_{pu} \), \( \omega' \) is \( \rho f_{py} / f_{pu} \), and \( \gamma e \) is 0.55 for \( f_{py} / f_{pu} \) not less than 0.80; 0.40 for \( f_{py} f_{pu} \) not less than 0.85; and 0.28 for \( f_{py} f_{pu} \) not less than 0.90.

If any compression reinforcement is taken into account when calculating \( f_{ps} \) in Eq. (18-3), the term

\[
\left[ \rho f_{pu} \frac{d}{d_p} + \frac{d}{d_p} (\omega - \omega') \right]
\]

shall be taken not less than 0.17 and \( d' \) shall be no greater than \( 0.15 d_p \).

(b) For members with unbonded tendons and with a span-to-depth ratio of 35 or less

\[
f_{ps} = f_{se} + 10,000 + \frac{f_{se}'}{100 \rho_p} \]

(18-4)

but \( f_{ps} \) in Eq. (18-4) shall not be taken greater than the lesser of \( f_{py} \) and \( (f_{se} + 60,000) \).

(c) For members with unbonded tendons and with a span-to-depth ratio greater than 35

\[
f_{ps} = f_{se} + 10,000 + \frac{f_{se}'}{300 \rho_p} \]

(18-5)

but \( f_{ps} \) in Eq. (18-5) shall not be taken greater than the lesser of \( f_{py} \) and \( (f_{se} + 30,000) \).

18.7.3 Nonprestressed reinforcement conforming to 3.5.3, if used with prestressing steel, shall be permitted to be considered to contribute to the tensile force and to be included in moment strength computations at a stress equal to \( f_y \). Other nonprestressed reinforcement shall be permitted.
18.8—Limits for reinforcement of flexural members

18.8.1 Prestressed concrete sections shall be classified as either tension-controlled, transition, or compression-controlled sections, in accordance with 10.3.3 and 10.3.4. The appropriate strength-reduction factors $\phi$ from 9.3.2 shall apply.

18.8.2 Total amount of prestressed and nonprestressed reinforcement shall be adequate to develop a factored load at least 1.2 times the cracking load computed on the basis of the modulus of rupture $f_c$ specified in 9.5.2.3. This provision shall be permitted to be waived for:
(a) Two-way, unbonded post-tensioned slabs; and
(b) Flexural members with shear and flexural strength at least twice that required by 9.2.

18.8.3 Part or all of the bonded reinforcement consisting of bars or tendons shall be provided as close as practicable to the tension face in prestressed flexural members. In members prestressed with unbonded tendons, the minimum bonded reinforcement consisting of bars or tendons shall be as required by 18.9.

18.9—Minimum bonded reinforcement

18.9.1 A minimum area of bonded reinforcement shall be provided in all flexural members with unbonded tendons as required by 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

$$A_s = 0.004 A_{cf}$$

(18-6)

where $A_{cf}$ is area of that part of cross section between the flexural tension face and center of gravity of gross section.

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 sustained load stress conditions.

18.9.3 For two-way flat slab systems, minimum area and distribution of bonded reinforcement shall be as required in 18.9.3.1, 18.9.3.2, and 18.9.3.3.

18.9.3.1 Bonded reinforcement shall not be required in positive moment areas where $f_t$, the extreme fiber stress in tension in the precompressed tensile zone at sustained loads, (after allowance for all prestress losses) does not exceed $2.5 f_t^c$.

18.9.3.2 In positive moment areas where computed tensile stress in concrete at sustained load exceeds $2.5 f_t^c$, minimum area of bonded reinforcement shall be computed by

$$A_s = \frac{N_s}{0.5 f_{ty}}$$

(18-7)

where the value of $f_{ty}$ used in Eq. (18-7) shall not exceed 60,000 psi. Bonded reinforcement shall be uniformly distributed over precompressed tensile zone as close as practicable to the extreme tension fiber.

18.9.3.3 In negative moment areas at column supports, the minimum area of bonded reinforcement $A_s$ in the top of the slab in each direction shall be computed by

$$A_s = 0.0075 A_{cf}$$

(18-8)

where $A_{cf}$ is the larger gross cross-sectional area of the slab.

Bonded reinforcement required by Eq. (18-8) shall be distributed between lines that are 1.5$h$ outside opposite faces of the column support. At least four bars or wires shall be provided in each direction. Spacing of bonded reinforcement shall not exceed 12 in.

18.9.4 Minimum length of bonded reinforcement required by 18.9.2 and 18.9.3 shall be as required in 18.9.4.1, 18.9.4.2, and 18.9.4.3.

18.9.4.1 In positive moment areas, minimum length of bonded reinforcement shall be 1/3 the clear span length $l_n$ and centered in positive moment area.

18.9.4.2 In negative moment areas, bonded reinforcement shall extend 1/6 the clear span $l_n$ on each side of support.

18.9.4.3 Where bonded reinforcement is provided for $\phi M_n$, 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 sustained load conditions and for adequate strength.

18.10.2 Performance at sustained load conditions shall be determined by elastic analysis, considering reactions, moments, shears, and axial forces induced by prestressing, creep, shrinkage, temperature change, axial deformation, restraint of attached structural elements, and foundation settlement.

18.10.3 Moments used to compute required strength shall be the sum of the moments due to reactions induced by prestressing (with a load factor of 1.0) and the moments due to factored loads. Adjustment of the sum of these moments shall be permitted as allowed in 18.10.4.

18.10.4 Redistribution of negative moments in continuous prestressed flexural members

18.10.4.1 Where bonded reinforcement is provided at supports in accordance with 18.9, it shall be permitted to increase or decrease negative moments calculated by elastic theory for any assumed loading, in accordance with 8.4.

18.10.4.2 The modified negative moments shall be used for calculating moments at sections within spans for the same loading arrangement.

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. 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 compressive stress in concrete due to effective prestress force only 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 compressive stress in concrete due to effective prestress force only equal to or greater than 225 psi shall have all tendons enclosed by spirals or lateral ties in accordance with (a) through (d):
(a) Spirals shall conform to 7.10.4;
(b) Lateral ties shall be at least No. 3 in size or welded wire reinforcement of equivalent area, and shall be spaced vertically not to exceed 48 tie bar or wire diameters, or the least dimension of the compression member;
(c) Ties shall be located vertically not more than half a tie spacing above top of footing or slab in any story, and not more than half a tie spacing below the lowest horizontal reinforcement in members supported above;
(d) Where beams or brackets frame into all sides of a column, ties shall be terminated not more than 3 in. below lowest reinforcement in such beams or brackets.

18.11.2.3 For walls with average compressive stress in concrete due to effective prestress force only 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 \( \phi M_n \) of prestressed slabs required by 9.3 at every section shall be greater than or equal to \( M_n \) considering 9.2, 18.10.3, and 18.10.4. \( \phi V_n \) of prestressed slabs at columns required by 9.3 shall be greater than or equal to \( V_u \) considering 9.2, 11.1, 11.12.2, and 11.12.6.2.

18.12.3 At sustained load conditions, all serviceability limitations, including limits on deflections, shall be met, with appropriate consideration of the factors listed in 18.10.2.

18.12.4 For uniformly distributed loads, spacing of tendons or groups of tendons in one direction shall not exceed the smaller of eight times the slab thickness and 5 ft. Spacing of tendons also shall provide a minimum average effective prestress 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 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—Post-tensioned tendon anchorage zones

18.13.1 Anchorage zone—The anchorage zone shall be considered as composed of two zones:
(a) The local zone is the rectangular prism (or equivalent rectangular prism for circular or oval anchorages) of concrete immediately surrounding the anchorage device and any confining reinforcement;
(b) The general zone is the anchorage zone as defined in 2.2 and includes the local zone.

18.13.2 Local zone

18.13.2.1 Design of local zones shall be based upon the factored prestressing force \( P_{pu} \) and the requirements of 9.2.5 and 9.3.2.5.

18.13.2.2 Local zone reinforcement shall be provided where required for proper functioning of the anchorage device.

18.13.3 General zone

18.13.3.1 Design of general zones shall be based on the factored prestressing force \( P_{pu} \) and the requirements of 9.2.5 and 9.3.2.5.

18.13.3.2 General zone reinforcement shall be provided where required to resist bursting, spalling, and longitudinal edge tension forces induced by anchorage devices. Effects of abrupt change in section shall be considered.

18.13.3.3 The general zone requirements of 18.13.3.2 are satisfied by 18.13.4, 18.13.5, and 18.13.6.

18.13.4 Nominal material strengths

18.13.4.1 Nominal tensile stress of bonded reinforcement is limited to \( f_{py} \) for nonprestressed reinforcement and to \( f_{py} \) for prestressed reinforcement. Nominal tensile stress of unbonded prestressed reinforcement for resisting tensile forces in the anchorage zone shall be limited to \( f_{cu} = f_{cu} + 10,000 \).

18.13.4.2 Except for concrete confined within spirals or hoops providing confinement equivalent to that corresponding to Eq. (10-5), nominal compressive strength of concrete in the general zone shall be limited to \( 0.7 f_{ci} \).

18.13.4.3 Compressive strength of concrete at time of post-tensioning shall be specified on the design drawings. Unless oversized anchorage devices sized to compensate for the lower compressive strength are used or the prestressing steel is stressed to no more than 50% of the final prestressing force, prestressing steel shall not be stressed until compressive strength of concrete as indicated by tests consistent with the curing of the member, is at least 4000 psi for multistrand tendons or at least 2500 psi for single-strand or bar tendons.

18.13.5 Design methods

18.13.5.1 The following methods shall be permitted for the design of general zones provided that the specific procedures used result in prediction of strength in substantial agreement with results of comprehensive tests:
(a) Equilibrium-based plasticity models (strut-and-tie models);
(b) Linear stress analysis (including finite element analysis or equivalent); or
(c) Simplified equations where applicable.

18.13.5.2 Simplified equations shall not be used where member cross sections are nonrectangular, where discontinuities in or near the general zone cause deviations in the force flow path, where minimum edge distance is less than 1-1/2 times the anchorage device lateral dimension in that
direction, or where multiple anchorage devices are used in other than one closely spaced group.

18.13.5.3 The stressing sequence shall be specified on the design drawings and considered in the design.

18.13.5.4 Three-dimensional effects shall be considered in design and analyzed using three-dimensional procedures or approximated by considering the summation of effects for two orthogonal planes.

18.13.5.5 For anchorage devices located away from the end of the member, bonded reinforcement shall be provided to transfer at least \(0.35P_{pu}\) into the concrete section behind the anchor. Such reinforcement shall be placed symmetrically around the anchorage devices and shall be fully developed both behind and ahead of the anchorage devices.

18.13.5.6 Where tendons are curved in the general zone, except for monostrand tendons in slabs or where analysis shows reinforcement is not required, bonded reinforcement shall be provided to resist radial and splitting forces.

18.13.5.7 Except for monostrand tendons in slabs or where analysis shows reinforcement is not required, minimum reinforcement with a nominal tensile strength equal to 2% of each prestressing force shall be provided in orthogonal directions parallel to the back face of all anchorage zones to limit spalling.

18.13.5.8 Tensile strength of concrete shall be neglected in calculations of reinforcement requirements.

18.13.6 Detailing requirements —Selection of reinforcement sizes, spacings, cover, and other details for anchorage zones shall make allowances for tolerances on the bending, fabrication, and placement of reinforcement, for the size of aggregate, and for adequate placement and consolidation of the concrete.

18.14—Intentionally left blank

18.15—Intentionally left blank

18.16—Corrosion protection for unbonded tendons

18.16.1 Unbonded prestressing steel shall be encased with sheathing. The prestressing steel shall be completely coated and the sheathing around the prestressing steel filled with suitable material to inhibit corrosion.

18.16.2 Sheathing shall be watertight and continuous over entire length to be unbonded.

18.16.3 For applications in corrosive environments, the sheathing shall be connected to all stressing, intermediate and fixed anchorages in a watertight fashion.

18.16.4 Unbonded single-strand tendons shall be protected against corrosion in accordance with ACI’s “Specification for Unbonded Single-Strand Tendons (ACI 423.6).”

18.17—Post-tensioning ducts

18.17.1 Ducts for grouted tendons shall be mortar-tight and nonreactive with concrete, prestressing steel, grout, and corrosion inhibitor.

18.17.2 Ducts for grouted single-wire, single-strand, or single-bar tendons shall have an inside diameter at least 1/4 in. larger than the prestressing steel diameter.

18.17.3 Ducts for grouted multiple-wire, multiple-strand, or multiple-bar tendons shall have an inside cross-sectional area at least two times the cross-sectional area of the prestressing steel.

18.17.4 Ducts shall be maintained free of ponded water if members to be grouted are exposed to temperatures below freezing prior to grouting.

18.18—Grout for bonded tendons

18.18.1 Grout shall consist of portland cement and water; or portland cement, sand, and water.

18.18.2 Materials for grout shall conform to 18.18.2.1 through 18.18.2.4.

18.18.2.1 Portland cement shall conform to 3.2.

18.18.2.2 Water shall conform to 3.4.

18.18.2.3 Sand, if used, shall conform to “Standard Specification for Aggregate for Masonry Mortar” (ASTM C 144) except that gradation shall be permitted to be modified as necessary to obtain satisfactory workability.

18.18.2.4 Admixtures conforming to 3.6 and known to have no injurious effects on grout, steel, or concrete shall be permitted. Calcium chloride shall not be used.

18.18.3 Selection of grout proportions

18.18.3.1 Proportions of materials for grout shall be based on either (a) or (b):

(a) Results of tests on fresh and hardened grout prior to beginning grouting operations; or

(b) Prior documented experience with similar materials and equipment and under comparable field conditions.

18.18.3.2 Cement used in the grout shall correspond to that on which selection of grout proportions was based.

18.18.3.3 Water content shall be minimum necessary for proper pumping of grout; however, water-cement ratio shall not exceed 0.45 by weight.

18.18.3.4 Water shall not be added to increase grout flowability that has been decreased by delayed use of the grout.

18.18.4 Mixing and pumping grout

18.18.4.1 Grout shall be mixed in equipment capable of continuous mechanical mixing and agitation that will produce uniform distribution of materials, passed through screens, and pumped in a manner that will completely fill the ducts.

18.18.4.2 Temperature of members at time of grouting shall be above 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.

18.18.4.3 Grout temperatures shall not be above 90 °F during mixing and pumping.

18.19—Protection for prestressing steel

Burning or welding operations in the vicinity of prestressing steel shall be performed so that prestressing steel is not subject to excessive temperatures, welding sparks, or ground currents.

18.20—Application and measurement of prestressing force

18.20.1 Prestressing force shall be determined by both of (a) and (b):

(a) Measurement of steel elongation. Required elongation shall be determined from average load-elongation curves for the prestressing steel used;
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 Shell instability shall be investigated and shown by design to be precluded. Nonlinear variations in circumferential and meridional stresses across the shell thickness shall be considered.

19.2.9 Supporting members

19.2.9.1 Supporting members shall be designed according to the 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 the 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 at all locations of discontinuities in geometric and material properties that 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 the 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 internal 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 the junction of the shell and supporting members or edge members shall be anchored in or through supporting members by embedment length, hooks, or other 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.

CHAPTER 20—STRENGTH EVALUATION OF EXISTING STRUCTURES
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 5.6.5.4 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 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 reinforcement, or tendons shall be determined by measurement. It shall be permitted to base reinforcement locations on available drawings if spot checks are made confirming the information on the drawings.
20.2.3 If required, concrete strength shall be based on results of cylinder tests or tests of cores removed from the part of the structure where the strength is in doubt. Concrete strengths shall be determined as specified in 5.6.5.
20.2.4 If required, reinforcement or prestressing steel strength shall be based on tensile tests of representative samples of the material in the structure in question.
20.2.5 If the required dimensions and material properties are determined through measurements and testing, and if calculations can be made in accordance with 20.1.2 and subject to the special requirements of 1.4, it shall be permitted to increase $\phi$ from those specified in 9.3. The engineer shall justify such an increase based on the load intensity used for the test compared to the governing load combination for structure design.

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 used to determine strength characteristics of specific elements such as anchorages and embeddings. 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 members 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 \(0.85(1.4D + 1.7L)\); however, the load intensity selected by the engineer for the test must be based on governing design requirements and load combinations for the structure in question.

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 1 hour before application of the first load increment. Measurements shall be made at locations where maximum response is expected. Additional measurements shall be made if required.

20.5.2 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 deflections shall satisfy one of the following conditions

\[
\Delta_1 \leq \frac{\xi^2}{20,000h} \tag{20-1}
\]

\[
\Delta_r \leq \frac{\Delta_1}{4} \tag{20-2}
\]

If the measured maximum and residual deflections, \(\Delta_1\) and \(\Delta_r\), 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 \(\Delta_r\) satisfies the condition

\[
\Delta_r \leq \frac{\Delta_2}{5} \tag{20-3}
\]

where \(\Delta_2\) 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 or Technical Safety Requirements.

20.7.2 No safety measures shall interfere with load test procedures or affect results.

CHAPTER 21—PROVISIONS FOR SEISMIC DESIGN

21.1—Definitions

base of structure—level at which earthquake motions are assumed to be imparted to a building. This level does not necessarily coincide with the ground level.

boundary elements—portions along structural wall and structural diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms shall be provided with boundary elements if required by 21.7.6 or 21.9.8.

collector elements—elements that serve to transmit the inertial forces within structural 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 in 9.2 (or Appendix C).

design story drift ratio—relative difference of design displacement between the top and bottom of a story, divided by the story height.

development length for a bar with a standard hook—the shortest distance from the critical section (where the strength of the bar is to be developed) to the outside end of the 90-degree hook.

factored loads and forces—loads and forces multiplied by appropriate load factors in 9.2 or in C.2.

hoop—a closed tie or continuously wound tie. A closed tie can be made up of several reinforcement elements each
having seismic hooks at both ends. A continuously wound tie shall have a seismic hook at both ends.

**joint**—portion of structure common to intersecting members. The effective cross-sectional area of the joint, \( A_{ij} \), for shear strength computations is defined in 21.5.3.1.

**lateral-force resisting system**—that portion of the structure composed of members proportioned and detailed to resist the design seismic forces.

**moment frame**—a frame in which members and joints resist forces through flexure, shear, and axial force. A moment frame is a cast-in-place frame complying with the requirements of 21.2 through 21.5. In addition, the requirements of Chapters 1 through 18 shall be satisfied.

**plastic hinge region**—length of frame element over which flexural yielding is intended to occur due to design displacements, extending not less than a distance \( h \) from the critical section where flexural yielding initiates.

**seismic hook**—a hook on a stirrup, hoop, or crosstie having a bend not less than 135 degrees, except that circular hoops shall have a bend not less than 90 degrees. Hoops shall have 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.

**specified lateral forces**—lateral forces corresponding to the appropriate distribution of the earthquake design base shear force.

**structural diaphragms**—structural members, such as floor and roof slabs, that transmit inertial forces to lateral-force-resisting members.

**structural trusses**—assemblages of reinforced concrete members subjected primarily to axial forces.

**structural walls**—walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shearwall is a cast-in-place structural wall complying with the requirements of 21.2 and 21.7 in addition to the requirements of Chapters 1 through 18.

**strut**—an element of a structural diaphragm used to provide continuity around an opening in the diaphragm.

**tie elements**—elements that serve to transmit inertia forces and prevent separation of building components such as footings and walls.

### 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 the Code.

21.2.1.2 The provisions of Chapters 1 through 18 shall apply except as modified by the provisions of this chapter.

21.2.2 **Analysis and proportioning of structural members**—All structural and nonstructural members, including foundation members that materially affect the response of the structure to earthquake motions, shall be considered explicitly in the design and analysis of the structure.

21.2.3 **Strength reduction factors**—Strength-reduction factors shall be as given in 9.3 (or in Appendix C).

21.2.4 **Concrete in members resisting earthquake-induced forces**—Specified compressive strength of concrete, \( f'_{c} \), 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 moment frames and structural walls shall comply with ASTM A 706. ASTM A 615 Grades 40 and 60 reinforcement shall be permitted in these members if:

(a) The actual yield strength based on mill tests does not exceed \( f_y \) 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 tensile strength to the actual yield strength is not less than 1.25.

The value of \( f_y \) for transverse reinforcement including spiral reinforcement shall not exceed 60,000 psi.

21.2.6 **Mechanical splices**—Mechanical splices shall conform to 12.14.3 and shall develop the specified tensile strength of the spliced bar.

21.2.7 **Welded splices**

21.2.7.1 Welded splices in reinforcement resisting earthquake-induced forces shall conform to 12.14.3.

21.2.7.2 Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement that is required by design shall not be permitted.

21.2.8 **Anchoring to concrete**—Anchors resisting earthquake-induced forces shall conform to Appendix D.

### 21.3—Flexural members of moment frames

21.3.1 **Scope**—Requirements of 21.3 apply to moment frame members (a) resisting earthquake-induced forces and (b) proportioned primarily to resist flexure. These frame members shall also satisfy the conditions of 21.3.1.1 through 21.3.1.4.

21.3.1.1 **Factored axial compressive force on the member**— \( P_u \), shall not exceed \( A_{nf,1} f_y / 10 \).

21.3.1.2 **Clear span for member**—\( t_u \), shall not be less than four times its effective depth.

21.3.1.3 **Width of member**—\( b_w \), shall not be less than the smaller of 0.3\( h \) and 10 in.

21.3.1.4 **Width of member**—\( b_w \), shall not exceed width of supporting member (measured on a plane perpendicular to the longitudinal axis of flexural member) plus distances on each side of supporting member not exceeding 3/4 of the depth of 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 and bottom reinforcement, the amount of reinforcement shall not be less than that given by Eq. (10-3) but not less than \( 200 d f_y / d f'_{c} \), and the reinforcement ratio \( \rho \) 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 the joint face** shall be not less than 1/2 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 1/4 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. Spacing of the transverse reinforcement enclosing the lapped bars shall not exceed the smaller of $d/4$ and 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) where analysis indicates flexural yielding is caused by inelastic lateral displacements of the frame.

21.3.2.4 Mechanical splices shall conform to 21.2.6 and welded splices shall conform to 21.2.7.

21.3.3 Transverse reinforcement

21.3.3.1 Hoops shall be provided in the following regions of frame members:
(a) Over a length equal to twice the member depth measured from the face of the supporting member toward midspan, at both ends of the flexural member;
(b) Over lengths equal to twice the member depth on both sides of a section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame.

21.3.3.2 The first hoop shall be located not more than 2 in. from the face of a supporting member. Spacing of the hoops shall not exceed the smallest of (a), (b), (c) and (d):
(a) $d/4$;
(b) eight times the diameter of the smallest longitudinal bars;
(c) 24 times the diameter of the hoop bars; and
(d) 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 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 flexural moment strength $M_{p_{\text{u}}}$ 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 $V_e = 0$ when both (a) and (b) occur:
(a) The earthquake-induced shear force calculated in accordance with 21.3.4.1 represents 1/2 or more of the maximum required shear strength within those lengths;
(b) The factored axial compressive force $P_u$, including earthquake effects, is less than $A_g f_{c^\prime}/20$.

21.4—Moment frame members subjected to bending and axial load

21.4.1 Scope—The requirements of this section apply to moment frame members (a) resisting earthquake-induced forces and (b) having a factored axial compressive force $P_u$ exceeding $A_g f_{c^\prime}/10$. These frame members shall also satisfy 21.4.1.1 and 21.4.1.2.

21.4.1.1 The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 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 $P_u$ exceeding $A_g f_{c^\prime}/10$ shall satisfy 21.4.2.2.

21.4.2.2 The flexural strengths of the columns shall satisfy Eq. (21-1)

$$\Sigma M_{nc} \geq (7/5)\Sigma M_{nb}$$

(21-1)

where $\Sigma M_{nc} = \text{sum of nominal flexural strengths of columns framing into the joint, evaluated at the faces of the joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength; and}$

$\Sigma M_{nb} = \text{sum of nominal flexural strengths of the beams framing into the joint, evaluated at the faces of the joint. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width defined in 8.10 shall be assumed to contribute to } M_{nb} \text{ if the slab reinforcement is developed at the critical section for flexure.}$

Flexural strengths shall be summed such that the column moments oppose the beam moments. Equation (21-1) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

21.4.3 Longitudinal reinforcement

21.4.3.1 Area of longitudinal reinforcement, $A_{st}$, shall not be less than $0.014 A_g$ or more than $0.064 A_g$.

21.4.3.2 Mechanical splices shall conform to 21.2.6 and welded splices shall conform to 21.2.7. Lap splices shall be permitted only within the center half of the member length, shall be designed as tension lap splices, and shall be enclosed within transverse reinforcement conforming to 21.4.4.2 and 21.4.4.3.

21.4.4 Transverse reinforcement

21.4.4.1 Transverse reinforcement required in (a) through (e) shall be provided unless a larger amount is required by 21.4.3.2 or 21.4.5.

(a) The volumetric ratio of spiral or circular hoop reinforcement, $\rho_s$, shall not be less than required by Eq. (21-2).
\[ \rho_s = 0.12 f'_c / f_y \]  

(21-2)

and shall not be less than required by Eq. (10-5).

(b) The total cross-sectional area of rectangular hoop reinforcement, \( A_{sh} \), shall not be less than required by Eq. (21-3) and (21-4).

\[ A_{sh} = 0.3 (s b_c f'_c / f_y) [(A_d / A_{ch}) - 1] \]  

(21-3)

\[ A_{sh} = 0.09 s b_c f'_c / f_y \]  

(21-4)

(c) Transverse reinforcement shall be provided by either single or overlapping hoops. Crossties of the same bar size and spacing as the hoops shall be permitted. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement.

(d) If the design strength of the core satisfies the requirement of the design loading combinations including \( E \), Eq. (21-3) and (10-5) need not be satisfied.

(e) If the thickness of the concrete outside the confining transverse reinforcement exceeds 4 in., additional transverse reinforcement shall be provided with a spacing not exceeding 6 in. Concrete cover to the additional transverse reinforcement shall not exceed 4 in.

21.4.4.2 Spacing of transverse reinforcement shall not exceed the smallest of (a), (b), and (c):

(a) one-quarter of the minimum member dimension;

(b) six times the diameter of the longitudinal reinforcement; and

(c) \( s_o \), as defined by Eq. (21-5)

\[ s_o = 4 + \left( \frac{14 - h_e}{3} \right) \]  

(21-5)

The value of \( s_o \) shall not exceed 6 in. and need not be taken less than 4 in.

21.4.4.3 Horizontal spacing of crossties or legs of overlapping hoops, \( h_e \), shall not exceed 14 in. on center.

21.4.4.4 Transverse reinforcement as specified in 21.4.4.1 through 21.4.4.3 shall be provided over a length \( l_o \) from each joint face and on both sides of any section where flexural yielding is likely to occur as a result of inelastic lateral displacements of the frame. Length \( l_o \) shall not be less than the largest of (a), (b), and (c):

(a) the depth of the member at the joint face or at the section where flexural yielding is likely to occur;

(b) one-sixth of the clear span of the member; and

(c) 18 in.

21.4.4.5 Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 over their full height beneath the level at which the discontinuity occurs if the factored axial compressive force in these members, related to earthquake effect, exceeds \( A_g f'_c / 10 \). Transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 shall extend at least the development length in tension, \( l_t \), into the discontinued member, where \( l_t \) is determined in accordance of 21.5.4 using the largest longitudinal reinforcement in the column. If the lower end of the column terminates on a wall, transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 shall extend into the wall at least \( l_t \) of the largest longitudinal column bar at the point of termination. If the column terminates on a footing or mat, transverse reinforcement as required in 21.4.4.1 through 21.4.4.3 shall extend at least 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, \( s \), not exceeding the smaller of six times the diameter of the longitudinal column bars and 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 \( M_{pr} \) at each end of the member associated with the range of factored axial loads, \( P_{au} \), acting on the member. The member shears need not exceed those determined from joint strengths based on \( M_{pr} \) 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 \( V_e = 0 \) when both (a) and (b) occur:

(a) The earthquake-induced shear force, calculated in accordance with 21.4.5.1, represents 1/2 or more of the maximum required shear strength within \( l_o \);

(b) The factored axial compressive force \( P_{au} \) including earthquake effects, is less than \( A_g f'_c / 20 \).

21.5—Joints of moment frames

21.5.1 General requirements

21.5.1.1 Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural tensile reinforcement is \( 1.25 f_y \).

21.5.1.2 Strength of joint shall be governed by the appropriate \( \phi \) factors in 9.3.

21.5.1.3 Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension according to 21.5.4 and in compression according to Chapter 12.

21.5.1.4 Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall not be less than 20 times the diameter of the largest longitudinal beam bar.

21.5.2 Transverse reinforcement

21.5.2.1 Transverse hoop reinforcement in 21.4.4 shall be provided within the joint, unless the joint is confined by structural members in 21.5.2.2.

21.5.2.2 Within \( h \) of the shallowest framing member, transverse reinforcement equal to at least 1/2 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 3/4 the column width. At these locations, the spacing required in 21.4.4.2 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

21.5.3.1 For joints confined on all four faces......................... 20 \(s\frac{f_y}{c} A_j\)

For joints confined on three faces or on two opposite faces ........................................... 15 \(s\frac{f_y}{c} A_j\)

For others.................................................................................. 12 \(s\frac{f_y}{c} A_j\)

A member that frames into a face is considered to provide confinement to the joint if at least 3/4 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.

\(\Delta j\) is the effective cross-sectional area within a joint computed from joint depth times effective joint width. Joint depth shall be the overall depth of the column. Effective joint width shall be the overall width of the column, except where a beam frames into a wider column, effective joint width shall not exceed the smaller of (a) and (b):

(a) beam width plus joint depth;
(b) twice the smaller perpendicular distance from longitudinal axis of beam to column side.

21.5.4 Development length of bars in tension

21.5.4.1 The development length \(\ell_{dh}\) for a bar with a standard 90-degree hook shall not be less than the largest of \(8d_b\), 6 in., and the length required by Eq. (21-6).

\[\ell_{dh} = \frac{f_y d_b}{(65 \sqrt{f_y})}\] (21-6)

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, \(\ell_{d}\), the development length in tension for a straight bar, shall not be less than the larger of (a) and (b):

(a) 2.5 times the length required by 21.5.4.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 12 in.;
(b) 3.25 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 \(\ell_{d}\) not within the confined core shall be increased by a factor of 1.6.

21.5.4.4 If epoxy-coated reinforcement is used, the development lengths in 21.5.4.1 through 21.5.4.3 shall be multiplied by the applicable factor in 12.2.4 or 12.5.2.

21.6—Intentionally left blank

21.7—Reinforced concrete structural walls and coupling beams

21.7.1 Scope—The requirements of this section apply to reinforced concrete structural walls and coupling beams serving as part of the earthquake force-resisting system.

21.7.2 Reinforcement

21.7.2.1 The distributed web reinforcement ratios, \(\rho_v\) and \(\rho_t\), for structural walls shall not be less than 0.0025, except that if \(V_n\) does not exceed \(A_{cv} \sqrt{f_y/c} \rho_v\) and \(\rho_t\) shall be permitted to be reduced to the values required in 14.3. Reinforcement spacing each way in structural walls shall not exceed 18 in. Reinforcement contributing to \(V_n\) shall be continuous and shall be distributed across the shear plane.

21.7.2.2 At least two curtains of reinforcement shall be used in a wall if \(V_n\) exceeds \(2A_{cw} \sqrt{f_y/c}\).

21.7.2.3 Reinforcement in structural walls shall be developed or spliced for \(f_y\) in tension in accordance with Chapter 12 except:

(a) The requirements of 12.11, 12.12 and 12.13 need not be satisfied.
(b) At locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements, the development length of longitudinal reinforcement shall be 1.10 times the value calculated for \(f_y\) in tension.
(c) Mechanical splices of reinforcement shall conform to 21.2.6 and welded splices of reinforcement shall conform to 21.2.7.

21.7.2.4 Web reinforcement at critical sections for flexure and shear shall not be lap spliced. Splices of web reinforcement in these regions, if required, shall comply with the provisions of 21.2.6 and 21.2.7 for mechanical and welded splices, respectively.

21.7.3 Design forces—\(V_n\) shall be obtained by analysis in accordance with the factored load combinations.

21.7.4 Shear strength

21.7.4.1 \(V_n\) of structural walls shall not exceed

\[V_n = A_{cv}(\alpha_c \sqrt{f_y/c} + \rho_t f_y)\] (21-7)

where the coefficient \(\alpha_c\) is 3.0 for \(h_w/l_w \leq 1.5\), is 2.0 for \(h_w/l_w \geq 2.0\), and varies linearly between 3.0 and 2.0 for \(h_w/l_w\) between 1.5 and 2.0.

21.7.4.2 In 21.7.4.1, the value of ratio \(h_w/l_w\) used for determining \(V_n\) for segments of a wall shall be the larger of the ratios for the entire wall and the segment of wall considered.

21.7.4.3 Walls shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. If \(h_w/l_w\) does not exceed 2.0, reinforcement ratio \(\rho_t\) shall not be less than reinforcement ratio \(\rho_r\).

21.7.4.4 For all wall piers sharing a common lateral force, \(V_n\) shall not be taken larger than \(8A_{cw} \sqrt{f_y/c}\), where \(A_{cw}\) is the gross area of concrete bounded by web thickness and length of section. For any one of the individual wall piers, \(V_n\) shall not be taken larger than \(10A_{cw} \sqrt{f_y/c}\), where \(A_{cw}\) is the area of concrete section of the individual pier considered.
21.7.4.5 For horizontal wall segments and coupling beams, \(V_n\) shall not be taken larger than \(10A_{cw} \sqrt{f_c}\), where \(A_{cw}\) is the area of concrete section of a horizontal wall segment or coupling beam.

21.7.5 Design for flexure and axial loads

21.7.5.1 Structural walls and portions of such walls subject to combined flexural and axial loads shall be designed in accordance with 10.2 and 10.3 except that 10.3.6 and the nonlinear strain distribution requirements of 10.2.2 shall not apply. Concrete and developed longitudinal reinforcement within effective flange widths, boundary elements, and the wall web shall be considered effective. The effects of openings shall be considered.

21.7.5.2 Unless a more detailed analysis is performed, effective flange widths of flanged sections shall extend from the face of the web a distance equal to the smaller of 1/2 the effective flange widths of flanged sections shall extend from the wall web shall be considered effective. The effects of openings shall be considered.

21.7.6 Boundary elements of reinforced concrete structural walls

21.7.6.1 For walls and wall piers with \(h_w/l_w\) less than or equal to 2.0, boundary elements are not required.

21.7.6.2 For walls and wall piers with \(h_w/l_w\) greater than 2.0, the need for boundary elements shall be established by cross-section analysis for flexural and axial loads. Boundary elements shall be provided if the maximum compression strain in the cross-section analysis exceeds 0.002. The boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of \(l_w\) or \(M_u/4V_u\).

21.7.6.3 Not used.

21.7.6.4 Where boundary elements are required by 21.7.6.2, (a) through (e) shall be satisfied:
(a) The boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of \(c = \frac{0.1l_w}{\alpha} \) and \(c/2\), where \(c\) is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with design displacement \(\delta_d\);
(b) In flanged sections, the boundary element shall include the effective flange width in compression and shall extend at least 12 in. into the web;
(c) Boundary element transverse reinforcement shall satisfy the requirements of 21.4.4.1 through 21.4.4.3, except Eq. (21-3) need not be satisfied;
(d) Boundary element transverse reinforcement at the wall base shall extend into the support at least the development length of the largest longitudinal reinforcement in the boundary element unless the boundary element terminates on a footing or mat, where boundary element transverse reinforcement shall extend at least 12 in. into the footing or mat;
(e) Horizontal reinforcement in the wall web shall be anchored to develop \(f_y\) within the confined core of the boundary element;

21.7.6.5 Where boundary elements are not required by 21.7.6.2, 21.7.6.5(a) and 21.7.6.5(b) shall be satisfied:
(a) If the longitudinal reinforcement ratio at the wall boundary is greater than \(430/f_y\), boundary transverse reinforcement shall satisfy 21.4.4.1(c), 21.4.4.3, and 21.7.6.4(a). The maximum longitudinal spacing of transverse reinforcement in the boundary shall not exceed the lesser of \(10d_{p}\) or 12 in.
(b) Horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

21.7.7 Coupling beams

21.7.7.1 Coupling beams with aspect ratio \((l_n/h) \geq 4\) shall satisfy the requirements of 21.3. The provisions of 21.3.1.3 and 21.3.1.4 need not be satisfied if it can be shown by analysis that the beam has adequate lateral stability.

21.7.7.2 Coupling beams with aspect ratio \((l_n/h) < 4\) shall be permitted to be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan.

21.7.7.3 Coupling beams with aspect ratio \((l_n/h) < 4\) and with \(V_n\) exceeding \(4\sqrt{f_c/A_{cw}}\) shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan.

21.7.7.4 Coupling beams reinforced with two intersecting groups of diagonally placed bars placed symmetrically about the midspan of the beam shall satisfy (a) through (f):
(a) Each group of diagonally placed bars shall consist of a minimum of four bars assembled in a core having sides measured to the outside of transverse reinforcement no smaller than \(b_{w}/2\) perpendicular to the plane of the beam and \(b_{w}/5\) in the plane of the beam and perpendicular to the diagonal bars;
(b) \(V_n\) shall be determined by
\[
V_n = 2A_{yd}f_y \sin \alpha \leq 10\sqrt{f_c/A_{cw}} \quad (21-8)
\]
where \(\alpha\) is the angle between the diagonally placed bars and the longitudinal axis of the coupling beam.
(c) Each group of diagonally placed bars shall be enclosed in transverse reinforcement satisfying 21.4.4.1 through 21.4.4.3. For the purpose of computing \(A_g\) for use in Eq. (10-5) and (21-3), the minimum concrete cover as required in 7.7 shall be assumed on all four sides of each group of diagonally placed reinforcing bars;
(d) The diagonally placed bars shall be developed for tension in the wall;
(e) The diagonally placed bars shall be considered to contribute to \(M_n\) of the coupling beam;
(f) Reinforcement parallel and transverse to the longitudinal axis shall be provided and, as a minimum, shall conform to 11.8.4 and 11.8.5.

21.7.8 Construction joints—All construction joints in structural walls shall conform to 6.4 and contact surfaces shall be roughened as in 11.7.9.

21.7.9 Discontinuous walls—Columns supporting discontinuous structural walls shall be reinforced in accordance with 21.4.4.5.
21.9—Structural diaphragms and trusses

21.9.1 Scope—Floor and roof slabs acting as structural diaphragms to transmit forces induced by earthquake ground motions shall be designed in accordance with this section. This section also applies to struts, ties, chords, and collector elements that transmit forces induced by earthquakes, as well as trusses serving as parts of the earthquake force-resisting systems.

21.9.2 Cast-in-place composite-topping slab diaphragms—A composite-topping slab cast in place on a precast floor or roof shall be permitted to be used as a structural diaphragm provided the topping slab is reinforced and its connections are proportioned and detailed to provide for a complete transfer of forces to chords, collector elements, and the lateral-force-resisting system. The surface of the previously hardened concrete on which the topping slab is placed shall be clean, free of laitance, and intentionally roughened.

21.9.3 Cast-in-place topping slab diaphragms—A cast-in-place noncomposite topping on a precast floor or roof shall be permitted to serve as a structural diaphragm, provided the cast-in-place topping acting alone is proportioned and detailed to resist the design forces.

21.9.4 Minimum thickness of diaphragms—Concrete slabs and composite topping slabs serving as structural diaphragms used to transmit earthquake forces shall not be less than 2 in. thick. Topping slabs placed over precast floor or roof elements, acting as structural diaphragms and not relying on composite action with the precast elements to resist the design seismic forces, shall have thickness not less than 2-1/2 in.

21.9.5 Reinforcement

21.9.5.1 The minimum reinforcement ratio for structural diaphragms shall be in conformance with 7.12. Reinforcement spacing each way in non-post-tensioned floor or roof systems shall not exceed 18 in. Where welded wire reinforcement is used as the distributed reinforcement to resist shear in topping slabs placed over precast floor and roof elements, the wires parallel to the span of the precast elements shall be spaced not less than 10 in. on center. Reinforcement provided for shear strength shall be continuous and shall be distributed uniformly across the shear plane.

21.9.5.2 Bonded tendons used as primary reinforcement in diaphragm chords or collectors shall be proportioned such that the stress due to design seismic forces does not exceed 60,000 psi. Precompression from unbonded tendons shall be permitted to resist diaphragm design forces if a complete load path is provided.

21.9.5.3 If the design seismic forces in structural truss elements, struts, ties, diaphragm chords, and collector elements exceed 50% of the design strength at any section, transverse reinforcement as given in 21.4.4.1 through 21.4.4.3 shall be provided over the length of the element. The special transverse reinforcement can be discontinued where the design seismic forces are less than 50% of the design strengths.

21.9.6 Intentionally left blank.

21.9.7 Shear strength

21.9.7.1 $V_n$ of structural diaphragms shall not exceed

\[
V_n = A_{cv}(2 \sqrt{f'} + \rho f_y) \tag{21-9}
\]

21.9.7.2 $V_n$ of cast-in-place composite-topping slab diaphragms and cast-in-place noncomposite topping slab diaphragms on a precast floor or roof shall not exceed

\[
V_n = A_{cv} f_y \tag{21-10}
\]

where $A_{cv}$ is based on the thickness of the topping slab. The required web reinforcement shall be distributed uniformly in both directions.

21.9.7.3 Nominal shear strength shall not exceed $84 \sqrt{f'}$, where $A_{cv}$ is the gross area of the diaphragm cross section.

21.9.8 Boundary elements of structural diaphragms

21.9.8.1 Boundary elements of structural diaphragms shall be proportioned to resist the sum of the factored axial forces acting in the plane of the diaphragm and the force obtained from dividing $M_y$ at the section by the distance between the boundary elements of the diaphragm at that section.

21.9.8.2 Lap splices of tension reinforcement in the chords and collector elements of diaphragms shall be Class B splices per 12.15.1 where $\ell_d$ is given by 21.7.2.3. Mechanical and welded splices shall conform to 21.2.6 and 21.2.7, respectively.

21.9.8.3 Reinforcement for chords and collectors at splices and anchorage zones shall satisfy either (a) or (b):

(a) A minimum center-to-center spacing of three longitudinal bar diameters, but not less than 1-1/2 in., and a minimum concrete clear cover of 2-/12 longitudinal bar diameters, but not less than 2 in.; or

(b) Transverse reinforcement as required by 11.5.6.3, except as required in 21.9.5.3.

21.9.9 Construction joints—All construction joints in diaphragms shall conform to 6.4 and contact surfaces shall be roughened in conformance with 11.7.9.

21.10—Foundations

21.10.1 Scope—Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground shall comply with 21.10 and other applicable Code provisions.

21.10.2 Footings, foundation mats, and pile caps

21.10.2.1 Longitudinal reinforcement of columns and structural walls resisting forces induced by earthquake effects shall extend into the footing, mat, or pile cap, and shall be fully developed for tension at the interface.
21.10.2.2 Columns designed assuming fixed-end conditions at the foundation shall comply with 21.10.2.1 and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90-degree hooks near the bottom of the foundation with the free end of the bars oriented toward the center of the column.

21.10.2.3 Columns or boundary elements of reinforced concrete structural walls that have an edge within 1/2 the footing depth from an edge of the footing shall have transverse reinforcement in accordance with 21.4.4 provided below the top of the footing. This reinforcement shall extend into the footing a distance no less than the smaller of the depth of the footing, mat, or pile cap, or the development length in tension of the longitudinal reinforcement.

21.10.2.4 Where earthquake effects create uplift forces in boundary elements of reinforced concrete structural walls or columns, flexural reinforcement shall be provided in the top of the footing, mat, or pile cap to resist the design load above the top of the footing, mat, or pile cap to resist the design load above the top of the footing, mat, or pile cap.

21.10.3 Grade beams and slabs-on-ground

21.10.3.1 Grade beams designed to act as horizontal ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supported column or anchored within the pile cap or footing at all discontinuities.

21.10.3.2 Grade beams designed to act as horizontal ties between pile caps or footings shall be proportioned such that the smallest cross-sectional dimension shall be equal to or greater than the clear spacing between connected columns divided by 20, but need not be greater than 18 in. Closed ties shall be provided at a spacing not to exceed the lesser of 1/2 the smallest orthogonal cross-sectional dimension or 12 in.

21.10.3.3 Grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the lateral-force-resisting system shall conform to 21.3.

21.10.3.4 Slabs-on-ground that resist seismic forces from walls or columns that are part of the lateral-force-resisting system shall be designed as structural diaphragms in accordance with 21.9. The design drawings shall clearly state that the slab-on-ground is a structural diaphragm and part of the lateral-force-resisting system.

21.10.4 Piles, piers, and caissons

21.10.4.1 Provisions of 21.10.4 shall apply to concrete piles, piers, and caissons supporting structures designed for earthquake resistance.

21.10.4.2 Piles, piers, or caissons resisting tension loads shall have continuous longitudinal reinforcement over the length resisting design tension forces. The longitudinal reinforcement shall be detailed to transfer tension forces within the pile cap to supported structural members.

21.10.4.3 Where tension forces induced by earthquake effects are transferred between pile cap or mat foundation and precast pile by reinforcing bars grouted or post-installed in the top of the pile, the grouting system shall have been demonstrated by test to develop the tensile strength of the bar.

21.10.4.4 Piles, piers, or caissons shall have transverse reinforcement in accordance with 21.4.4 at locations (a) and (b):

(a) At the top of the member for at least five times the member cross-sectional dimension, but not less than 6 ft below the bottom of the pile cap;

(b) For the portion of piles in soil that is not capable of resisting flexure from columns that are part of the lateral-force-resisting system.

21.10.4.5 For precast concrete driven piles, the length of transverse reinforcement provided shall be sufficient to account for potential variations in the elevation in pile tips.

21.10.4.6 Intentionally left blank.

21.10.4.7 Pile caps incorporating batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns. The slenderness effects of batter piles shall be considered for the portion of the piles in soil that is not capable of providing lateral support, or in air or water.

APPENDIX A—STRUT-AND-TIE MODELS

A.1—Definitions

B-region—a portion of a member in which the plane sections assumption of flexure theory from 10.2.2 can be applied.

discontinuity—an abrupt change in geometry or loading.

D-region—the portion of a member within a distance h from a force discontinuity or a geometric discontinuity.

deep beam—see 10.7.1 and 11.8.1.

node—the point in a joint in a strut-and-tie model where the axes of the struts, ties, and concentrated forces acting on the joint intersect.

nodal zone—the volume of concrete around a node that is assumed to transfer strut-and-tie forces through the node.

strut—a compression member in a strut-and-tie model. A strut represents the resultant of a parallel or a fan-shaped compression field.

bottle-shaped strut—a strut that is wider at midlength than at its ends.

strut-and-tie model—a truss model of a structural member, or of a D-region in such a member, made up of struts and ties connected at nodes, capable of transferring the factored loads to the supports or to adjacent B-regions.

tie—a tension member in a strut-and-tie model.

A.2—Strut-and-tie model design procedure

A.2.1 It shall be permitted to design structural concrete members, or D-regions in such members, by modeling the member or region as an idealized truss. The truss model shall contain struts, ties, and nodes as defined in A.1. The truss model shall be capable of transferring all factored loads to the supports or adjacent B-regions.

A.2.2 The strut-and-tie model shall be in equilibrium with the applied loads and the reactions.

A.2.3 In determining the geometry of the truss, the dimensions of the struts, ties, and nodal zones shall be taken into account.

A.2.4 Ties shall be permitted to cross struts. Struts shall cross or overlap only at nodes.
A.2.5 The angle $\phi$ between the axes of any strut and any tie entering a single node shall not be taken as less than 25 degrees.

A.2.6 Design of struts, ties, and nodal zones shall be based on

$$\phi F_n \geq F_u$$  \hspace{1cm} (A-1)

where $F_u$ is the factored force acting in a strut, in a tie, or on one face of a nodal zone; $F_n$ is the nominal strength of the strut, tie, or nodal zone; and $\phi$ is specified in 9.3.2.6.

A.3—Strength of struts

A.3.1 The nominal compression strength of a strut without longitudinal reinforcement, $F_{ns}$, shall be taken as the smaller value of

$$F_{ns} = f_{ce}A_{cs}$$  \hspace{1cm} (A-2)

at the two ends of the strut, where $A_{cs}$ is the cross-sectional area at one end of the strut, and $f_{ce}$ is the smaller of (a) and (b):

(a) the effective compressive strength of the concrete in the strut given in A.3.2;

(b) the effective compressive strength of the concrete in the nodal zone given in A.5.2.

A.3.2 The effective compressive strength of the concrete, $f_{ce}$, in a strut shall be taken as

$$f_{ce} = 0.85\beta_s f_c'$$  \hspace{1cm} (A-3)

A.3.2.1 For a strut of uniform cross-sectional area over its length ............... $\beta_s = 1.0$

A.3.2.2 For struts located such that the width of the midsection of the strut is larger than the width at the nodes (bottle-shaped struts):

(a) with reinforcement satisfying A.3.3 ............... $\beta_s = 0.75$

(b) without reinforcement satisfying A.3.3 ............... $\beta_s = 0.60$

A.3.2.3 For struts in tension members, or the tension flanges of members, ............... $\beta_s = 0.40$

A.3.2.4 For all other cases ............... $\beta_s = 0.60$

A.3.3 If the value of $\beta_s$ specified in A.3.2.2(a) is used, the axis of the strut shall be crossed by reinforcement proportioned to resist the transverse tensile force resulting from the compression force spreading in the strut. It shall be permitted to assume the compressive force in the strut spreads at a slope of 2 longitudinal to 1 transverse to the axis of the strut.

A.3.3.1 For $f_c'$ not greater than 6000 psi, the requirement of A.3.3 shall be permitted to be satisfied by the axis of the strut being crossed by layers of reinforcement that satisfy Eq. (A-4)

$$\sum \frac{A_{si}}{b_i s_i} \sin \alpha_i \geq 0.003$$  \hspace{1cm} (A-4)

where $A_{si}$ is the total area of surface reinforcement at spacing $s_i$ in the $i$-th layer of reinforcement crossing a strut at an angle $\alpha_i$ to the axis of the strut.

A.3.3.2 The reinforcement required in A.3.3 shall be placed in either two orthogonal directions at angles $\alpha_1$ and $\alpha_2$ to the axis of the strut, or in one direction at an angle $\alpha$ to the axis of the strut. If the reinforcement is in only one direction, $\alpha$ shall not be less than 40 degrees.

A.3.4 If documented by tests and analyses, it shall be permitted to use an increased effective compressive strength of a strut due to confining reinforcement.

A.3.5 The use of compression reinforcement shall be permitted to increase the strength of a strut. Compression reinforcement shall be properly anchored, parallel to the axis of the strut, located within the strut, and enclosed in ties or spirals satisfying 7.10. In such cases, the nominal strength of a longitudinally reinforced strut is

$$F_{ns} = f_{ce}A_{cn} + A_f'f_c'$$  \hspace{1cm} (A-5)

A.4—Strength of ties

A.4.1 The nominal strength of a tie, $F_{nt}$, shall be taken as

$$F_{nt} = A_{ts}f_y + A_{tp}(f_{se} + \Delta f_p)$$  \hspace{1cm} (A-6)

where $(f_{se} + \Delta f_p)$ shall not exceed $f_{py}$, and $A_{tp}$ is zero for nonprestressed members.

In Eq. (A-6), it shall be permitted to take $\Delta f_p$ equal to 60,000 psi for bonded prestressed reinforcement, or 10,000 psi for unbonded prestressed reinforcement. Other values of $\Delta f_p$ shall be permitted when justified by analysis.

A.4.2 The axis of the reinforcement in a tie shall coincide with the axis of the tie in the strut-and-tie model.

A.4.3 Tie reinforcement shall be anchored by mechanical devices, post-tensioning anchorage devices, standard hooks, or straight bar development as required by A.4.3.1 through A.4.3.4.

A.4.3.1 Nodal zones shall develop the difference between the tie force on one side of the node and the tie force on the other side.

A.4.3.2 At nodal zones anchoring one tie, the tie force shall be developed at the point where the centroid of the reinforcement in a tie leaves the extended nodal zone and enters the span.

A.4.3.3 At nodal zones anchoring two or more ties, the tie force in each direction shall be developed at the point where the centroid of the reinforcement in the tie leaves the extended nodal zone.

A.4.3.4 The transverse reinforcement required by A.3.3 shall be anchored in accordance with 12.13.

A.5—Strength of nodal zones

A.5.1 The nominal compression strength of a nodal zone, $F_{nn}$, shall be

$$F_{nn} = f_{ce}A_{nz}$$  \hspace{1cm} (A-7)

where $f_{ce}$ is the effective compressive strength of the concrete in the nodal zone as given in A.5.2 and $A_{nz}$ is the smaller of (a) and (b):

(a) the area of the face of the nodal zone on which $F_u$ acts, taken perpendicular to the line of action of $F_u$. 

(b) the effective compressive strength of the concrete in the face of the nodal zone.
(b) the area of a section through the nodal zone, taken perpendicular to the line of action of the resultant force on the section.

A.5.2 Unless confining reinforcement is provided within the nodal zone and its effect is supported by tests and analysis, the calculated effective compressive stress $f_{ce}$ on a face of a nodal zone due to the strut-and-tie forces shall not exceed the value given by

$$f_{ce} = 0.85\beta_nf'_c$$

(A-8)

where the value of $\beta_n$ is given in A.5.2.1 through A.5.2.3.

A.5.2.1 In nodal zones bounded by struts or bearing areas, or both $\beta_n = 1.0$

A.5.2.2 In nodal zones anchoring one tie $\beta_n = 0.80$

or

A.5.2.3 In nodal zones anchoring two or more ties $\beta_n = 0.60$

A.5.3 In a three-dimensional strut-and-tie model, the area of each face of a nodal zone shall not be less than that given in A.5.1, and the shape of each face of the nodal zones shall be similar to the shape of the projection of the end of the struts onto the corresponding faces of the nodal zones.

**APPENDIX B—INTENTIONALLY LEFT BLANK**

**APPENDIX C—ALTERNATIVE LOAD AND STRENGTH-REDUCTION FACTORS**

**C.1—General**

C.1.1 Structural concrete shall be permitted to be designed using the load combinations and strength-reduction factors of Appendix C. Chapter 9 notations are applicable to this Appendix.

**C.2—Required strength**

C.2.1 Required strength $U$ shall be at least equal to the greatest of the following

$$U = 1.05D + 1.05F + 1.3L + 1.3H + 1.05T_0 + 1.3R_0$$

(C-9)

$$U = 1.05D + 1.05F + 1.3L + 1.3E_0 + 1.05T_0 + 1.3R_0$$

(C-10)

$$U = 1.05D + 1.05F + 1.3L + 1.3H + 1.3E_0 + 1.05T_0 + 1.3R_0$$

(C-11)

C.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 Eq. (C-4) through (C-11). Estimation of these effects shall be based on a realistic assessment of such effects occurring in service.

C.2.3 For the load combinations in C.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.

C.2.4 Where applicable, impact effects of moving loads shall be included with the crane load $C_{cr}$.

C.2.5 In Load Combinations (C-6) to (C-8), the maximum values of $P_a$, $T_a$, $R_a$, $Y_f$, $Y_r$, and $Y_m$, including an appropriate dynamic load factor, shall be used unless an appropriate time-history analysis is performed to justify otherwise.

C.2.6 Load Combinations (C-5), (C-7), and (C-8) shall be satisfied first without the tornado missile load in (C-5), and without $Y_f$, $Y_r$, and $Y_m$ in (C-7) and (C-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 F.

C.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 (C-5).

C.2.8 For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum prestressing steel jacking force.

C.2.9 In load combination (C-4), the crane load $C_{cr}$ may be omitted if probability analysis demonstrates that the simultaneous occurrence of a SSE (DBE) with crane usage is not credible.

**C.3—Design strength**

C.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 the $\phi$ factors in C.3.2.

C.3.2 Strength reduction-factor $\phi$ shall be as follows:

C.3.2.1 Tension-controlled sections, as defined in 10.3.4

(See also C.3.2.7) .......................................................... $0.90$
C.3.2.2 Compression-controlled sections, as defined in 10.3.3:
(a) Members with spiral reinforcement conforming to 10.9.3 .................................................. 0.75
(b) Other reinforced members ........................................ 0.70
For sections in which the net tensile strain in the extreme tension steel at nominal strength, \( \varepsilon_t \), is between the limits for compression-controlled and tension-controlled sections, \( \phi \) shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as \( \varepsilon_t \) increases from the compression-controlled strain limit to 0.005.

C.3.2.3 Shear and torsion .................................................. 0.85
C.3.2.4 Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models) ......................... 0.85
C.3.2.5 Post-tensioned anchorage zones ......................... 0.70
C.3.2.6 Strut-and-tie models (Appendix A), and struts, ties, nodal zones, and bearing areas in such models........ 0.85
C.3.2.7 Flexure sections without axial load in pretensioned members where strand embedment is less than the development length as provided in 12.9.1.1 .................. 0.85

C.3.3 Development lengths specified in Chapter 12 do not require a \( \phi \)-factor.

APPENDIX D—ANCHORING TO CONCRETE

D.1—Definitions
anchor—a steel element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads, including headed bolts, 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 loads 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 concrete spall behind short, stiff anchors 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 D.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 element meeting the requirements of ASTM A 307 shall be considered ductile.
direct shear—a shear acting on a sleeve or plug and the expansion is controlled by the length of travel of the sleeve or plug.
expansion anchor—a post-installed anchor inserted into hardened concrete that transfers loads to or from the concrete by direct bearing or friction or both. Expansion anchors may be torque-controlled, where the expansion is achieved by a torque acting on the screw or bolt; or displacement-controlled, where the expansion is achieved by impact forces acting on a sleeve or plug and the expansion is controlled by the length of travel of the sleeve or plug.
expansion sleeve—the outer part of an expansion anchor that is forced outward by the center part, either by applied torque or impact, to bear against the sides of the predrilled hole.
five percent fractile—a statistical term meaning 90\% confidence that there is 95\% probability of the actual strength exceeding the nominal strength.
head 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 members.
supplementary reinforcement—reinforcement proportioned to tie a potential concrete failure prism to the structural member.
undercut anchor—a post-installed anchor that develops its tensile strength from the mechanical interlock provided by undercutting of the concrete at the embedded end of the anchor. The undercutting is achieved with a special drill.
before installing the anchor or alternatively by the anchor itself during its installation.

**D.2—Scope**

**D.2.1** This appendix provides design requirements for structural embedments in concrete used to transmit structural loads by means of tension, shear, or a combination of tension and shear between (a) connected structural members; or (b) safety-related attachments and structural members. Safety levels specified are intended for in-service conditions, rather than for short-term handling and construction conditions.

**D.2.2** This appendix applies to both cast-in anchors and post-installed anchors. Through bolts, multiple anchors connected to a single steel plate at the embedded end of the 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 this Code. Grouted embedments shall meet the requirements of D.12.

**D.2.3** Headed studs and headed bolts having a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal or exceeding \(1.4\phi N_p\) (where \(N_p\) is given by Eq. (D-13)) are included. Post-installed anchors are included provided that D.3.3 is satisfied.

**D.2.4** Load applications that are predominantly high-cycle fatigue are not covered by this appendix.

**D.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.

**D.2.6** The jurisdiction of this Code covers steel material below the surface of the concrete and the anchors extending above the surface of the concrete. The requirements for the attachment to the embedment shall be in accordance with applicable Codes and are beyond the scope of this appendix.

**D.3—General requirements**

**D.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.

**D.3.2** The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in 9.2 or C.2.

**D.3.3** Post-installed structural anchors shall be tested before use, simulating the conditions of the intended field of application, 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 licensed professional engineer with full description and details of the testing programs, procedures, results, and conclusions.

**D.3.4** All provisions for anchor axial tension and shear strength apply to normalweight concrete only.

**D.3.5** The values of \(f'c\) used for calculation purposes in this appendix shall not exceed 10,000 psi for cast-in anchors, and 8000 psi for post-installed anchors. Testing is required for post-installed anchors when used in concrete with \(f'c\) greater than 8000 psi.

**D.3.6 Embedment design**

**D.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 D.4.4(a) or D.4.5(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 nominal tensile strength of the embedment steel and when the design concrete breakout shear strength and design concrete pryout strength exceed the nominal shear strength of the embedment steel. The design concrete tensile strength, the design side blowout strength, the design pullout strength, the design concrete pryout strength, and the design concrete breakout shear strength shall be taken as 0.85 times the nominal strengths.

**D.3.6.2** As an alternate to D.3.6.1, the attachment shall be designed to yield at a load level corresponding to anchor or group forces not greater than 75% of the anchor design strength specified in D.4.1.2. The anchor design strength shall be determined using the strength-reduction factors specified in D.4.4 or D.4.5.

**D.3.6.3** It shall be permitted to design anchors as nonductile anchors for tension or shear loading, or both. The design strength of such anchors shall be taken as \(0.60\phi N_n\) and \(0.60\phi V_n\), where \(\phi\) is given in D.4.4 or D.4.5, and \(N_n\) and \(V_n\) are determined in accordance with D.4.1.

**D.3.7** 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.

**D.3.8** Embedment materials for ductile anchors other than reinforcing bars shall be ductile steel elements.

**D.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 nominal tensile strength of the reduced section shall be greater than the yield strength of the unreduced section;

(b) For bolts, the length of thread in the load path shall be at least two anchor diameters.

**D.3.10** The design strength of embedment materials is permitted to be increased in accordance with Appendix F for embedments subject to impactive and impulsive loads.

**D.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 D.3.6.

**D.4—General requirements for strength of anchors**

**D.4.1** Strength design of anchors shall be based either on computation using design models that satisfy the require-
ments of D.4.2, or on test evaluation using the 5% fractile of test results for the following:
(a) steel strength of anchor in tension (D.5.1);
(b) steel strength of anchor in shear (D.6.1);
(c) concrete breakout strength of anchor in tension (D.5.2);
(d) concrete breakout strength of anchor in shear (D.6.2);
(e) pullout strength of anchor in tension (D.5.3);
(f) concrete side-face blowout strength of anchor in tension
(D.5.4); and
(g) concrete pryout strength of anchor in shear (D.6.3).

In addition, anchors shall satisfy the required edge distances, spacings, and thicknesses to preclude splitting failure, as required in D.8.

D.4.1.1 For the design of anchors,
\[ \phi V_n \geq V_{ua} \quad (D-1) \]
\[ \phi N_n \geq N_{ua} \quad (D-2) \]

D.4.1.2 In Eq. (D-1) and (D-2), \( \phi N_n \) and \( \phi V_n \) are the lowest design strengths determined from all appropriate failure modes. \( \phi N_n \) is the lowest design strength in tension of an anchor or group of anchors as determined from consideration of \( \phi N_{sa} \), \( \phi N_{pu} \), either \( \phi N_{eb} \) or \( \phi N_{ebg} \), and either \( \phi N_{cb} \) or \( \phi N_{cbg} \). \( \phi V_n \) is the lowest design strength in shear of an anchor or a group of anchors as determined from consideration of: \( \phi V_{sa} \), either \( \phi V_{eb} \) or \( \phi V_{ebg} \), and either \( \phi V_{cp} \) or \( \phi V_{cpbg} \).

D.4.1.3 When both \( N_{ua} \) and \( V_{ua} \) are present, interaction effects shall be considered in accordance with D.4.3.

D.4.2 The nominal strength for any anchor or group of anchors shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests. The materials used in the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5% fractile of the basic individual anchor strength. For nominal strengths related to concrete strength, modifications for size effects, the number of anchors, the effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loadings of anchor groups, and presence or absence of cracking shall be taken into account. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

D.4.2.1 The effect of supplementary reinforcement provided to confine or restrain the concrete breakout, or both, shall be permitted to be included in the design models used to satisfy D.4.2.

D.4.2.2 For anchors with diameters not exceeding 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 D.5.2 and D.6.2.

D.4.3 Resistance to combined tensile and shear loads shall be considered in design using an interaction expression that results in computation of strength in substantial agreement with results of comprehensive tests. This requirement shall be considered satisfied by D.7.

D.4.4 Strength-reduction factor \( \phi \) for anchors in 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 .............................................0.75
   ii) Shear loads .............................................0.65
(c) Anchor governed by concrete breakout, side-face blowout, pullout, or pryout strength
   i) Shear loads .............................................0.75
   ii) Tension loads .............................................0.70
   iii) Cast-in headed studs or headed bolts 0.85 0.75
   iv) Post-installed 0.85 0.75
   v) Embedded plates and shear lugs
       a) Shear toward free edge ............................0.80
       b) Cast-in headed studs or headed bolts 0.90 0.70
       c) Post-installed 0.90 0.70

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.

(d) Anchor controlled by concrete bearing .....................0.65
(e) Structural plates, shapes, and specialty inserts
   i) Tension, compression, and bending loads ........0.90
   ii) Shear loads .............................................0.55
(f) Embedded plates and shear lugs
   a) Shear toward free edge ............................0.80
   b) Cast-in headed studs or headed bolts 0.75 0.70
   c) Post-installed 0.75 0.65
   d) Embedded plates and shear lugs

D.4.5 Strength-reduction factor \( \phi \) for anchors in concrete shall be as follows when the load combinations referenced in Appendix C 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
   i) Shear loads .............................................0.85
   ii) Tension loads .............................................0.75
   a) Cast-in headed studs or headed bolts 0.85 0.75
   b) Post-installed 0.85 0.75

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.

(d) Anchor controlled by concrete bearing .....................0.70
(e) Structural plates, shapes, and specialty inserts
   i) Tension, compression, and bending loads ........0.90
   ii) Shear loads .............................................0.55
(f) Embedded plates and shear lugs

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
Shear toward free edge............................................... 0.85

D.4.6 Bearing strength

D.4.6.1 A combination of bearing and shear friction mechanisms shall not be used to determine the nominal shear strength of embedments in accordance with 9.2 or C.2. If the requirements of 9.2.3 (or C.2.6) are satisfied, however, it shall be 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.

D.4.6.2 The design bearing strength used for concrete or grout placed against shear lugs shall not exceed with shear lugs.

D.5—Design requirements for tensile loading

D.5.1 Steel strength of anchor in tension

D.5.1.1 The nominal strength of an anchor in tension as governed by the steel, \( N_{sa} \), shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

\[
N_{sa} = nA_{se}f_{uta}
\]  

(D-3)

where \( n \) is the number of anchors in the group, and \( f_{uta} \) shall not be taken greater than the smaller of 1.95 \( f_{sa} \) and 125,000 psi.

D.5.2 Concrete breakout strength of anchor in tension

D.5.2.1 The nominal concrete breakout strength, \( N_{cb} \) or \( N_{cbg} \) of a single anchor or group of anchors in tension shall not exceed

(a) for a single anchor

\[
N_{cb} = \frac{A_{sc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b
\]  

(D-4)

(b) for a group of anchors

\[
N_{cbg} = \frac{A_{sc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b
\]  

(D-5)

Factors \( \psi_{ed,N} \), \( \psi_{c,N} \), \( \psi_{cp,N} \) are defined in D.5.2.4, D.5.2.5, D.5.2.6, and D.5.2.7, respectively. \( A_{sc} \) is the projected concrete failure area of a single anchor or group of anchors that shall be approximated as the base of the rectangular geometrical figure that results from projecting the failure surface outward 1.5 \( h_{ef} \) from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors. \( A_{Nco} \) shall not exceed \( nA_{Nco} \), where \( n \) is the number of tensioned anchors in the group. \( A_{Nco} \) is the projected concrete failure area of a single anchor with an edge distance equal to or greater than 1.5 \( h_{ef} \),

\[
A_{Nco} = 9h_{ef}^2
\]  

(D-6)

D.5.2.2 The basic concrete breakout strength of a single anchor in tension in cracked concrete, \( N_b \), shall not exceed

\[
N_b = k_c \sqrt{f_c'} h_{ef}^{1.5}
\]  

(D-7)

where \( k_c = 24 \) for cast-in anchors; and
\( k_c = 17 \) for post-installed anchors.

The value of \( k_c \) for post-installed anchors shall be permitted to be increased above 17 based on D.3.3 product-specific tests, but shall in no case exceed 24.

Alternatively, for cast-in headed studs and headed bolts with 11 in. \( \leq h_{ef} \leq 25 \) in., \( N_b \) shall not exceed

\[
N_b = 16 \sqrt{f_c'} h_{ef}^{5/3}
\]  

(D-8)

D.5.2.3 Where anchors are located less than 1.5 \( h_{ef} \) from three or more edges, the value of \( h_{ef} \) used in Eq. (D-4) through (D-11) shall be the greater of \( e_{max} \) and 1/3 of the maximum spacing between anchors within the group.

D.5.2.4 The modification factor for anchor groups loaded eccentrically in tension is

\[
\psi_{ec,N} = \frac{1}{1 + \frac{2e_N}{3h_{ef}}} \leq 1.0
\]  

(D-9)

If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension shall be considered when determining the eccentricity \( e_N \) for use in Eq. (D-9) and for the calculation of \( N_{cbg} \) in Eq. (D-5).

In the case where eccentric loading exists about two axes, the modification factor \( \psi_{ec,N} \) shall be computed for each axis individually and the product of these factors used as \( \psi_{ec,N} \) in Eq. (D-5).

D.5.2.5 The modification factor for edge effects for single anchors or anchor groups loaded in tension is

\[
\psi_{ed,N} = 1 \text{ if } e_{a,min} \geq 1.5h_{ef}
\]  

(D-10)

\[
\psi_{ed,N} = 0.7 + 0.3 \frac{e_{a,min} - 1.5h_{ef}}{1.5h_{ef}} \text{ if } e_{a,min} < 1.5h_{ef}
\]  

(D-11)

D.5.2.6 For anchors located in a region of a concrete member where analysis indicates no cracking (\( f_c < f_p \)) under the load combinations specified in 9.2 or C.2 with load factors taken as unity, the following modification factor shall be permitted:

\( \psi_{c,N} = 1.25 \) for cast-in anchors; and
\( \psi_{c,N} = 1.4 \) for post-installed anchors, where the value of \( k_c \) used in Eq. (D-7) is 17.

When analysis indicates cracking under the load combinations specified in 9.2 or C.2 with load factors taken as unity,
ψ_{c,N} shall be taken as 1.0 for both cast-in anchors and post-installed anchors. Post-installed anchors shall be qualified for use in cracked concrete in accordance with D.3.3. The cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with 10.6.4, or equivalent crack control shall be provided by confining reinforcement.

D.5.2.7 The modification factor for post-installed anchors designed for uncracked concrete in accordance with D.5.2.6 without supplementary reinforcement to control splitting is

\[ \psi_{cp,N} = 1.0 \text{ if } c_{a,min} \geq c_{ac} \]

\[ \psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}} \text{ if } c_{a,min} < c_{ac} \]

where the critical distance \( c_{ac} \) is defined in D.8.6.

For all other cases, including cast-in anchors, \( \psi_{cp,N} \) shall be taken as 1.0.

D.5.2.8 Where 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 1.5\( h_{ef} \) from the effective perimeter of the plate or washer. The effective perimeter shall not exceed the value at a section projected outward more than the thickness of the washer or plate from the outer edge of the head of the anchor.

D.5.2.9 For post-installed anchors, it shall be permitted to use a coefficient \( k_c \) in Eq. (D-7) or (D-8) based on the 5% fractile of results from product-specific tests. For such cases, the modification factor \( \psi_{c,N} \) shall be based on a direct comparison between the average ultimate failure loads and the characteristic loads based on the 5% fractile of the product-specific testing in cracked concrete and otherwise identical product-specific testing in uncracked concrete.

D.5.3 Pullout strength of anchor in tension

D.5.3.1 The nominal pullout strength of a single anchor in tension, \( N_{pn} \), shall not exceed

\[ N_{pn} = \psi_{c,p} N_p \]

where \( \psi_{c,p} \) is defined in D.5.3.5.

D.5.3.2 For post-installed expansion and undercut anchors, the values of \( N_p \) shall be based on the 5% fractile of results of tests performed and evaluated according to D.3.3. It is not permissible to calculate the pullout strength in tension for such anchors.

D.5.3.3 For single cast-in headed studs and headed bolts, it shall be permitted to evaluate the pullout strength in tension using D.5.3.4. Alternatively, it shall be permitted to use values of \( N_p \) based on the 5% fractile of tests performed and evaluated in accordance with D.3.3 but without the benefit of friction.

D.5.3.4 The pullout strength in tension of a single headed stud or headed bolt, \( N_p \), for use in Eq. (D-14), shall not exceed

\[ N_p = 8A_{brg}f_c' \]

D.5.3.5 For an anchor located in a region of a concrete member where analysis indicates no cracking \( f_{t} < f_{c} \) under the load combinations specified in 9.2 or C.2 with load factors taken as unity, the following modification factor shall be permitted:

\[ \psi_{c,p} = 1.4 \]

Otherwise, \( \psi_{c,p} \) shall be taken as 1.0.

D.5.4 Concrete side-face blowout strength of a headed anchor in tension

D.5.4.1 For a single-headed anchor with deep embedment close to an edge \( (c_{a1} < 0.4h_{ef}) \), the nominal side-face blowout strength, \( N_{sb} \), shall not exceed

\[ N_{sb} = 160c_{a1}\sqrt{A_{brg}f_y} \]

where \( c_{a2} \) for the single-headed anchor is less than \( 3c_{a1} \), the value of \( N_{sb} \) shall be multiplied by the factor \((1 + c_{a2}/c_{a1})/4\) where \( 1.0 \leq c_{a2}/c_{a1} \leq 3.0 \).

D.5.4.2 For multiple-headed anchors with deep embedment close to an edge \( (c_{a1} < 0.4h_{ef}) \) and anchor spacing less than \( 6c_{a1} \), the nominal strength of anchors along the edge in a group for a side-face blowout failure \( N_{sbg} \) shall not exceed

\[ N_{sbg} = \left(1 + \frac{s}{6c_{a1}}\right)N_{sb} \]

The nominal strength of the group of anchors shall be taken as the nominal strength of the outer anchors along the edge multiplied by the number of rows parallel to the edge.

D.6—Design requirements for shear loading

D.6.1 Steel strength of anchor in shear

D.6.1.1 The nominal strength of an anchor in shear as governed by steel, \( V_{sa} \), shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

D.6.1.2 The nominal strength of a single anchor or group of anchors in shear, \( V_{sa} \), shall not exceed (a) through (c):

(a) for cast-in headed stud anchors

\[ V_{sa} = nA_{se}f_{uta} \]

where \( n \) is the number of anchors in the group and \( f_{uta} \) shall not be taken greater than the smaller of 1.9\( f_{y} \) and 125,000 psi.

(b) for cast-in headed bolt and for post-installed anchors where sleeves do not extend through the shear plane

\[ V_{sa} = n0.6A_{se}f_{uta} \]

where \( n \) is the number of anchors in the group and \( f_{uta} \) shall not be taken greater than the smaller of 1.9\( f_{y} \) and 125,000 psi.

(c) for post-installed anchors where sleeves extend through the shear plane, \( V_{sa} \) shall be based on the results of tests
permitted to evaluate row selected as critical. The value of the top of the half-pyramid is given by the axis of the anchor half-pyramid projected on the side face of the member where a side length parallel to the edge of distance from the edge to this axis.

D.6.2.6, and D.6.2.7, respectively.

When the anchor is installed so that the critical failure plane does not pass through the sleeve, the area of the sleeve in Eq. (D-19) shall be taken as zero.

D.6.1.3 Where anchors are used with built-up grout pads, the nominal strengths of D.6.1.2 shall be multiplied by a 0.80 factor.

D.6.1.4 Friction between the baseplate and concrete shall be permitted to be considered to contribute to the nominal steel shear strength of the anchor in shear. The nominal shear strength resulting from friction between the baseplate and concrete (that is, without any contribution from anchors) may be taken as $0.40C_F$.

D.6.2 Concrete breakout strength of anchor in shear

D.6.2.1 The nominal concrete breakout strength, $V_{cb}$ or $V_{cbg}$, in shear of a single anchor or group of anchors shall not exceed:

(a) for shear force perpendicular to the edge on a single anchor

$$V_{cb} = \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} V_b$$  \hspace{1cm} (D-20)

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

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \psi_{ec} \psi_{ed,V} \psi_{c,V} V_b$$  \hspace{1cm} (D-21)

(c) for shear force parallel to an edge, $V_{cb}$ or $V_{cbg}$ shall be permitted to be twice the value of the shear force determined from Eq. (D-20) or (D-21), respectively, with the shear force assumed to act perpendicular to the edge and with $\psi_{ed,V}$ taken equal to 1.0.

(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.

Factors $\psi_{ec,V}$, $\psi_{ed,V}$, and $\psi_{c,V}$ are defined in D.6.2.5, D.6.2.6, and D.6.2.7, respectively. $V_b$ is the basic concrete breakout strength for a single anchor. $A_{Vc}$ 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 $A_{Vc}$ 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 $c_{a1}$ shall be taken as the distance from the edge to this axis. $A_{Vco}$ shall not exceed $nA_{Vco}$, where $n$ is the number of anchors in the group.

$A_{Vco}$ is the projected area for a single anchor in a deep member with a distance from edges equal or greater than 1.5$c_{a1}$ the direction perpendicular to the shear force. It shall be permitted to evaluate $A_{Vco}$ as the base of a half-pyramid with a side length parallel to the edge of 3$c_{a1}$ and a depth of 1.5$c_{a1}$

$$A_{Vco} = 4.5(c_{a1})^2$$  \hspace{1cm} (D-22)

Where anchors are located at varying distances from the edge and the anchors are welded to the attachment so as to distribute the force to all anchors, it shall be permitted to evaluate the strength based on the distance to the farthest row of anchors from the edge. In this case, it shall be permitted to base the value of $c_{a1}$ on the distance from the edge to the axis of the farthest anchor row that is selected as critical, and all of the shear shall be assumed to be carried by this critical anchor row alone.

D.6.2.2 The basic concrete breakout strength in shear of a single anchor in cracked concrete, $V_b$, shall not exceed

$$V_b = 7\left(\frac{\ell_e}{d_o}\right)^{0.2} \sqrt{\frac{d_o}{f'_{c1}} (c_{a1})^{1.5}}$$  \hspace{1cm} (D-23)

where $\ell_e$ is the load-bearing length of the anchor for shear:

$\ell_e = h_{ef}$ for anchors with a constant stiffness over the full length of embedded section, such as headed studs are post-installed anchors with one tubular shell over full length of the embedment depth;

$\ell_e = h_{ef}$ for torque-controlled expansion anchors with a distance sleeve separated from expansion sleeve; and

in no case shall $\ell_e$ exceed 8$d_o$.

D.6.2.3 For cast-in headed studs or headed bolts that are continuously 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 in shear of a single anchor in cracked concrete, $V_b$, shall not exceed

$$V_b = 8\left(\frac{\ell_e}{d_o}\right)^{0.2} \sqrt{\frac{d_o}{f'_{c1}} (c_{a1})^{1.5}}$$  \hspace{1cm} (D-24)

where $\ell_e$ is defined in D.6.2.2, 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) anchor spacing $s$ is not less than 2.5 in.; and

(c) supplementary reinforcement is provided at the corners if $c_{a2} \leq 1.5h_{ef}$.

D.6.2.4 Where anchors are influenced by three or more edges, the value of $c_{a1}$ used in Eq. (D-20) through (D-27) shall not exceed the greatest of: $c_{a2}/1.5$ in either direction, $h_{ef}/1.5$; and 1/3 of the maximum spacing between anchors within the group.

D.6.2.5 The modification factor for anchor groups loaded eccentrically in shear is

$$\psi_{ec,V} = \frac{1}{\left(1 + \frac{2e_{V}'}{3c_{a1}}\right)} \leq 1$$  \hspace{1cm} (D-25)

If the loading on an anchor group is such that only some anchors are loaded in shear in the same direction of the free edge, only those anchors that are loaded in shear in the direction of the free edge shall be considered when determining the eccentricity of $e_{V}'$ for use in Eq. (D-25) and for the calculation of $V_{cbg}$ in Eq. (D-21).
D.6.2.6 The modification factor for edge effect for a single anchor or group of anchors loaded in shear is

\[ \psi_{ed,V} = 1.0 \text{ if } c_{a2} \geq 1.5c_{a1} \]  \hspace{1cm} (D-26)

\[ \psi_{ed,V} = 0.7 + 0.3 \frac{c_{a2}}{1.5c_{a1}} \text{ if } c_{a2} < 1.5c_{a1} \]  \hspace{1cm} (D-27)

D.6.2.7 For anchors located in a region of a concrete member where analysis indicates no cracking \((f_t < f_c)\) under the load combinations specified in 9.2 or C.2 with load factors taken as unity, the following modification factors shall be permitted

\[ \psi_{e,V} = 1.4 \]

For anchors located in a region of a concrete member where analysis indicates cracking under the load combinations specified in 9.2 or C.2 with load factors taken as unity, the following modification factors shall be permitted

\[ \psi_{e,V} = 1.0 \text{ for anchors in cracked concrete with no supplementary reinforcement or edge reinforcement smaller than a No. 4 bar;} \]

\[ \psi_{e,V} = 1.2 \text{ for anchors in cracked concrete with supplementary reinforcement of a No. 4 bar or greater between the anchor and the edge; and} \]

\[ \psi_{e,V} = 1.4 \text{ for anchors in cracked concrete with supplementary reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the supplementary reinforcement enclosed within stirrups spaced at not more than 4 in.} \]

To be considered as supplementary reinforcement, the reinforcement shall be designed to intersect the concrete breakout failure surface defined in D.6.2.1.

D.6.3 Concrete pryout strength of anchor in shear

D.6.3.1 The nominal pryout strength, \(V_{cp}\) or \(V_{cpg}\), shall not exceed

(a) for a single anchor

\[ V_{cp} = k_{cp}N_{eb} \]  \hspace{1cm} (D-28)

(b) for a group of anchors

\[ V_{cpg} = k_{cp}N_{ebg} \]  \hspace{1cm} (D-29)

where

\[ k_{cp} = 1.0 \text{ for } h_{ef} < 2.5 \text{ in.}; \text{ and} \]

\[ k_{cp} = 2.0 \text{ for } h_{ef} \geq 2.5 \text{ in.} \]

\(N_{eb}\) and \(N_{ebg}\) shall be determined from Eq. (D-4) and (D-5), respectively.

D.7—Interaction of tensile and shear forces

Unless determined in accordance with D.4.3, anchors or groups of anchors that are subjected to both shear and axial loads shall be designed to satisfy the requirements of D.7.1 through D.7.3. The value of \(\phi N_n\) shall be as required in D.4.1.2. The value of \(\phi V_n\) shall be as defined in D.4.1.2.

D.7.1 If \(V_{ua} < 0.2\phi V_n\), then full strength in tension shall be permitted: \(\phi N_n \geq N_{ua}\).

D.7.2 If \(N_{ua} \leq 0.2\phi N_n\), then full strength in shear shall be permitted: \(\phi V_n \geq N_{ua}\).

D.7.3 If \(V_{ua} > 0.2\phi V_n\) and \(N_{ua} > 0.2\phi N_n\), then

\[ \frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \]  \hspace{1cm} (D-30)

D.8—Required edge distances, spacings, and thicknesses to preclude splitting failure

Minimum spacings and edge distances for anchors and minimum thicknesses of members shall conform to D.8.1 through D.8.6, unless supplementary reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with D.3.3 shall be permitted.

D.8.1 Minimum center-to-center spacing of anchors shall be \(4d_{o}\) for untorqued cast-in anchors, and \(6d_{o}\) for torqued cast-in anchors and post-installed anchors.

D.8.2 Minimum edge distances for cast-in headed anchors that will not be torqued shall be based on minimum cover requirements for reinforcement in 7.7. For cast-in headed anchors that will be torqued, the minimum edge distances shall be based on the greater of the minimum cover requirements for reinforcement in 7.7 or \(6d_{o}\).

D.8.3 Minimum edge distances for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement in 7.7, or the minimum edge distance requirements for the products as determined by tests in accordance with D.3.3, and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific test information, the minimum edge distance shall be taken as not less than:

- Undercut anchors…………………………………….. \(6d_{o}\)
- Torque-controlled anchors………………………….. \(8d_{o}\)
- Displacement-controlled anchors …………………….. \(10d_{o}\)

D.8.4 Deleted section.

D.8.5 The value of \(h_{ef}\) for an expansion or undercut post-installed anchor shall not exceed the greater of 2/3 of the member thickness and the member thickness less 4 in.

D.8.6 Unless determined from tension tests in accordance with D.3.3, the critical edge distance \(c_{ac}\) shall not be taken less than:

- Undercut anchors…………………………………….. \(2.5h_{ef}\)
- Torque-controlled anchors………………………….. \(4h_{ef}\)
- Displacement-controlled anchors …………………….. \(4h_{ef}\)

D.8.7 Project drawings and project specifications shall specify use of anchors with a minimum edge distance as assumed in design.

D.9—Installation of anchors

D.9.1 Anchors shall be installed in accordance with the project drawings and project specifications and the requirements stipulated by the anchor manufacturer.

D.9.2 The engineer shall establish an inspection program to verify proper installation of the anchors.
D.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.

D.10—Structural plates, shapes, and specialty inserts
D.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 $\phi$ in accordance with D.4.4 or D.4.5.

D.10.2 For structural shapes and fabricated steel sections, the web shall be designed for the shear, and the flanges shall be designed for the tension, compression, and bending.

D.10.3 The nominal strength of specialty inserts shall be based on the 5% fractile of results of tests performed and evaluated according to D.3. Embedment design shall be according to D.3 with strength-reduction factors according to D.4.4 or D.4.5.

D.11—Shear strength of embedded plates and shear lugs
D.11.1 General—The shear strength of grouted or cast-in embeddings 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 forces 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.

D.11.2 Shear toward free edge—For embedded plates and shear lugs bearing toward a free edge, unless reinforcement is provided to develop the required strength, the design shear strength for each lug or plate edge shall be determined based on a uniform tensile stress of $4\sigma_{ct}$ acting on an effective stress area defined by projecting a 45-degree plane from 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 $\phi$ factor shall be taken in accordance with D.4.4 when using load combinations in 9.2 or in accordance with D.4.5 when using load combinations in C.2.

D.11.3 Shear strength of embeddings with embedded base plates—For embeddings having a base plate whose contact surface is below the surface of concrete, shear strength shall be permitted to be calculated 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.

D.12—Grouted embeddings
D.12.1 Grouted embeddings shall meet the applicable requirements of this appendix.

D.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 the contract documents.

D.12.3 Grouted embeddings shall be tested to verify embedment strength. Grouted embeddings 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 licensed professional engineer with full description and details of the testing programs, procedures, results, and conclusions.

D.12.4 Grouted embeddings shall be tested for the installed condition by testing randomly selected grouted embeddings to a minimum of 100% of the required strength. The testing program shall be established by the engineer.

D.12.5 The tests required by D.12.3 and D.12.4 shall be permitted to 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 E—THERMAL CONSIDERATIONS
E.1—Scope
E.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.

E.1.2 The provisions of this appendix apply to concrete structures that are subjected to elevated or differential temperatures and that have restraint such that thermal strains would result in thermal stresses.

E.1.3 The design provisions of this appendix are based on the strength design method.

E.1.4 The assumptions, principles, and requirements specified in 10.2 and 10.3 are applicable for both normal operating and accident conditions.

E.1.5 This appendix does not address temperature requirements during curing, nor does it address temperature and shrinkage reinforcement.

E.2—Definitions
base temperature—the temperature at which a concrete member is cured.

elevated or differential temperature—elevated or differential temperature refer to the temperatures that are different than the construction temperatures, which are assumed to be stress free.

gradient temperature distribution—the temperature distribution minus the mean temperature distribution across a section at a point in time.
local area — region of structure exposed to elevated temperature that may affect concrete performance and stress distribution in the immediate area, without changing the overall component or member behavior (for example, areas around hot piping penetrating transversely through the component).

long term — the time period following stabilization of the temperature distribution across the concrete section.

mean temperature distribution — average temperature distribution across a section at a point in time.

short term — the time period before stabilization of the temperature distribution across the concrete section. Short term for the purposes of E.4.2 is considered up to 30 days.

temperature distribution — the variation of the temperature across a section at a point in time.

thermal strain — Strain produced by thermal expansion or contraction due to thermal loads, including the effects of thermal restraint.

thermal stress — stress produced by restraint of thermal strain.

E.3— General design requirements

E.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.

E.3.2 Time-dependent variations of temperature distributions shall be considered in evaluating thermal strains for both normal operating conditions and accident conditions.

E.3.3 Thermal stress shall be evaluated considering the stiffness characteristics 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 and C.2.1, are considered; and
(d) The coefficient of thermal expansion of concrete may be taken as $5.5 \times 10^{-6}$ per degree Fahrenheit unless other values are substantiated by “tests.”

E.3.4 Thermal force is not allowed to reduce the design forces due to other loads 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 relaxation and creep.

E.4— Concrete temperatures

E.4.1 The following temperature limitations are for normal operation or any other long-term period. The concrete surface 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.

E.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. After exposure to these temperatures, the serviceability of the structure needs to be assessed before resuming the operation of the plant. The extent of the assessment will be determined by the engineer and it may be limited to visual inspection only. For higher temperatures follow provision of E.4.3.

E.4.3 Higher temperatures than those given in E.4.1 and E.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 that verifies that the increased temperatures do not cause deterioration of the concrete either with or without load.

APPENDIX F—SPECIAL PROVISIONS FOR IMPULSIVE AND IMPACTIVE EFFECTS

F.1—Scope

F.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 or C.1 of this Code and in accordance with requirements of F.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 members subjected to these loads.

F.1.2 The provisions of this appendix apply to those structural members directly affected by the impactive and impulsive loads and where failure of the structural elements must be precluded.

F.1.3 Applicable theoretical or experimental evidence may be used to justify requirements less conservative than those of this appendix.

F.1.4 Impactive loads are time-dependent loads due to collision of masses that 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.

F.1.5 Impulsive loads are time-dependent loads that 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.

F.2—Dynamic strength increase

F.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:
**F.3—Deformation**

**F.3.1** Permissible ductility ratio \( \mu_d \) 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 member (refer to Fig. F.3.1). 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 F.3.3 and F.3.4, the maximum deformation shall not result in the loss of intended function of the structural member nor impair the safety-related function of other systems and components.

**F.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 the first fundamental period of the structural member. Consideration shall also be given to the requirements of F.8.

**F.3.3** For beams, walls, and slabs where flexure controls design, the permissible ductility ratio shall either be taken as 0.05/\( \rho \) or 0.1 times the ratio producing balanced strain conditions as defined in F.3.4.

**F.3.4** When flexure controls design, the rotational capacity \( rd_0 \) in radians of any yield hinge shall be limited to 0.0065/(d/e) but shall not exceed 0.07 radians.

**F.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.

**F.3.6** For flexure to control the design, thus allowing the ductility ratios or rotational capacities given in F.3.3, F.3.4 and F.3.5 to be used, the load capacity of a structural member in shear shall be at least 20\% greater than the load capacity in flexure, otherwise, the ductility ratios given in F.3.7 or F.3.9 shall be used.

**F.3.7** For beams, walls, and slabs where shear controls design, the permissible ductility ratio shall be taken as:
(a) For shear resisted by concrete alone, the permissible ductility ratio shall be 1.3;
(b) For shear resisted by concrete and stirrups or bent bars, the permissible ductility ratio shall be 1.6; or
(c) For shear resisted completely by stirrups, the permissible ductility ratio shall be 3.0.

**F.3.8** For beam-columns, walls, and slabs resisting 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\( f_y A_g \) or 1/3 of that which would produce balanced strain conditions, whichever is smaller, the permissible ductility ratio shall be as given in F.3.3 or F.3.4; and
(c) The permissible ductility ratio shall vary linearly from 1.3 to that given in F.3.3 or F.3.4 for conditions between those specified in (a) and (b).

**F.3.9** For axial compressive impulsive or impactive loads, the permissible axial ductility ratio shall be 1.3.

**F.4—Requirements to assure ductility**

**F.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 bars with higher yield strengths or greater cross-sectional areas shall not be permitted.

Reinforcement in structural members 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.

**F.4.2** The maximum value of \( (\rho - \rho') \) shall not exceed 0.5 times the ratio producing balanced strain 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 \( \rho \) of \( 200/f_y \) throughout the entire length of the beam.

**F.4.3** At least 1/3 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 1/3 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 members.

**F.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.

**F.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 1/6 of the clear height of the column.

**F.4.5.1** Where a spiral is used, the volumetric ratio $\rho_s$ shall be not less than indicated by Eq. (10-5), but not less than $0.12f'_c/f_y$.

**F.4.5.2** Where rectangular hoop reinforcement is used, the required area of the bar shall be computed by

$$A_{sh} = \frac{\ell_p \rho_s f_y s_h}{2} \quad (F-1)$$

where $\rho_s$ is the volumetric ratio required by F.4.5.1. 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 $\ell_p$. 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.

**F.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 F.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 F.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 F.2.

**F.6—Impulsive effects**

**F.6.1** Impulsive loads shall be considered in combination with other loads as required by 9.1 or C.1 of this Code and in accordance with F.8 of this appendix.

**F.6.2** When reinforced concrete structural members or structural systems 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 F.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 F.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 members under consideration. Simplified bilinear definitions of stiffness are acceptable.

Maximum predicted response is governed by the ductility criteria in F.3.

**F.7—Impactive effects**

**F.7.1** Design for impactive loads shall satisfy the criteria for both local effects and overall structural response.

**F.7.2** Local impact effects may include penetration, perforation, scabbing, and punching shear.

**F.7.2.1** The penetration depth and required concrete thickness to prevent perforation shall be based on applicable formulas or pertinent test data. When perforation of concrete structural members must be precluded, the concrete thickness shall be at least 20% greater than that required to prevent perforation.

**F.7.2.2** Reinforced concrete structural members protecting required system or equipment that 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.

**F.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 F.5 is not required.

**F.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.

**F.7.3** When reinforced concrete structural members or structural systems are subjected to impactive loads, the structural response may be determined by the methods described in F.6.2.

**F.8—Impactive and impulsive loads**

Impactive and impulsive loads must be considered concurrent with other loads (for example, dead and live load) in determining the required resistance of structural members.
APPENDIX G—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 the convenience of users.

In this tabulation, SI metric units are based on the standards given in IEEE/ASTM SI 10-2002 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

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<td>150</td>
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#### Weight (density)

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Note: For U.S. customary units, density is typically a “weight” density, whereas for SI metric units, density is typically a “mass” density.

#### Length

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* Exact

* Exact
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### Volume

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### SI METRIC EQUIVALENTS OF LIMITING VALUES

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### Section U.S. customary Metric equivalent

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<td>$0.4 + \frac{f_t}{700}$</td>
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<td>$\frac{800 + 0.005f_t}{1100}$</td>
<td>$\frac{1104 + f_t}{1318}$</td>
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<tr>
<td>10.5.1</td>
<td>$A_{n,\min} = \frac{3}{f_t}b_{w,c}d$</td>
<td>$A_{n,\min} = 0.25\frac{f_t}{b_{w,c}d}$</td>
</tr>
<tr>
<td></td>
<td>but not less than $200b_{w,c}df_{t}$</td>
<td>but not less than $1.4b_{w,c}df_{t}$</td>
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<td>10.6.4</td>
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<td>$\left(\frac{f_t\psi_1\psi_2}{21,\text{MPa}}\right)d_b$</td>
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<td>$\left(\frac{3f_t\psi_1\psi_2}{50,\text{ksi}}\right)d_b$</td>
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<tr>
<td>12.2.3</td>
<td>$t_d = \frac{3}{40,\text{ksi}}\left(\frac{\psi_1\psi_2}{c_3 + K_i}\right)d_b$</td>
<td>$t_d = \frac{f_t}{1.1,\text{MPa}}\left(\frac{\psi_1\psi_2}{c_3 + K_i}\right)d_b$</td>
</tr>
<tr>
<td></td>
<td>$\frac{1500,\text{ksi}}{1500,\text{ksi}}$</td>
<td>$\frac{10,\text{ksi}}{10,\text{ksi}}$</td>
</tr>
<tr>
<td>Section</td>
<td>U.S. customary</td>
<td>Metric equivalent</td>
</tr>
<tr>
<td>-------------</td>
<td>----------------</td>
<td>--------------------</td>
</tr>
<tr>
<td>12.3.2</td>
<td>( \frac{0.02 f_y d}{\sqrt{f_y'}} )</td>
<td>( \frac{0.024 f_y d}{\sqrt{f_y'}} )</td>
</tr>
<tr>
<td></td>
<td>((0.0003 f_y d_b))</td>
<td>((0.043 f_y d_b))</td>
</tr>
<tr>
<td>12.5.2</td>
<td>((0.02 \psi_s f_y / \sqrt{f_y'}) d_b)</td>
<td>((0.24 \psi_s f_y / \sqrt{f_y'}) d_b)</td>
</tr>
<tr>
<td>12.7.2</td>
<td>( \frac{(f_y - 35,000)}{f_y'})</td>
<td>( \frac{(f_y - 240)}{f_y'})</td>
</tr>
<tr>
<td>12.8</td>
<td>( \psi d = 0.27 \frac{A_{sl}}{k} \left( \frac{f_y}{\sqrt{f_y'}} \right) )</td>
<td>( \psi d = 3.4 \frac{A_{sl}}{k} \left( \frac{f_y}{\sqrt{f_y'}} \right) )</td>
</tr>
<tr>
<td>12.10.5.2</td>
<td>( 60 b_w \alpha f_{sl} )</td>
<td>( 0.41 b_w \alpha f_{sl} )</td>
</tr>
<tr>
<td>12.13.2.2</td>
<td>( 0.014 d_f d_f' \sqrt{f_y'} )</td>
<td>( 0.17 d_f d_f' \sqrt{f_y'} )</td>
</tr>
<tr>
<td>12.6.1</td>
<td>((0.0005 f_y d_b))</td>
<td>((0.073 f_y d_b))</td>
</tr>
<tr>
<td>17.5.3.1 and 17.5.3.2</td>
<td>( 80 b_w d )</td>
<td>( 0.55 b_w d )</td>
</tr>
<tr>
<td>17.5.3.3</td>
<td>( 500 b_w d )</td>
<td>( 3.5 b_w d )</td>
</tr>
</tbody>
</table>

**SI METRIC CONVERSIONS OF NONHOMOGENEOUS EQUATIONS**

<table>
<thead>
<tr>
<th>Equation</th>
<th>Metric equivalent</th>
</tr>
</thead>
<tbody>
<tr>
<td>(10-5)</td>
<td>( \rho_s = 0.45 \left( \frac{A_{sl}}{A_{sh}} - \frac{V_c}{V_{c,0}} \right) f_y' )</td>
</tr>
<tr>
<td>(11-3)</td>
<td>( V_c = 0.17 \frac{\sqrt{f_y'}}{b_w d} )</td>
</tr>
<tr>
<td>(11-4)</td>
<td>( V_c = 0.17 \left( 1 + 0.073 \frac{N}{A_s} \right) \frac{\sqrt{f_y'}}{b_w d} )</td>
</tr>
<tr>
<td>(11-5)</td>
<td>( V_c = \left( 0.16 \frac{\sqrt{f_y'}}{b_w} + 17.2 \rho_s \frac{V_{d,0}}{M_s} \right) b_w d )</td>
</tr>
<tr>
<td>(11-7)</td>
<td>( V_c = 0.29 \frac{\sqrt{f_y'}}{b_w d} \left( 1 + 0.29 \frac{N}{A_s} \right) )</td>
</tr>
<tr>
<td>(11-8)</td>
<td>( V_c = 0.17 \left( 1 + 0.29 \frac{N}{A_s} \right) \frac{\sqrt{f_y'}}{b_w d} )</td>
</tr>
<tr>
<td>(11-10)</td>
<td>( V_c = 0.05 \frac{\sqrt{f_y'}}{b_w d} + V_{d,0} + V_{d,0} \frac{M_{c,ext}}{M_{c,ext}} )</td>
</tr>
<tr>
<td>(11-11)</td>
<td>( M_{c,ext} = (1/\gamma)(0.5 \frac{\sqrt{f_y'}}{b_w} + f_{m} - f_{d}) )</td>
</tr>
<tr>
<td>(11-12)</td>
<td>( V_{c,0} = (0.29 \frac{\sqrt{f_y'}}{b_w d} + 0.3 f_{m,0}) b_w d + V_{d,0} )</td>
</tr>
<tr>
<td>(11-13)</td>
<td>( A_{v,0} = 0.062 \frac{\sqrt{f_y'}}{b_w d} )</td>
</tr>
<tr>
<td>(11-18)</td>
<td>( \left( \frac{V_{d,0}}{b_w d} \right)^2 + \left( \frac{T_{d,0}}{1.7 A_{sh}} \right)^2 \leq \phi \left( \frac{V_{d,0}}{b_w d} + \frac{2 \sqrt{f_y'}}{3} \right) )</td>
</tr>
<tr>
<td>(11-19)</td>
<td>( \left( \frac{V_{d,0}}{b_w d} \right)^2 + \left( \frac{T_{d,0}}{1.7 A_{sh}} \right)^2 \leq \phi \left( \frac{V_{d,0}}{b_w d} + \frac{2 \sqrt{f_y'}}{3} \right) )</td>
</tr>
<tr>
<td>(11-23)</td>
<td>( (A_y + 2A_s) = 0.062 \frac{\sqrt{f_y'}}{b_w d} )</td>
</tr>
<tr>
<td>(11-24)</td>
<td>( A_{w,0} = \frac{0.42 \frac{\sqrt{f_y'}}{A_{sh}} - \frac{A_s}{k}}{f_y} )</td>
</tr>
</tbody>
</table>

Section 11.6.5.2 | \( (0.35 b_w x)/f_y \)
### SI METRIC CONVERSIONS OF NONHOMGEOUS EQUATIONS (cont.)

<table>
<thead>
<tr>
<th>Equation</th>
<th>Metric equivalent</th>
</tr>
</thead>
<tbody>
<tr>
<td>(11-29)</td>
<td>$V_c = 0.27 \sqrt{f_c'}h_d + \frac{N_p d}{4l_o}$</td>
</tr>
<tr>
<td>(11-30)</td>
<td>$V_c = \left[ 0.05 \sqrt{f_c'} + \frac{\epsilon_o \left( 0.1 \sqrt{f_c'} + 0.2 \frac{N_p}{l_o} \epsilon_o \right)}{M_x V_c} \right] l_o$</td>
</tr>
<tr>
<td>(11-33)</td>
<td>$V_c = 0.083 \left( 2 + \frac{4 \beta}{b_o} \right) \sqrt{f_c'} b_o d$</td>
</tr>
<tr>
<td>(11-34)</td>
<td>$V_c = 0.083 \left( \frac{\alpha d}{b_o} + 2 \right) \sqrt{f_c'} b_o d$</td>
</tr>
<tr>
<td>(11-35)</td>
<td>$V_c = 0.33 \sqrt{f_c'} b_o d$</td>
</tr>
<tr>
<td>(11-36)</td>
<td>$V_c = (\beta_p \sqrt{f_c'} + 0.3 f_{p_c}) b_o d + V_p$</td>
</tr>
<tr>
<td>(12-4)</td>
<td>$\epsilon_o = \left( \frac{f_{c_a}}{2} \right) b_o + \left( \frac{f_{c_a} - f_{c_o}}{7} \right) b_o$</td>
</tr>
<tr>
<td>(18-4)</td>
<td>$f_{p_c} = f_{c_a} + 70 + \frac{f_{c_o}}{100 \rho_p}$</td>
</tr>
<tr>
<td>(18-5)</td>
<td>$f_{p_c} = f_{c_a} + 70 + \frac{f_{c_o}}{300 \rho_p}$</td>
</tr>
</tbody>
</table>

### SI METRIC CONVERSIONS FOR APPENDIX D CODE AND COMMENTARY

<table>
<thead>
<tr>
<th>Section</th>
<th>Metric</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.5.2.2</td>
<td>$k_c = 10$ for cast-in anchors; and $k_c = 7$ for post-installed anchors. The $k_c$ factor for post-installed anchors shall be permitted to be increased above $k_c = 7$ based on D.3.3 product-specific tests, but shall in no case exceed 10.</td>
</tr>
<tr>
<td>Eq. (D-8)</td>
<td>$N_b = 3.9 \sqrt{f_c'} h_d^{0.53}$</td>
</tr>
<tr>
<td>D.5.4.1</td>
<td>$N_{sb} = 13 c_{sl} \sqrt{f_{c_{sl}}} (D-16)$</td>
</tr>
<tr>
<td>D.6.2.2</td>
<td>$V_b = 0.6 \left( \frac{f_{c'}}{d_h} \right)^{0.2} \sqrt{f_{c_{sl}}} (c_{sl})^{1.5}$ (D-24)</td>
</tr>
<tr>
<td>D.6.2.3</td>
<td>$V_b = 0.7 \left( \frac{f_{c'}}{d_h} \right)^{0.2} \sqrt{f_{c_{sl}}} (c_{sl})^{1.5}$ (D-25)</td>
</tr>
</tbody>
</table>
COMMENTARY

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 349-06)” hereinafter called the Code. The Code is based on “Building Code Requirements for Reinforced Concrete (ACI 318-05),” which is hereinafter referred to as the Building Code. This commentary discusses provisions in the Code that differ from the Building Code. In preparing ACI 349-06, 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 (ACI 318R) are to the 2005 revision unless specifically noted otherwise. Provisions of ACI 318R-05 apply, except that the term “building official” shall be replaced with the term “engineer.”

CHAPTER R1—GENERAL REQUIREMENTS

The commentary on ACI 318-05 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 349-06),” 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 provides requirements 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. Some special structures involve unique problems that are not covered by the Code, such as structures that function as leakage barriers to contain the effects of the loss of coolant accident. The owner is to identify nuclear safety-related structures and establish which of them are covered by “Code for Concrete Containments (ACI 359)” and its latest revisions instead of this Code. The Code is applicable to radioactive waste repository structures; however, considerations of thermal loads, load combinations, and long-term durability should be considered.

R1.1.1 In general, the Code formulations are based on test results from concrete specimens having a compressive strength of 6000 psi or less. Although no maximum concrete compressive strength is specified, the applicability of various requirements and formulations should be verified when concrete compressive strengths are (significantly) higher than 6000 psi.

R1.1.6 The provisions of this Code apply to slabs-on-ground. In addition to the requirements of this Code, the designer should consider other issues, as outlined in ACI 360R-13 and PTI’s Design of Post-Tensioned Slabs-on-Ground.14

R1.2—Drawings and specifications

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. 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 professional engineer competent in the field of design of concrete structures, who is required to sign these documents signifying his or her approval. This Code requires that the owner be responsible for drawings and calculations, but does not preclude him or her from assigning the function of detailed implementation to others.

R1.3—Inspection

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 ACI certified as applicable, and qualified by the owner. ANSI/ASME NQA-1 or ACI 359 Appendix VII may be used to qualify inspectors. The inspectors should be thoroughly familiar with the applicable ACI and ASTM standards (for example, ACI 311.4R). The inspection agency should be accredited to ASTM E 329. The concrete and aggregate testing laboratory should be accredited to ASTM C 1077. If the owner(s) directly employ inspection personnel, the owner shall be permitted to follow ASTM E 329 as a quality enhancement.

Requirements for the retention of inspection records should follow ANSI/ASME NQA-1.

R1.4—Approval of special systems of design or construction

New methods of design, new materials, and new uses of materials should undergo a period of development before being specifically covered in a code. Hence, potentially suitable 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 United States.

Some special concrete structures also need to be evaluated for their effectiveness as radiation shields. Specific guidance for this purpose may be obtained from ANSI/ANS-6.4-1997.

R1.5—Quality assurance program

Title 10 of the Code of Federal Regulation, Part 50, Appendix B, and Title 10 of the Code of Federal Regulations, Part 830, Subpart A, 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.\textsuperscript{1,5} Additional guidance is contained in ACI 311.4R\textsuperscript{1,6} and ACI 121R.\textsuperscript{1,8}

References

1.1. ACI Committee 349, “Evaluation of Existing Nuclear Safety-Related Concrete Structures (ACI 349.3R-02),” American Concrete Institute, Farmington Hills, Mich., 2002, 18 pp.


CHAPTER R2—NOTATION AND DEFINITIONS

The notations and definitions in this Code are the same as those in the Building Code (ACI 318-05) except for a few that are added or modified to meet the structures and materials concerned in this Code.

R2.1—Commentary notation

The terms used in this list are used in the commentary, but not in the Code.

Units of measurement are given in the notation to assist the user and are not intended to preclude the use of other correctly applied units for the same symbol, such as feet or kips.

The notations that differ from or are not listed in ACI 318-05 are:

- \( c'_{\text{el}} \) = limiting value of \( c_{\text{el}} \) when anchors are located less than 1.5\( h_{\text{ef}} \) from three or more edges (see Fig. RD.6.2.4), Appendix D
- \( c_{\text{v}} \) = effective side dimension of the loaded zone, Appendix F
- \( D_{\text{h}} \) = effective side dimension of the plastic hinge zone, Appendix F
- \( e_{\text{N}} \) = actual eccentricity of a normal force on an attachment, in., (see Fig. RD.5.2.4), Appendix D
- \( h'_{\text{ef}} \) = limiting value of \( h_{\text{ef}} \) when anchors are located less than 1.5\( h_{\text{ef}} \) from three or more edges (see Fig. RD.5.2.3), Appendix D
- \( K_{\text{05}} \) = coefficient associated with the 5% fractile, Appendix D
- \( K_{\text{C}} \) = confinement factor (RD.11), Appendix D
- \( L_{\text{w}} \) = distance between local loads, Appendix F
- \( N_{\text{y}} \) = yield strength of tension anchors equal to \( nA_{\text{s}}f_{\text{y}} \), Appendix D
- \( P_{\text{v}} \) = punching shear capacity, Appendix F
- \( q_{\text{b}} \) = tensile reinforcement index for balanced, ultimate strength conditions, Appendix F
- \( r_{\text{u}} \) = rotational capacity of a yield hinge, Appendix F
- \( z \) = distance from the point of maximum moment to zero moment, in., Appendix F
- \( \varepsilon_{\text{u}} \) = ultimate compressive strain of concrete, Appendix F
- \( \psi_{\text{u}} \) = ultimate curvature, Appendix F

R2.2—Definitions

The definitions that differ from or are not listed in ACI 318-05 are:

- authority having jurisdiction (AHJ);
- creep;
- design basis tornado (DBT);
- embedment;
- engineer;
- evaluation;
- load, dead;
- load, live;
- load, sustained;
- massive concrete;
- moment frame;
- operating basis earthquake (OBE);
- operating basis wind;
- owner;
- safe shutdown earthquake (SSE);
- shrinkage;
- stress relaxation; and
- structural walls.

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 SEI/ASCE 7 for ACI 349 applications should be warned of the differences, such as the need to recreate Table 6-2 in SEI/ASCE 7 and modify equations listed in 6.5.12 and 6.5.13 for 100-year recurrence using Table C6-3 in SEI/ASCE 7 and its commentary.

CHAPTER R3—MATERIALS

The commentary on ACI 318-05 is applicable to this chapter except as described as follows.

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 by the owner 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. In addition, the designer 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 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.

R3.3.3.1 Anytime there is a basic change in aggregate source and periodically during production, 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 359-01.

R3.5—Steel reinforcement

R3.5.3 Deformed reinforcement—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 et al. 3.1 concluded that zinc coatings provide little value in providing long-term protection of reinforcing steel, and cautionary statements in ACI 201.2R 3.2 support this position. These industry concerns have prompted ACI Committee 349 to prohibit the 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 not permitted because of inadequate traceability.

R3.5.3.2, R3.5.3.3, R3.5.3.4, R3.5.3.5, R3.5.3.6, R3.5.3.7, and R3.5.4.2—To limit the widths 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 R9.4. (For reference to R9.4, see ACI 318-05.)

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 before 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 (CMTRs) provided for concrete material, reinforcing steel system material, prestressing system material, and welding and brazing material, should include the following:

(a) Certified reports of the actual results and specifications 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. The name and qualifications (accreditation) of the agency actually performing the tests should be indicated on the report.

(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 3.3

R3.7.4 To prevent detrimental corrosion, prestressing systems should normally be stored in a completely enclosed building.

R3.8.6 The Commentary in Appendix D, RD.3.3, references ACI 355.2 as an acceptable qualification test standard for post-installed anchors.
CHAPTER R4—DURABILITY REQUIREMENTS

The commentary on ACI 318-05 is applicable to this chapter. Certain sections of the commentary in ACI 318R, 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 material 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 ACI 211.1.4.1 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 percentage point lower air content for concrete with 5000 psi. Such high-strength concretes will have lower water-cementitious material ratios and porosity and, therefore, improved frost resistance. References to lightweight aggregates and lightweight concrete have been omitted from this code and commentary discussions.

R4.2.2 For normal weight aggregate concrete, the use of both minimum strength and maximum water-cementitious material ratio provide additional assurance that a high-quality cement paste will be obtained.

Table 4.2.3 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 material ratios for concrete exposed to deicing chemicals.4.2-4.4 Recent research has demonstrated that the use of fly ash and silica fume produce concrete with a finer pore structure and, therefore, lower permeability.4.5-4.7 For concretes exposed to deicing salts, the inclusion of maximum percentages of fly ash in the calculation of water-cementitious material 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 318-05 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 618,4.8 or have been tested by ASTM 10124.9 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 2014.10 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 referenced4.12 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 ACI 201.2R. Some of the total chloride ions present in the ingredients will either be insoluble or will react with the cement during hydration and become insoluble under the test procedures described in ASTM C 1218.

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, 201.2R, and 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 material ratio and specified compressive strength of 4.2.2 are the minimum requirements that are to be considered. The designer should evaluate conditions in structures where chlorides may be applied or in structures near seawater. Epoxy-coated bars or concrete cover greater than the minimum required in 7.7 may be desirable. Use of slag meeting ASTM C 989, 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-

References


3.2. ACI Committee 201, “Guide to Durable Concrete (ACI 201.2R-01),” American Concrete Institute, Farmington Hills, Mich., 2001, 41 pp.


3.4. ACI Committee 201, “Guide for Durable Concrete” reported by ACI Committee 2014.10 and “Corrosion of Metals in Concrete” reported by ACI Committee 222.4.11

ASTM C 1218.
range water reducer (ASTM C 494, Types F and G, or ASTM C 1017), can also provide additional protection. The use of ASTM C 1202 to test concrete mixtures can also provide additional protection. The use of ASTM C 1202 to test concrete mixtures proposed for use will provide additional information on the performance of the mixtures.

References


CHAPTER R5—CONCRETE QUALITY, MIXING, AND PLACING

The commentary on ACI 318-05 is applicable to this chapter except as described as follows. Certain sections of the Commentary in ACI 318R, Chapter 5, have been repeated in this commentary chapter to maintain correspondence with footnote numbers in the reference section.

The splitting tensile strength requirements have been eliminated because they only apply to lightweight aggregates, which also have been eliminated.

R5.1—General

R5.1.3 Concrete members 24 in. or thicker 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 318-05, which has been omitted from discussion in this Code and commentary.

R5.1.5 See R5.2.

R5.3—Proportioning on the basis of field experience or trial mixtures, or both

The engineer should verify that test records are applicable to the proposed materials and mixture designs.

R5.4—Proportioning without field experience or trial mixtures

Figure R5.4 is a flowchart outlining the mixture selection and documentation procedure.

R5.4.1 (Previous commentary provision has been deleted since it was applicable to ACI 318-89.)

R5.6—Evaluation and acceptance of concrete

R5.6.2 Frequency of testing—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 ensure 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
Concrete production facility has field strength test records for the specified class or within 1000 psi of the specified class of concrete

- Yes
  - ≥ 30 consecutive tests
    - No
      - Calculate $s$<sub>5</sub>
    - Yes
      - Calculate average $s$<sub>5</sub>
  - No (No data for $s$<sub>5</sub>)
  - No
    - Two groups of consecutive tests (total ≥30)
    - No
      - Calculate $s$<sub>5</sub> and increase using Table 5.3.1.2
    - Yes
      - Required average strength from Table 5.3.2.2

- No
  - 15 to 29 consecutive tests
    - Required average strength using Table 5.3.2.1
    - Field record of at least ten consecutive test results using similar materials and under similar conditions is available
      - No
        - Make trial mixtures using at least three different water-cementitious materials ratios or cementitious materials contents according to 5.3.3.2
      - Yes
        - Results represent one mixture
          - No
            - Results represent two or more mixtures
              - Yes
                - Average ≥ required average
                  - No
                    - Plot average strength versus proportions and interpolate for required average strength
                  - Yes
                    - Submit for approval
        - No
          - Plot average strength versus proportions and interpolate for required average strength
          - Determine mixture proportions according to 5.4 (requires special permission)

Fig. R5.4—Flow chart for selection and documentation of concrete proportions.
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.

**R5.6.4 Field-cured specimens**

Field-cured cylinders do not represent the in-place strength of typical concrete members used in nuclear plant construction, particularly during the first 24 hours 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 properly correlated against laboratory-cured cylinders during the concrete design phase, 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.

**R5.6.5 Investigation of low-strength test results**—Instructions are provided concerning the procedure to be followed when strength tests have failed to meet the specified acceptance criteria. For obvious reasons, these instructions cannot be dogmatic. The engineer should apply judgment as to the significance of low test results and whether they indicate need for concern. If further investigation is deemed necessary, such investigation may include nondestructive tests, or in extreme cases, strength tests of cores taken from the structure.

Nondestructive tests of the concrete in place, such as by probe penetration, impact hammer, ultrasonic pulse velocity or pullout may be useful in determining whether 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. See ASTM for additional cautions and clarifications of these tests. For cores, if required, conservatively safe acceptance criteria are provided that should ensure structural adequacy for virtually any type of construction. 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). Short of load tests, if time and conditions permit, an effort may be made to improve the strength of the concrete in place by supplemental wet curing. Effectiveness of such a treatment should be verified by further strength evaluation using procedures previously discussed.

A core obtained through the use of a water-cooled bit results in a moisture gradient between the exterior and interior of the core being created during drilling. This adversely affects the core’s compressive strength. The restriction on the commencement of core testing provides a minimum time for the moisture gradient to dissipate.

Core tests having an average of 85% of the specified strength are 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 Section 5.7.1(f) requires free water deeper than 1/4 in. in hollows be removed from the forms. It is not necessary that all water be removed. Code Section 5.7.1(g): 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.

**R5.8—Conveying**

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.

**R5.10—Depositing**

The wording has been changed from ACI 318-05 to be more specific and to exclude the use of retempered concrete.

**R5.11—Curing**

In many areas, protective coatings are required, and no curing procedure should be used that may be incompatible with those coatings.

**R5.12—Cold weather requirements**

Recommendations for cold weather concreting are given in detail in “Cold Weather Concreting” reported by ACI Committee 306. (presents 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,”5.9 for additional information on crack control.

References

CHAPTER R6—FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION JOINTS
The commentary on ACI 318R-05 is applicable to this chapter except as described as follows.

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 R6.2.3—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-05 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 piping 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.

R6.4—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 R7—DETAILS OF REINFORCEMENT
The commentary on ACI 318R-05 is applicable to this chapter except as described as follows.

R7.4—Surface conditions of reinforcement
R7.4.1 For an explanation of the service conditions (that is, 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 it is 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 $f_y''$ 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 $0.60f_y$ was selected for this purpose.

R7.12.4 The application of massive concrete is common in design and construction of concrete nuclear safety-related structures. The requirements for minimum reinforcement and physical limitation of massive concrete, however, have not been clear to the designers. ACI Committee 207 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


CHAPTER R8—ANALYSIS AND DESIGN—GENERAL CONSIDERATIONS

Chapter 8 parallels the general considerations for analysis and design presented in Chapter 8 of ACI 318-05. Some modifications have been made that reflect particular requirements applicable to concrete nuclear safety-related structures. Reference 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 318-05 should be referenced for concrete nuclear safety-related structures except as noted herein.

R8.2—Loading

The commentary on ACI 318-05 is not applicable for concrete nuclear safety-related structures. Commentary regarding design loads and load combinations is presented in Chapter 9 of this document.

R8.3—Methods of analysis

R8.3.3(c) 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 318-05 has been deleted.

R8.11—Joist construction

Reference to permanent burned clay or concrete tile fillers in ACI 318-05 has been deleted.

CHAPTER R9—STRENGTH AND SERVICEABILITY REQUIREMENTS

Chapter 9 parallels the requirements for strength and serviceability presented in Chapter 9 of ACI 318-05. 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-related 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 318-05 do not appear in this standard. Chapter 9 of the commentary on ACI 318-05 should be referenced for concrete nuclear safety-related structures except as noted herein.

In ACI 349, some notations have been added, and some notations have been modified. The notations added as a result of additional notations in ACI 318-05 are explained in its commentary. The ACI 349 changes are explained as follows:

Crane load rated capacity ($C_{cr}$) is added as a separate load. This addition will help treat the crane hook load according to its variability and its probability of occurrence. The crane component dead loads have to be considered in dead load ($D$) estimation. The live load ($L$) definition has been accordingly modified.

The definition of temperature load ($T_w$) has been modified to include other temperature-induced loads. These loads could be caused by the restraint of concrete member on the attached metallic components, or by the prolonged cyclic temperature changes in the concrete members. In the first scenario, the thermal loads will depend on the restraint of the concrete member. In the second scenario, the induced thermal loads will continuously reduce as the deformation of the concrete members increases in response to the thermal load. Eventually, the concrete members will arrive at a deformed state when the internal forces will equilibrate the thermal load. In general, this load will not necessitate additional reinforcement, but will cause additional deformation. This additional deformation may require further investigation.
The provisions of Appendix E should be useful in investigating these loads.

To be consistent with the expanded definition in Chapter 2, $E_o$ and $E_{ss}$ definitions have been modified.

In the notation for $D$, $E_o$, $E_{ss}$, $R_o$, and $R_p$, “equipment” is to include all items that are permanently attached to the concrete structure, such as cable trays and conduits.

R9.1—General

In ACI 349-06, the load factor combinations and strength-reduction factors of the 349-01 Code are moved to Appendix C. Even though Section 9.1 of ACI 318-05 has been completely revised, the commentary on ACI 318-05 for this section is generally applicable. In ACI 349, the loads appropriate for nuclear safety-related structures are defined in Section 2.1 and categorized in Section 9.1 as: 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. In ACI 349, $R_o$ is considered to have lower variability in its estimation than that for the live load. This is based on the existing practice of estimating $T_o$ as the maximum possible operating temperature, and conservatively estimated flow transients. However, if it is recognized that there could be a large uncertainty in estimating these parameters, the designer should consider the use of larger load factors for $T_o$ and $R_o$. Note that dead load and earthquake reactions are not included in $R_p$.

When considering impactive and impulsive 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, and machinery, should be increased to allow for impact in accordance with the recommendations of SEI/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.

R9.1.1.1 Normal loads—Dead and live loads form a generic category of normal loads. During initial design, most of the piping loads and suspended system loads (HVAC and cable trays) are not available, and the load allowance for these items is included in $L$ as an area-averaged load. Once the final attachment loads are determined, the initial load assumptions should be confirmed. When designing for weights or pressures from fluids, either existing in the structure or due to hydrostatic heads, both cases (with fluid present or absent) must be evaluated to establish the governing load condition. When a detailed dynamic analysis is performed for crane systems, elevators, or other moving machinery, the resulting load with dynamic amplification may be used instead of the load increases (dynamic impact factors) specified in SEI/ASCE 7.

R9.1.1.2 Severe environmental loads—These loads include loads generated by specified wind or earthquake. For the safety-related concrete structures in nuclear facilities, wind load $W$, if not specified by the plant-specific documents, may be defined as the wind load computed in accordance with Section 6 of SEI/ASCE 7, considering them as Category IV structures. The operating basis earthquake load $E_o$ should be in accordance with the requirements set by the authority having jurisdiction (AHJ). In the case of nuclear power plants, $E_o$ should be in accordance with Appendix S of 10CFR50.

R9.1.1.3 Extreme environmental loads—These loads include loads generated by design basis tornado $W_t$ and Safe Shutdown Earthquake $E_{ss}$. For nuclear facilities, they are defined by the government authorities having jurisdiction at the site (for example, Nuclear Regulatory Commission). For older nuclear power plants and Department of Energy facilities, $E_{ss}$ is defined as design basis earthquake (DBE). These are low-probability environmental loads. These loads are explicitly defined in the documents controlling the facility design. For example, $W_t$ for nuclear power plants could have recurrence period as low as $10^{-7}$, while the recurrence period for $E_{ss}$ could vary from $10^{-3}$ to $10^{-6}$.

R9.1.1.4 Abnormal loads—In nuclear power plants, these loads are from a design-basis accident (DBA) and are postulated to result from:

(a) A break in any of the high-energy piping existing in the plant. This can create compartment pressurization, short-term high temperatures, and dynamic loads of reaction and/or impingement associated with the postulated pipe rupture.

(b) A break in a small line containing high-temperature fluids or steam. This would result in a long-term high temperature and associated pressure loading.

(c) Other extreme load phenomena, which have a probability of occurrence larger than $10^{-7}$ events per year, the consequence of which could lead to release of radiation in excess of 10CFR100 limits.

In addition to the pressure $P_a$, temperature $T_a$, and reaction loads $R_o$ associated with a DBA, the loads could include the consequences of a rupture of a high energy pipe, that would give rise to loads $Y_r$, $Y_j$, and $Y_m$. Other nuclear facilities could have other abnormal loads depending upon the type of accidents postulated for those facilities.

R9.2—Required strength

Section 9.2 of ACI 318-05 and its commentary have been completely revised. The basic concept of ACI 318-05 has been adopted for ACI 349-06 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.
The basic criterion used in arriving at the Load Combinations (9-1) to (9-5) (normal and severe environmental loads) follows the same logic as in ACI 318-05, which is based on SEI/ASCE 7, Section 2.3. For consideration of extreme environmental and abnormal loads, the criterion is based on the philosophy developed in References 9.3 to 9.5 and, to some extent, on Reference 9.6. The overall approach used, however, is the same as that explained in the commentary of Section 2.5 of SEI/ASCE 7. Load Combinations (9-1) to (9-5) involve normal loads and normal loads in combination with severe environmental loads. Similar to ACI 318, this standard uses load factors of 1.4, 1.2, 1.6 and fractional load factors (that is, 0.5) for roof loads, and loads due to snow and rain. Load factors for loads \( R_p, T_p, W, \) and \( E_p \) are reduced significantly, to be compatible with ACI 318-05, compared to those in ACI 349-01. This can be justified by the cumulative approach used by SEI/ASCE 7 in defining the load combinations. However, the load factor on \( L \) has been kept as 0.8, compared to 0.5 in ACI 318-02. This is based on the consensus estimate made in Reference 9.6, where the mean value of \( L \) is estimated as 0.8L. The operating temperature \( T_p \) induced loads have been treated consistent with the earlier editions of ACI 349.

While considering the Load Combinations (9-4), (9-6), and (9-9), for the local design of floors and beams, a fraction of the live load \( L \) or roof load \( L_r \), which is most likely to be existing on the members, should be considered as an input into seismic analysis, instead of the factors associated with \( L \) and \( L_r \). A common practice in the design of concrete structures in nuclear power plants would be to use 25% of \( L \), or 75% of \( L_r \), for the purpose. The gravity load effects of these loads should be in accordance with the factors established in the load combinations. For the global seismic analysis, \( L \) and \( L_r \) should be based on the mean value associated with these loads, that is, \( 0.8L \) or \( 0.5L_r\), as applicable.

Unit load factors are used in Load Combinations (9-6) and (9-7) involving extreme environmental loads, except that a factor of 0.8 is used for live load as it is a companion load in these load combinations. This recognizes that the loads caused by the safe shutdown earthquake or the DBT are extreme and are of low probability. Load Combination (9-8) is associated with the basic loads generated as a result of a postulated accident, and Load Combination (9-9) is associated with the combined effects of a postulated accident and \( E_{ss} \). The following guidance will be useful in designing for such low-probability events.

Dynamic load effects should be considered with maximum values assumed acting simultaneously, unless actual time history analysis shows a different time-phase relationship, in which case loads may be combined as a function of time. Loads due to postulated accidents and natural phenomena often yield dynamic response of short duration and rapidly varying amplitude in the exposed structures and components. For some loading phenomena, the accident analysis provides a definitive time-history response and allows a straightforward addition of responses where more than one load is acting concurrently. In other cases, no specified time-phase relationship exists, either because the loads are random in nature or because the loads have simply been postulated to occur together (for example, LOCA and SSE) without a known or defined coupling. Where a defined time-phase relationship is lacking, system designers have used several approaches to account for the potential interaction of the loads. One approach, the so-called absolute or linear summation (ABS) method, linearly adds the absolute values of the peak structural response due to the individual dynamic loads. A second approach, referred to as the square root of the sum of the squares (SRSS) method, results in a combined response equal to the square root of the sum of the squares of the peak responses due to the individual dynamic loads. Research conducted over the past two decades shows that this method of combining dynamic responses is conservative unless the structural responses are stochastically dependent. The SRSS method of load combination is acceptable to the U.S. Nuclear Regulatory Commission, contingent upon the performance of a linear elastic dynamic analysis. Thus, the loads from a loss of coolant accident (LOCA) and a seismic event combined in Load Combination (9-9) may be combined by the SRSS method, provided that the responses are determined by elastic analysis. However, this does not prohibit the use of more conservative load combination schemes. In all cases, resultant dynamic loads should be combined absolutely, considering both maximum positive and negative values, with applicable static 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 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 extreme environmental loads.

R9.2.9 The Code recognizes the low likelihood event of a SSE (DBE) occurring while the crane is in use. A probability analysis should be performed to demonstrate that this combination is incredible. A probability of 10E-06 or smaller is a reasonable measure to demonstrate incredibility.

R9.2.10 Load factors and strength-reduction factors were revised in ACI 318-05 and for the normal load conditions in ACI 349 in a manner that resulted in an equivalent structural design. The reduction in strength-reduction factor for shear would require a reduction in the load factor from 1.0 to 0.9 to give comparable requirements for the design for earthquake loads. Instead of reducing the load factors below 1.0, Section 9.2.10 permits a reduction of 10%, if the facility specific \( E_{ss} \) is the minimum required by the AHJ. For all new nuclear power plant applications, the NRC has established \( E_{ss} \) as having the probability of exceedance (POE) lower than 1.0E-5/year (median). For Performance Category 4 structures in DOE nuclear facilities, the POE has been established as approximately 1.0E-4/year (mean). If the POE of a nuclear facility structure is higher than 1.0E-4/year, this reduction should not be allowed. Thus, for most nuclear
facility structures of importance, this provision would result in an equivalent design to that required by ACI 349-01.

**R9.3—Design strength**

Design strength of a member refers to the nominal strength to be calculated in accordance with the requirements of ACI 318-05. The basic concept used in the ACI 318, SEI/ASCE 7, and the revised load combinations in Section 9.2 is similar, in the sense that the load factors are assigned with the realistic assessment of the overload conditions in Load Combinations (9-1) to (9-5). Consideration of extreme loads is as required by the AHJ, and proper guidance for combining the abnormal loads in concert with the extreme environmental loads is provided. Thus, the strength-reduction factors of ACI 318-05 are applicable in determining the design strength of the members.

In Section 9.3.4 of ACI-318-05, $\phi = 0.60$ is used for shear-critical members, reflecting the lower ductility of shear-critical members than flexure-critical members. Under cyclic inelastic loading, shear-critical members such as squat shear walls exhibit hysteresis that is characterized by degradation of stiffness and strength, and pinching. Although structural components in safety-related nuclear structures are detailed typically for inelastic response per conventional building construction, the inelastic deformation demands on components in nuclear structures are not as high as those in conventional special building structures. As such, loss of strength and stiffness due to cyclic inelastic loading in components of nuclear structures will be smaller than those in conventional building structures, and $\phi = 0.75$ (as in 9.3.2.3) is considered to be adequate for the conditions delineated in Section 9.3.4 of ACI 318-05. Thus, Section 9.3.4 of ACI 318-05 is not incorporated in ACI 349-06.

**R9.4—Design strength for reinforcement**

As the structures under ACI 349’s scope are required to withstand earthquake-induced forces, the strength of transverse reinforcement is limited to 60,000 psi (see Section 21.2.5 and its commentary).

**R9.5—Control of deflections**

This section contains different deflection requirements from ACI 318-05. The deflection requirements in ACI 318-05 are based on comfort levels for human occupancy, cracks in ceilings, 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 $\ell / d$ 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 members.

Table 9.5(a) provides the deflection limits under unfactored loads.

Consistent with ACI 349 loading combinations, the table indicates anticipated deflection values for normal, severe environmental, and extreme environmental 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 9.5(a).

For normal and severe environmental loading conditions, the anticipated deflections are selected to be $\ell / 400$ for beams and ribbed one-way slabs and $\ell / 320$ 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 members. A more liberal or a more stringent criteria for the anticipated deflection can be chosen, however, the $\ell / d$ ratio has to be modified accordingly. The deflection limits in the Code will provide a reasonable $\ell / d$ ratio and at the same time will not require deflection calculations for a large number of structural members.

The $\ell / d$ ratios given in the Code were established from:

1. Recommendations by ACI 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 $\ell / d$ 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 installed at least a year after the structure has been built. Therefore, part of the long-term deflection due to structural dead load has already occurred at the time the equipment is installed.

The Code defines the deflections that need to be considered. Immediate deflection is no longer a consideration. Because the minimum $\ell / d$ ratios are more stringent than ACI 318-05, the immediate deflection limits of ACI 318-05 will be satisfied.

The considerations for camber in prestressed concrete members are included in the Code.

A deflection requirement for walls is also included. Because the walls in ACI 349 structures frequently resist 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 $\gamma$ and moment of inertia modification factor $I/I_e$ 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.
due to wind, tornado, or equipment). The concrete shear strength for this condition is given in Section 11.12.2.3, and the expressions for $V_{s1}$ and $V_{s2}$ are based on punching shear tests of biaxially tensioned wall-like elements, reported in References 11.1 and 11.2. These tests indicated that biaxial tension levels up to nearly $0.9f_y$ produced only minor decreases in the punching shear strength for members without any shear reinforcement. For membrane tension stress ratios of $0.9f_y$ or $0.6f_y$ exceeding 0.9, the corresponding components of shear strength are taken as $0.5f_yb_1h_1$ and $0.5f_yb_2h_2$, 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 and 11.12.2.3.4 apply when the membrane tensile stresses $f_{m1}$ or $f_{m2}$ exceed $0.6\rho_1f_y$ or $0.6\rho_2f_y$, respectively. The membrane reinforcement is required to be increased to resist the punching shear force $V_h$ 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


R12.6—Mechanical anchorage
The provisions of Sections 12.6.2 and 12.6.3 of ACI 318-05 have been replaced in ACI 349 by a reference to Appendix D 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 Section 12.6.3 of ACI 318-05, should only be used if substantiated by experimental or detailed analytical investigation.

R12.14—Splices of reinforcement—general
R12.14.3.2 The strength requirement for mechanical splices has been revised to that for a Type 2 splice in 21.2.6.1 of ACI 318-05. Such splices are intended to avoid a splice failure when the reinforcement is subjected to expected stress levels in the yielding region. ACI 349 structures are detailed for ductile response in the event of an earthquake larger than the SSE. Consistent with this philosophy, welded splices are also required to develop the full tension strength of the bar.

R12.14.3.5 All splices are required to be qualified to 12.14.3.2 and 12.14.3.4 to assure ductile behavior. See also R12.14.3.2.

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.

R12.15—Splices of deformed bars and deformed wire in tension
R12.15.3 All splices are required to be qualified to 12.14.3.2 and 12.14.3.4 to assure ductile behavior. See also R12.14.3.2.

CHAPTER R13—TWO-WAY SLAB SYSTEMS
The commentary of ACI 318-05 is applicable to this chapter.

CHAPTER R14—WALLS
The commentary of ACI 318-05 is applicable to this chapter except as noted as follows.

R14.3—Minimum reinforcement
The minimum reinforcement requirements of Section 7.12 are more applicable for the thick concrete nuclear safety structures than the corresponding requirements of ACI 318-05. See Section R7.12 for discussion.

CHAPTER R15—FOOTINGS
The commentary of ACI 318-05 is applicable to this chapter.

CHAPTER R16—PRECAST CONCRETE
The commentary of ACI 318-05 is applicable to this chapter.

CHAPTER R17—COMPOSITE CONCRETE FLEXURAL MEMBERS
The commentary of ACI 318-05 is applicable to this chapter.

CHAPTER R18—PRESTRESSED CONCRETE
The commentary of ACI 318-05 is applicable to this chapter except that references to lightweight concrete are to be deleted, and Sections 18.14 and 18.15 are deleted because those provisions are typically not applicable to nuclear facilities.

Section 18.14 is added to define sustained load conditions to include both normal mechanical and severe environmental loads.

CHAPTER R19—SHELLS
Chapter 19 of ACI 318-05 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 applicable to this type of shell structure. The commentary to Chapter 19 of ACI 318-05 is applicable to the ACI 349 Code, supplemented by the discussion that follows.

R19.1—Scope
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, R19.2.2, R19.2.3—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 upon. 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 1/10 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-related structures than the corresponding requirements of ACI 318. See Commentary Section R7.12 for discussion.

R19.4.6 Stirrups or other types of web reinforcement may be used, if required, to tie back curved reinforcement and transfer 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 used 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 318-05 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 and structural members, 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 approximately 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 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 members, the amount, size, arrangement, and location should be determined at the critical sections for reinforcement or tendons, or both, designed to resist applied load. Nondestructive investigation methods are acceptable. In large structures, determination of these data for approximately 5% of the reinforcement or tendons in critical regions may suffice if these measurements confirm the data provided in the construction drawings.

R20.2.3 The number of tests may depend on the size of the structure and the sensitivity of structural safety to concrete strength. In cases where the potential problem involves flexure only, investigation of concrete strength can be minimal for a lightly reinforced section ($f_y f_c' / \phi \leq 0.15$ for rectangular section).

R20.2.4 The number of tests required depends on the uniformity of the material and is best determined by the engineer for the specific application.

R20.2.5 Strength-reduction factors used in the strength evaluation should be established on the basis of accurate derivation of material properties, actual in-place dimensions, and physical condition of the structure, and may be larger than those specified in Chapter 9 if justified. Justification should be based on comparison of governing design conditions and load combinations for the structure, compared to the computation of the load intensity selected for the test. Additional information and criteria that may be used in the analytical investigation of a structure that is contained in ACI 349.3R. 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 ACI 437R.
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 members will help resist some of the load, the load should 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 questions. The increase in live load is determined from analysis of the loading conditions in relation to the selected pass/fail criterion for the test. Because the design basis and load combinations used for nuclear safety-related structures often involve extreme environmental loads, the engineer should carefully select a load intensity for use in the test that is representative of such.

R20.5—Loading criteria

R20.5.2 Inspecting the structure after each load increment is advisable.

R20.5.3 Arching refers to the tendency for the load to be transmitted nonuniformly to the flexural member 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 does not show evidence of failure. Evidence of failure includes cracking, spalling, or deflection of such magnitude and extent that the observed result is obviously excessive and incompatible with the safety requirements of the structure. No simple rules have been developed for application to all types of structures and conditions. If sufficient damage has occurred so that the structure is considered to have failed that test, restesting is not permitted because it is considered that damaged members should not be put into service even at a lower load rating.

Local spalling or flaking of the compressed concrete in flexural members related to casting imperfections need not indicate overall structural distress. Crack widths are good indicators of the state of the structure and should be observed to help determine whether the structure is satisfactory. However, exact prediction or measurement of crack widths in reinforced concrete members is not likely to be achieved under field conditions. Establish criteria before the test, relative to the types of cracks anticipated; where the cracks will be measured; how they will be measured; and approximate limits or criteria to evaluate new cracks or limits for the changes in crack width.

R20.6.2 The deflection limits and the retest option follow past practice. If the structure shows no evidence of failure, recovery of deflection after removal of the test load is used to determine whether the strength of the structure is satisfactory. In the case of a very stiff structure, however, the errors in measurements under field conditions may be of the same order as the actual deflections and recovery. To avoid penalizing a satisfactory structure in such a case, recovery measurements are waived if the maximum deflection is less than \( \frac{2}{20,000}h \). The residual deflection \( \Delta_r \) 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 that is enhanced by clamping action of transverse stirrup reinforcement 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 the professionals in charge of the test be aware of 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 member 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 (ACI 349.3R-02),” American Concrete Institute, Farmington Hills, Mich., 2002, 18 pp.


CHAPTER R21—PROVISIONS FOR SEISMIC DESIGN

R21.1—Definitions

Design of reinforced concrete nuclear safety-related structures per ACI 349 is based on elastic response in the safe shutdown earthquake (SSE). Failure of such structures during shaking more severe than the SSE, termed the beyond-design-basis earthquake in this commentary, is substantially reduced by detailing the components for ductile response using provisions drawn from Chapter 21 of ACI 318.

R21.2—General requirements

R21.2.1 Scope—The intended scope of ACI 349 is the design and detailing of safety-related structures of nuclear power facilities and other such facilities as might be required by the appropriate governing bodies. The predominant structural form in such facilities is shear-wall-and-slab construction of generally heavy proportions. The safety-related structures and structural members and components of nuclear power plants are proportioned and reinforced so as to exhibit elastic behavior under all loads (except impulsive
and impactive loads per Appendix F) and load combinations including those associated with the SSE.

The performance objectives for conventional buildings and safety-related nuclear structures are significantly different: significant damage but no collapse in the design earthquake for conventional building structures versus elastic response (no significant damage) in the design earthquake for safety-related nuclear structures. The definitions of the return period of the design earthquake also differ substantially, namely, 500 to 1000 years for conventional building structures and 1000 to 10,000 years for safety-related nuclear structures. The beyond-design-basis earthquake shaking is also considered, albeit indirectly, for both conventional buildings (ACI 318) and safety-related nuclear structures (ACI 349). For conventional building structures, it is expected that a building proportioned for design-basis shaking and detailed per Chapter 21 of ACI 318-05 will not collapse in a maximum earthquake that is characterized by a return period of approximately 2500 years. For safety-related nuclear structures, the use of the prescriptive details of Chapter 21 of ACI 318-05 for special construction will provide a wide margin against collapse in the event of earthquake shaking characterized by a return period in excess of that of the SSE.

Many of the special seismic component provisions of Chapter 21 of ACI 318-05 are adopted for the seismic design of safety-related nuclear structures for three key reasons. First, and as noted previously, the adoption of the special seismic provisions provides substantial assurance that structural integrity will be maintained in the unlikely event of beyond-design-basis earthquake shaking or other unforeseen circumstances. Second, the adoption of the special seismic component provisions provides reinforcing bar detailing requirements consistent with the toughness needs of structural members designed for Special Facilities class of structures of the Department of Energy non-reactor nuclear production plants wherein limited inelastic response to design-basis earthquake shaking is permitted. ACI 349 is cited as the design Code by the governing design criteria document of these facilities. Third, adoption of many of the special seismic provisions maximizes the possible compatibility between ACI 318-05 and ACI 349.

The structural form of typical safety-related nuclear facilities differs substantially from the structural forms assumed in the development of ACI 318-05. As such, great care must be taken in the application of the prescriptive details of Chapter 21 of ACI 318-05 in safety-related nuclear structures. For components in safety-related nuclear structures that are substantially different from those found in building structures (for example, tunnels and dock walls), design and detailing should be based on fundamental principles of mechanics. Peer review of such designs is recommended.

The commentary that follows is taken in the large part from the commentary to ACI 318. As such, this commentary should be read in conjunction with that of ACI 318. This commentary has been adapted to suit the features and framing systems of safety-related nuclear structures.

**R21.2.2 Analysis and proportioning of structural members**—It is assumed that the distribution of required strength to the various components of a lateral-force-resisting system will be guided by the analysis of a linearly elastic model of the structure acted upon by the factored forces.

The main concern of Chapter 21 is the safety and integrity of the structure in SSE and beyond-design-basis earthquake shaking. All members of the structure that contribute strength and stiffness to the structure should be included in the linearly elastic model of the structure. All members of the structure should be detailed to sustain the deformations and displacements expected in beyond-design-basis earthquake shaking.

In most nuclear safety-related facilities, structural walls are used to resist earthquake-induced inertial forces. Other structural framing in these facilities include columns, beams and slabs, which might not contribute substantially to the earthquake resistance of the facility. In such structures, the columns, beams, and slabs can be exempted from the special design and detailing provisions for moment-frame structures of Sections 21.3 through 21.5, provided that the framing can sustain the forces and deformations associated with beyond-design-basis earthquake shaking.

Sections 21.2.2.2 through 21.2.2.4 of ACI 318-05 were not included in ACI 349. In ACI 349, all members, both rigid and flexible, must be included in the mathematical model of the nuclear facility negating the utility of Section 21.2.2.2 of ACI 318. Section 21.2.2.3 of ACI 318-05 does not apply to nuclear structures. Section 21.2.2.4 of ACI 318-05 applies to components of commercial building construction and not to members of nuclear facilities.

**R21.2.4 Concrete in members resisting earthquake-induced forces**—Requirements of this section refer to concrete quality in frames, trusses, or walls proportioned to resist earthquake-induced forces. The minimum specified compressive strength of concrete is 3000 psi. Lightweight concrete is not permitted in nuclear safety-related structures. The design rules of Chapter 21 apply for normalweight and heavyweight concrete.

**R21.2.5 Reinforcement in members resisting earthquake-induced forces**—The use of longitudinal reinforcement with strength substantially higher than that assumed in design will lead to higher shear and bond stresses at the time of development of yield moments. These conditions may lead to brittle failures in shear or bond and should be avoided even if such failures may occur at higher loads than those anticipated in design. Therefore, a limit is placed on the actual yield strength of the steel (see 21.2.5(a)).

The requirement for a tensile strength larger than the yield strength of the reinforcement is based on the assumption that the capability of a structural member to develop inelastic rotation capacity is a function of the length of the yielded region along the axis of the member. In interpreting experimental results, the length of the yielded region has been related to the relative magnitudes of ultimate and yield moments. According to this interpretation, the larger the ratio of ultimate to yield moment, the longer the yielded region. Chapter 21 requires that the ratio of actual
tensile strength to actual yield strength is not less than 1.25. Members with reinforcement not satisfying this condition can also develop inelastic rotation, but their behavior is sufficiently different to exclude them from direct consideration on the basis of rules derived from experience with members reinforced with strain-hardening steel.

R21.2.6 Mechanical splices—In a structure undergoing inelastic deformations during an earthquake, the tensile stresses in reinforcement may approach the tensile strength of the reinforcement. The ACI 349 requirements for mechanical splices are intended to avoid a splice failure when the reinforcement is subjected to stress levels expected in yielding regions in beyond-design-basis earthquake shaking. Type 1 splices of ACI 318-05 are not permitted in nuclear safety-related reinforced concrete structures.

Recommended detailing practice would preclude the use of splices in regions of potential yield in members resisting earthquake effects. If the use of mechanical splices in regions of potential yielding in beyond-design-basis earthquake shaking cannot be avoided, the designer should have documentation on the actual strength characteristics of the bars to be spliced, on the force-deformation characteristics of the spliced bars, and on the ability of the splice to be used to meet the specified performance requirements.

R21.2.7 Welded splices

R21.2.7.1 Welding of reinforcement should conform to ANSI/AWS D1.4 as required in Chapter 3. Locations for welded splices are not restricted in ACI 349 because the welded splice must develop the specified tensile strength of the reinforcing bars being spliced per Section 12.14.3.4.

R21.2.7.2 Welding of overlapping reinforcing bars can lead to local embrittlement of the steel. If welding of crossing bars is used to facilitate fabrication or placement of reinforcement, it should be done only on bars added for such purposes. The prohibition of welding crossing reinforcing bars does not apply to bars that are welded with welding operations under continuous, competent control as in the manufacture of welded wire reinforcement.

R21.3—Flexural members of moment frames

R21.3.1 Scope—This section refers to beams of special moment frames resisting lateral loads induced by earthquake motions. Any frame member subjected to a factored axial compressive force exceeding \( \frac{A_g f'c}{10} \) is to be proportioned and detailed as described in 21.4.

Experimental evidence\(^{21,2} \) indicates that, under reversals of displacement into the nonlinear range, behavior of continuous members having length-to-depth ratios of less than 4 is significantly different from the behavior of relatively slender members. Design rules derived from experience with relatively slender members do not apply directly to members with length-to-depth ratios less than 4, especially with respect to shear strength.

The geometric constraints indicated in 21.3.1.3 and 21.3.1.4 were derived from practice with reinforced concrete frames resisting earthquake-induced forces. \(^{21,3} \)

R21.3.2 Longitudinal reinforcement—Section 10.3.5 limits the net tensile strain \( \varepsilon_t \), thereby indirectly limiting the tensile reinforcement ratio in a flexural member to a fraction of the amount that would produce balanced strain conditions. For a section subjected to bending only and loaded monotonically to yielding, this approach is feasible because the likelihood of compressive failure can be estimated reliably with the behavioral model assumed for determining the reinforcement ratio corresponding to balanced failure. The same behavioral model, because of incorrect assumptions such as linear strain distribution, well-defined yield point for the steel, limiting compressive strain in the concrete of 0.003, and compressive stresses in the shell concrete, fails to describe the conditions in a flexural member subjected to reversals of displacements well into the inelastic range. Thus, there is little rationale for continuing to refer to balanced strain conditions in earthquake-resistant design of flexural members in reinforced concrete structures.

R21.3.2.1 The limiting reinforcement ratio of 0.025 is based primarily on considerations of steel congestion and, indirectly, on limiting shear stresses in flexural members of typical proportions. The requirement of at least two bars, top and bottom, refers again to construction rather than behavioral requirements.

R21.3.2.3 Lap splices of reinforcement are prohibited at regions where flexural yielding is anticipated because such splices are not reliable under conditions of cyclic loading into the inelastic range. Transverse reinforcement for lap splices at any location is mandatory because of the likelihood of loss of shell concrete.

R21.3.3 Transverse reinforcement—Transverse reinforcement is required primarily to confine the concrete and maintain lateral support for the reinforcing bars in regions where yielding is expected. Examples of hoops suitable for flexural members of frames are shown in Fig. R21.3.3.

In the case of members with varying strength along the span or members for which the permanent load represents a
large proportion of the total design load, concentrations of inelastic rotation may occur within the span. If such a condition is anticipated, transverse reinforcement also should be provided in regions where yielding is expected in beyond-design-basis earthquake shaking.

Because spalling of the concrete shell might occur during beyond-design-basis earthquake shaking, especially at and near regions of flexural yielding, all web reinforcement should be provided in the form of closed hoops as defined in 21.3.3.5.

R21.3.4 Shear strength requirements

R21.3.4.1 Design forces—In ACI 349, it is assumed that frame members will dissipate energy in the nonlinear range of response during beyond-design-basis earthquake shaking. Unless a frame member possesses a strength that is a multiple on the order of 2 or 4 of the SSE forces, it should be assumed that it will yield in the event of beyond-design-basis earthquake shaking. The design shear force should be a good approximation of the maximum shear that may develop in a member. Therefore, required shear strength for frame members is related to flexural strengths of the designed member rather than to factored shear forces indicated by SSE analysis. The conditions described by 21.3.4.1 are illustrated in Fig. R21.3.4.

Because the actual yield strength of the longitudinal reinforcement may exceed the specified yield strength and because strain hardening of the reinforcement is likely to take place at a joint subjected to large rotations, required shear strengths are determined using a stress of at least $1.25f_y$ in the longitudinal reinforcement. Consideration should also be given to increases in flexural strength associated with slab effects and the use of reinforcing bars with yield strengths in excess of the specified minimum yield strength.

R21.3.4.2 Transverse reinforcement—Experimental studies21.4,21.5 of reinforced concrete members subjected to cyclic loading have demonstrated that more shear reinforcement is required to ensure a flexural failure if the member is subjected to alternating nonlinear displacements than if the member is loaded in only one direction: the necessary increase of shear reinforcement being higher in the case of no axial load. This observation is reflected in the Code (see 21.3.4.2) by eliminating the term representing the contribution of concrete to shear strength. The added conservatism on shear strength is deemed necessary in locations where potential flexural hinging may occur. However, this stratagem, chosen for its relative simplicity, should not be interpreted to mean that no concrete is required to resist shear. On the contrary, it may be argued that the concrete core resists the entire shear with the shear (transverse) reinforcement confining and strengthening the concrete. The confined concrete core plays an important role in the behavior of the beam and should not be reduced to a minimum just because the design expression does not explicitly recognize it.

R21.4—Moment frame members subjected to bending and axial load

R21.4.1 Scope—Section 21.4.1 is intended primarily for columns of moment frames. Frame members, other than columns, that do not satisfy 21.3.1 are to be proportioned and detailed according to this section. The geometric constraints in 21.4.1.1 and 21.4.1.2 follow from previous practice21.3

R21.4.2 Minimum flexural strength of columns—The intent of 21.4.2.2 is to reduce the likelihood of yielding in columns in beyond-design-basis earthquake shaking. The strong-column/weak-beam philosophy of ACI 318-05 must be followed in ACI 349. The factor of 1.4 (= 7/5) is that
recommended by ACI Committee 352. In the case of weak columns, flexural yielding might occur at both ends of all columns in a given story in beyond-design-basis earthquake shaking, resulting in a column failure mechanism that can lead to collapse, which is unacceptable for nuclear structures.

When determining the nominal flexural strength of a girder section in negative bending (top in tension), longitudinal reinforcement contained within an effective flange width of a top slab that acts monolithically with the girder increases the girder strength. Research\textsuperscript{21.6} on beam-column subassemblies under lateral loading indicates that using the effective flange widths defined in 8.10 gives reasonable estimates of girder negative bending strengths of interior connections at interstory displacement levels approaching 2% of story height. This effective width is conservative where the slab terminates in a weak spandrel beam.

**R21.4.3 Longitudinal reinforcement**—The lower limit of the reinforcement ratio is to control time-dependent deformations and to have the yield moment exceed the cracking moment. The upper limit of the section reflects concern for steel congestion, load transfer from floor members to columns (especially in low-rise construction), and the development of high shear stresses.

Spalling of the shell concrete, which is likely to occur near the ends of the column in frames of typical configuration, makes lap splices in these locations vulnerable in beyond-design-basis earthquake shaking. If lap splices are to be used, they should be located near the midheight of a column where stress reversal is likely to be limited to a smaller stress range than at locations near the joints. Special transverse reinforcement is required along the lap-splice length because of the uncertainty in moment distributions along the height and the need for confinement of lap splices subjected to stress reversals.\textsuperscript{21.7}

**R21.4.4 Transverse reinforcement**—Requirements of this section are concerned with confining the concrete and providing lateral support to the longitudinal reinforcement.

The effect of helical (spiral) reinforcement and adequately configured rectangular hoop reinforcement on strength and ductility of columns is well established.\textsuperscript{21.8} While analytical procedures exist for calculation of strength and ductility capacity of columns under axial and moment reversals,\textsuperscript{21.9} the axial load and deformation demands required during beyond-design-basis earthquake loading are not known with sufficient accuracy to justify calculation of required transverse reinforcement as a function of beyond-design-basis earthquake demands. Instead, Eq. (10-5) and (21-3) are required, with the intent that spalling of shell concrete will not result in a loss of axial load strength of the column. Equations (21-2) and (21-4) govern for large-diameter columns, and are intended to ensure adequate flexural curvature capacity in yielding regions.

Figure R21.4.4 shows an example of transverse reinforcement provided by one hoop and three crossties. Crossties with a 90-degree hook are not as effective as either crossties with 135-degree hooks or hoops in providing confinement. Tests show that if crosstie ends with 90-degree hooks are alternated, confinement will be sufficient.

**Consecutive crossties engaging the same longitudinal bar have their 90-deg hooks on opposite sides of column.**

**Fig. R21.4.4—Example of transverse reinforcement in columns.**

Sections 21.4.4.2 and 21.4.4.3 are interrelated requirements for configuration of rectangular hoop reinforcement. The requirement that spacing not exceed 1/4 of the minimum member dimension is to obtain adequate concrete confinement. The requirement that spacing not exceed six bar diameters is intended to restrain longitudinal reinforcement buckling after spalling. The 4 in. spacing is for concrete confinement; 21.4.4.2 permits this limit to be relaxed to a maximum of 6 in. if the spacing of crossties or legs of overlapping hoops is limited to 8 in.

The unreinforced shell may spall as the column deforms to resist beyond-design-basis earthquake shaking. Separation of portions of the shell from the core caused by local spalling creates a falling hazard. The additional reinforcement is required to reduce the risk of portions of the shell falling away from the column. The 6 in. limit on the spacing of the additional transverse reinforcement reduces the risk below that for conventional reinforced concrete structures.

Section 21.4.4.4 stipulates a minimum length over which to provide closely-spaced transverse reinforcement at the member ends, where flexural yielding might occur during beyond-design-analysis earthquake shaking. Research results indicate that the length should be increased by 50% or more in locations, such as the base of the building, where axial loads and flexural demands may be especially high\textsuperscript{21.10} or the point of zero moment in the column is remote from the midheight of the column.

Columns supporting discontinued stiff members, such as walls or trusses, may develop considerable inelastic response in beyond-design-basis earthquake shaking. Therefore, it is required that these columns have special transverse reinforcement throughout their length. This covers all columns beneath the level at which the stiff member has been discontinued, unless the factored forces corresponding to earthquake effect are extremely low (see 21.4.4.5).

Field observations have shown significant damage to columns in the unconfined region near the midheight. The requirements of 21.4.4.6 are to ensure a relatively uniform toughness of the column along its length.
R21.4.5 Shear strength requirements

R21.4.5.1 Design forces—The provisions of 21.3.4.1 also apply to members subjected to axial loads—for example, columns. Above the ground floor, the moment at a joint may be limited by the flexural strength of the beams framing into the joint. Where beams frame into opposite sides of a joint, the combined strength may be the sum of the negative moment strength of the beam on one side of the joint and the positive moment strength of the beam on the other side of the joint. Moment strengths are to be determined using a strength-reduction factor of 1.0 and reinforcing steel stress equal to at least 1.25fy. Distribution of the combined moment strength of the beams to the columns above and below the joint should be based on analysis. The value of Mpr in Fig. R21.3.4 may be computed from the flexural member strengths at the beam-column joints. Consideration should be given to both reinforcing steel stresses higher than 125% of the specified minimum yield strength and the slab contribution to the strength of the beam. See also 21.3.4.1.

R21.5—Joints of moment frames

R21.5.1 General requirements—Development of inelastic rotations at the faces of joints of reinforced concrete frames is associated with strains in the flexural reinforcement well in excess of the yield strain. Consequently, joint shear force generated by the flexural reinforcement is calculated for a stress of 1.25fy in the reinforcement. A detailed explanation of the reasons for the possible development of stresses in excess of the yield strength in girder tensile reinforcement is provided in Reference 21.1.

R21.5.1.4 Research21.15 has shown that straight beam bars may slip within the beam-column joint during a series of large moment reversals. The bond stresses on these straight bars may be very large. To substantially reduce slip during the formation of adjacent beam hinging, it would be necessary to have a ratio of column dimension to bar diameter of approximately 1/32, which would result in very large joints. On reviewing the available tests, the limit of 1/20 of the column depth in the direction of loading for the maximum size of beam bars was chosen. This limit provides reasonable control on the amount of potential slip of the beam bars in a beam-column joint, considering the number of anticipated inelastic excursions of the building frames during a major earthquake. A thorough treatment of this topic is given in Reference 21.16.

R21.5.2 Transverse reinforcement—How ever low the calculated shear force in a joint of a frame resisting earthquake-induced forces, confining reinforcement (see 21.4.4) should be provided through the joint around the column reinforcement (see 21.5.2.1). In 21.5.2.2, confining reinforcement may be reduced if horizontal members frame into all four sides of the joint.

Section 21.5.2.3 refers to a joint where the width of the beam exceeds the corresponding column dimension. In that case, beam reinforcement not confined by the column reinforcement should be provided lateral support either by a beam framing into the same joint or by transverse reinforcement.

R21.5.3 Shear strength—The requirements in Chapter 21 for proportioning joints are based on Reference 21.1 in that behavioral phenomena within the joint are interpreted in terms of a nominal shear strength of the joint. Because tests of joints21.11 and deep beams21.2 indicated that shear strength was not as sensitive to joint (shear) reinforcement as implied by the expression developed by Joint ACI-ASCE Committee 32621.17 for beams and adopted to apply to joints by ACI Committee 352,21.18 Committee 318 set the strength of the joint as a function of only the compressive strength of the concrete (see 21.5.3) and to require a minimum amount of transverse reinforcement in the joint (see 21.5.2). The three levels of shear strength required by 21.5.3 are based on the recommendation of ACI Committee 352. The effective area of joint Aj is illustrated in Fig. R21.5.3. In no case should Aj be greater than the column cross-sectional area.

R21.5.4 Development length of bars in tension—Minimum development length for deformed bars with standard hooks embedded in normalweight concrete is determined using Eq. (21-6). Equation (21-6) is based on the requirements of 12.5. Because Chapter 21 stipulates that the hook is to be embedded in confined concrete, the coefficients 0.7 (for concrete cover) and 0.8 (for ties) have been incorporated in the constant used in Eq. (21-6). The development length that would be derived directly from 12.5 is increased to reflect the effect of load reversals.

The development length in tension for a reinforcing bar with a standard hook is defined as the distance, parallel to the bar, from the critical section (where the bar is to be developed) to a tangent drawn to the outside edge of the hook. The tangent is to be drawn perpendicular to the axis of the bar (see Fig. R12.5 in ACI 318).

Factors such as the actual stress in the reinforcement being more than the yield strength and the effective development length not necessarily starting at the face of the joint were implicitly considered in the evolution of the expression for basic development length that has been used as the basis for Eq. (21-6).
Section 21.5.4.2 specifies the minimum development length for straight bars as a multiple of the length indicated by 21.5.4.1. Section 21.5.4.2(b) refers to top bars.

If the required straight embedment length of a reinforcing bar extends beyond the confined volume of concrete (as defined in 21.3.3, 21.4.4, or 21.5.2), the required development length is increased on the premise that the limiting bond stress outside the confined region is less than that inside.

\[ \ell_{dm} = 1.6(\ell_d - \ell_{dc}) + \ell_{dc} \]

or

\[ \ell_{dm} = 1.6\ell_d - 0.6\ell_{dc} \]

where

- \( \ell_{dm} \) = required development length if bar is not entirely embedded in confined concrete;
- \( \ell_d \) = required development length for straight bar embedded in confined concrete (see 21.5.4.3);
- \( \ell_{dc} \) = length of bar embedded in confined concrete.

Lack of reference to No. 14 and No. 18 bars in 21.5.4 is due to the paucity of information on anchorage of such bars subjected to load reversals simulating earthquake effects.

R21.6—Intentionally left blank

Section 21.6 in ACI 318-05 provides design provisions for precast concrete moment-frame construction. Because (a) precast construction is not used for new nuclear safety-related concrete structures at this time, and (b) the provisions have not yet seen widespread use in the building-structures community, Committee 349 decided to eliminate Section 21.6 from Chapter 21 of ACI 349.

R21.7—Reinforced concrete structural walls and coupling beams

R21.7.1 Scope—This section contains requirements for the dimensions and details of special reinforced concrete structural walls and coupling beams. Provisions for diaphragms are in 21.9.

R21.7.2 Reinforcement—Minimum reinforcement requirements (see 21.7.2.1) follow from preceding Codes. The uniform distribution requirement of the shear reinforcement is related to the intent to control the width of inclined cracks. The requirement for two layers of reinforcement in walls resisting substantial shear forces (see 21.7.2.2) is based on the observation that, under ordinary construction conditions, the probability of maintaining a single layer of reinforcement near the middle of the wall section is quite low. Furthermore, presence of reinforcement close to the surface tends to inhibit fragmentation of the concrete in the event of severe cracking during beyond-design-basis earthquake shaking.

R21.7.2.3 Requirements for splicing longitudinal (horizontal and vertical) reinforcement were modified in ACI 318-05 Code to remove the reference to beam-column joints in 21.5.4, which were unclear when applied to walls. Because actual forces in longitudinal reinforcement in structural walls may exceed calculated forces, reinforcement should be developed or spliced to reach the yield strength of the bar in tension. Requirements of 12.11, 12.12, and 12.13 address issues related to beams and do not apply to walls. At locations where yielding of the reinforcement is expected, a 1.10 multiplier is applied to account for the likelihood that the actual yield strength exceeds the specified yield strength of the bars. (This factor is smaller than the value of 1.25 imposed in ACI 318-05 that is intended to account for actual yield strength exceeding the specified yield strength, the influence of strain hardening, and cyclic load reversals, because substantial cyclic inelastic reversals are not expected in wall designs using this standard.) Splices should be Class B per Section 12.15 with \( \ell_d \) calculated per 21.7.2.3.

Where transverse reinforcement is used, development lengths for straight and hooked bars may be reduced as permitted in 12.2 and 12.5, respectively, because closely-spaced transverse reinforcement improves the performance of splices and hooks subjected to repeated inelastic demands.\textsuperscript{21.19} For walls with \( h_w/l_w > 2.0 \), the effective depth may be taken as \( 0.8l_w \) per Section 12.10.3 for establishing termination points for flexural reinforcement used to resist forces in the plane of the wall.

R21.7.4 Shear strength—Equation (21-7) recognizes the higher shear strength of walls with high shear-to-moment ratios.\textsuperscript{21.1,21.17,21.20} The nominal shear strength is given in terms of the net area of the section resisting shear. For a rectangular section without openings, the term \( A_{cv} \) refers to the gross area of the cross section rather than to the product of the width and the effective depth. The definition of \( A_{cv} \) in Eq. (21-7) facilitates design calculations for walls with uniformly distributed reinforcement and walls with openings.

Equation (21-7) is taken from Section 21.7 of ACI 318-05 and provides similar estimates of nominal shear strength to Eq. (11-31) for low values of axial load. The nominal shear strength calculated using Eq. (21-7) is for rectangular walls, without boundary elements or flanges. Nominal shear strengths substantially greater than that predicted by Eq. (21-7) will likely be attained in walls with boundary elements or flanges, but the hysteretic response of such walls is not well understood at this time. The nominal shear strength limit imposed by Eq. (21-7) is therefore retained.

A wall segment refers to a part of a wall bounded by openings or by an opening and an edge. Traditionally, a vertical wall segment bounded by two window openings has been referred to as a pier.

The ratio \( h_w/l_w \) may refer to overall dimensions of a wall, or of a segment of the wall bounded by two openings, or an opening and an edge. The intent of 21.7.4.2 is to make certain that any segment of a wall is not assigned a unit strength larger than that for the whole wall. However, a wall segment with a ratio of \( h_w/l_w \) higher than that of the entire wall should be proportioned for the unit strength associated with the ratio \( h_w/l_w \) based on the dimensions for that segment.

To restrain the inclined cracks effectively, reinforcement included in \( \rho_t \) and \( \rho_p \) should be appropriately distributed along the length and height of the wall (see 21.7.4.3). Chord reinforcement provided near wall edges in concentrated amounts for resisting bending moment is not to be included...
in determining $\rho_t$ and $\rho_c$. Within practical limits, shear reinforcement distribution should be uniform and at a small spacing.

If the factored shear force at a given level in a structure is resisted by several walls or several piers of a perforated wall, the average unit shear strength assumed for the total available cross-sectional area is limited to $8\sqrt{f_{c}'}$ with the additional requirement that the unit shear strength assigned to any single pier does not exceed $10\sqrt{f_{c}'}$. The upper limit of strength to be assigned to any one member is imposed to limit the degree of redistribution of shear force.

“Horizontal wall segments” in 21.7.4.5 refers to wall sections between two vertically aligned openings (refer to Fig. R21.7.4.5). It is, in effect, a pier rotated through 90 degrees. A horizontal wall segment is also referred to as a coupling beam when the openings are aligned vertically over the building height.

**R21.7.5 Design for flexure and axial loads**

**R21.7.5.1** Flexural strength of a wall or wall segment is determined according to procedures commonly used for columns. Strength should be determined considering the applied axial and lateral forces. Reinforcement concentrated in boundary elements and distributed in flanges and webs should be included in the strength computations based on a strain compatibility analysis. The foundation supporting the wall should be designed to develop the wall boundary and web forces for beyond-design-basis earthquake shaking. For walls with openings, the influence of the opening or openings on flexural and shear strengths is to be considered and a load path around the opening or openings should be verified. Capacity design concepts and strut-and-tie models may be useful for this purpose.$^{21.21}$

**R21.7.5.2** Where wall sections intersect to form L-, T-, C-, or other cross-sectional shapes, the influence of the flange on the behavior of the wall should be considered by selecting appropriate flange widths. Tests$^{21.22}$ show that effective flange width increases with increasing drift level and the effectiveness of a flange in compression differs from that for a flange in tension. The value used for the effective compression flange width has little impact on the strength and deformation capacity of the wall; therefore, to simplify design, a single value of effective flange width based on an estimate of the effective tension flange width is used in both tension and compression.$^{21.22}$

**R21.7.6 Boundary elements of reinforced concrete structural walls**

**R21.7.6.1** ACI 349 procedures for assessing the need for boundary elements differ substantially from those of ACI 318-05. The procedures of ACI 318-05 apply to flexural walls with a single critical section for flexure at the base of the wall. Most walls in nuclear power-plant construction are squat walls (typical aspect ratio $h_w/l_w \leq 2.0$) and not flexural walls.

Squat walls are typically shear-critical, and the flexural strain limits of Sections 21.7.6.2 and 21.7.6.3 of ACI 318-05 do not apply to such walls. Accordingly, checking for boundary elements is not required for squat walls in ACI 349. This approach is consistent with that adopted in ACI 349-01.

**R21.7.6.2** Section 21.7.6.2 of ACI 318-05 is based on the assumption that inelastic response of the wall is dominated by flexural action at a critical, yielding section. Equation (21-8) in ACI 318-05 follows from a displacement-based approach$^{21.23,21.24}$ and avoids the need for explicit calculation of maximum concrete strains. The approach of ACI 349 is nearly identical to that of ACI 318, namely, that boundary elements are required to confine the concrete where the strain at the extreme compression fiber of the wall exceeds a critical value when the wall is displaced to the maximum displacement. In safety-related reinforced concrete nuclear structures, boundary elements might be required to resist the effects of beyond-design-basis earthquake shaking but are not required to resist the effects of SSE shaking because nuclear structures are designed for elastic response for such shaking.

The horizontal dimension of the boundary element is intended by ACI 318-05 to extend at least over the length where the compression strain exceeds the assumed spalling strain of 0.004. The height of the boundary element is based on upper-bound estimates of plastic hinge length and extends beyond the zone over which concrete spalling is likely to occur.

The displacement-oriented approach of ACI 349 requires explicit calculation of maximum concrete compressive strains by cross-section analysis as described in 10.2. The strain limit of 0.002 for combined actions including SSE shaking was adopted by ACI Committee 349 instead of seismic analysis for beyond-design-basis earthquake shaking, recognizing that a maximum concrete compressive strain in excess of 0.004 is the trigger for boundary elements, as assumed in ACI 318-05. As such, the beyond-design-basis earthquake displacement is assumed to be less than twice the SSE displacement. If beyond-design-basis earthquake displacements greater than twice the SSE displacement are anticipated, direct calculation of maximum concrete strains is recommended and the need for boundary elements should be assessed by comparing the maximum strain with a threshold value that is assumed herein to be 0.004.

A direct displacement-based approach can be used to establish the need for boundary elements in flexural walls for beyond-design-basis earthquake shaking. Such an approach
must follow rational strain-based analysis procedures but will require explicit calculation of beyond-design-basis earthquake displacements. Peer review is recommended if a direct displacement-based design is adopted.

R21.7.6.3 The index method of ACI 318-05 for flexural walls is not included in ACI 349 because the method is indirect and produces conservative estimates of the need for boundary elements. The strain-based approach of Section 21.7.6.2 eliminates the need for the index method.

R21.7.6.4 The value of $c/2$ in 21.7.6.4(a) is to provide a minimum length of the boundary element. The value of $c$ shall be calculated by cross-section analysis per 10.2. The neutral axis depth $c$ is calculated for SSE displacements and forces on the assumption that the neutral axis depth will not change substantially for beyond-design-basis earthquake shaking forces and displacements.

Where flanges are heavily stressed in compression, the web-to-flange interface is likely to be heavily stressed and may sustain local crushing failure unless boundary element reinforcement extends into the web. Equation (21-3) does not apply to walls.

Because horizontal reinforcement is likely to act as web reinforcement in walls requiring boundary elements, it should be fully anchored in boundary elements that act as flanges (21.7.6.4). Achievement of this anchorage is difficult when large transverse cracks occur in the boundary elements. Therefore, standard 90-degree hooks or mechanical anchorage schemes are recommended instead of straight bar development.

R21.7.6.5 Cyclic load reversals in beyond-design-basis earthquake shaking may lead to buckling of boundary longitudinal reinforcement that has previously yielded in tension even in cases where the demands on the boundary of the wall do not require boundary elements. For walls with moderate amounts of boundary longitudinal reinforcement, transverse ties are required to inhibit buckling of bars that yield in compression. The longitudinal reinforcement ratio is intended to include only the reinforcement at the wall boundary as indicated in Fig. R21.7.6.5. The limit on reinforcement adopted in ACI 349 is similar to that proposed in Reference 21.25 ($\rho = 430/f_y$) for walls of restricted ductility. The maximum spacing of ties of the lesser of 12 in. or $10d_b$ is a conservative interpretation of Reference 21.25 for walls of restricted ductility. The larger spacing of ties relative to 21.7.6.4(c) and 21.7.6.5 of ACI 318-05 is due to the lower inelastic deformation demands on walls in reinforced concrete nuclear structures.

The addition of hooks or U-stirrups at the ends of horizontal wall reinforcement provides anchorage so that the reinforcement will be effective in resisting shear forces. It will also tend to inhibit the buckling of the vertical edge reinforcement. In walls with low in-plane shear, the development of horizontal reinforcement is not necessary.

R21.7.7 Coupling beams—Coupling beams connecting structural walls can provide stiffness and energy dissipation. In many cases, geometric limits result in coupling beams that are deep in relation to their clear span. Deep coupling beams may be controlled by shear and may be susceptible to strength and stiffness deterioration under earthquake loading. Test results have shown that confined diagonal reinforcement provides adequate resistance in deep coupling beams.

Experiments show that diagonally oriented reinforcement is effective only if the bars are placed with a large inclination. Therefore, diagonally reinforced coupling beams are restricted to beams having aspect ratio $l_n/h < 4$.

Each diagonal element consists of a cage of longitudinal and transverse reinforcement as shown in Fig. R21.7.7. The cage contains at least four longitudinal bars and confines a concrete core. The requirement on side dimensions of the cage and its core is to provide adequate toughness and stability to the cross section when the bars are loaded beyond yielding. The minimum dimensions and required reinforcement clearances may control the wall width.

Tests in Reference 21.27 demonstrated that beams reinforced as described in Section 21.7.7 have adequate ductility at shear forces exceeding $10f_yb_wd$. Consequently, the use of a limit of $10f_yb_wA_{cw}$ provides an acceptable upper limit.

When the diagonally oriented reinforcement is used, additional reinforcement in 21.7.7.4(f) is to contain the concrete...
outside the diagonal cores if the concrete is damaged by beyond-design-basis earthquake shaking (Fig. R21.7.7).

**R21.8—Intentionally left blank**

Section 21.8 in ACI 318-05 provides design provisions for precast concrete wall construction. Because (a) precast construction is not used for new nuclear safety-related concrete structures at this time, and (b) the provisions have not yet seen widespread use in the building-structures community, Committee 349 decided to eliminate Section 21.8 from Chapter 21 of ACI 349-06.

**R21.9—Structural diaphragms and trusses**

**R21.9.1 Scope**—Diaphragms as used in building construction are structural members (such as a floor or roof) that provide some or all of the following functions:

(a) Support for building elements (such as walls, partitions, and cladding) resisting horizontal forces but not acting as part of the building vertical lateral-force-resisting system;

(b) Transfer of lateral forces from the point of application to the building vertical lateral-force-resisting system;

(c) Connection of various members of the building vertical lateral-force-resisting system with appropriate strength, stiffness, and toughness so the building responds as intended in the design.

**R21.9.2 Cast-in-place composite-topping slab diaphragms**—A bonded topping slab is required so that the floor or roof system can provide restraint against slab buckling. Reinforcement is required to ensure the continuity of the shear transfer across precast joints. The connection requirements are introduced to promote a complete system with necessary shear transfers.

**R21.9.3 Cast-in-place topping slab diaphragms**—Composite action between the topping slab and the precast floor elements is not required, provided that the topping slab is designed to resist the design seismic forces.

**R21.9.4 Minimum thickness of diaphragms**—The minimum thickness of concrete diaphragms reflects current practice in joist and waffle systems and composite topping slabs on precast floor and roof systems. Thicker slabs are required when the topping slab does not act compositely with the precast system to resist the design seismic forces.

**R21.9.5 Reinforcement**—Minimum reinforcement ratios for diaphragms correspond to the required amount of shrinkage and temperature reinforcement (7.12). The maximum spacing for web reinforcement is intended to control the width of inclined cracks. Minimum average prestress requirements (7.12.6) are considered to be adequate to limit the crack widths in post-tensioned floor systems; therefore, the maximum spacing requirements do not apply to these systems.

The minimum spacing requirement for welded wire reinforcement in topping slabs on precast floor systems (see 21.9.5.1) is to avoid fracture of the distributed reinforcement during an earthquake. Cracks in the topping slab open immediately above the boundary between the flanges of adjacent precast members, and the wires crossing those cracks are restrained by the transverse wires. Therefore, all the deformation associated with cracking should be accommodated in a distance not greater than the spacing of the transverse wires. A minimum spacing of 10 in. for the transverse wires is required in 21.9.5.1 to reduce the likelihood of fracture of the wires crossing the critical cracks during a design earthquake. The minimum spacing requirements do not apply to diaphragms reinforced with individual bars because strains are distributed over a longer length.

Following the design philosophy of ACI 349, diaphragms in nuclear structures must remain elastic in SSE shaking. In the event of beyond-design-basis earthquake shaking, the demand on a diaphragm might exceed the strength of the diaphragm. In that event, special transverse reinforcement is required to provide confinement for the concrete and the reinforcement (21.9.5.3).

Because calculations are not required for diaphragm demands associated with beyond-design-basis earthquake shaking, an indirect approach is adopted in 21.9.5.3 to establish the need for boundary elements. Specifically, beyond-design-basis earthquake displacements are assumed to be twice the SSE displacements, and boundary elements are required if the diaphragm demand during SSE shaking exceeds 50% of the diaphragm strength. Similar to structural walls, the 50% limit can be relaxed if displacement-oriented calculations are made to establish maximum concrete strains due to beyond-design-basis earthquake shaking.

Bar development and lap splices are designed according to requirements of Chapter 12 for reinforcement in tension. Reductions in development or splice length for calculated stresses less than $f_y$ are not permitted, as indicated in 12.2.5.

**R21.9.7 Shear strength**—The shear strength requirements for monolithic diaphragms, Eq. (21-9) in 21.9.7.1, are the same as those for slender structural walls. The term $A_{cp}$ refers to the thickness times the width of the diaphragm. This corresponds to the gross area of the effective deep beam that forms the diaphragm. The shear reinforcement should be placed perpendicular to the span of the diaphragm.

The shear strength requirements for topping slab diaphragms are based on a shear friction model, and the contribution of the concrete to the nominal shear strength is not included in Eq. (21-10) for topping slabs placed over precast floor elements. Following typical construction practice, the topping slabs are roughened immediately above the boundary between the flanges of adjacent precast floor members to direct the paths of shrinkage cracks. As a result, critical sections of the diaphragm are cracked under service loads, and the contribution of the concrete to the shear capacity of the diaphragm may have already been reduced before the design earthquake occurs.

**R21.9.8 Boundary elements of structural diaphragms**—For structural diaphragms, the factored moments are assumed to be resisted entirely by chord forces acting at opposite edges of the diaphragm. Reinforcement located at the edges of collectors should be fully developed for its yield strength. Adequate confinement of lap splices is also required. If chord reinforcement is located within a wall, the
joint between the diaphragm and the wall should be provided with adequate shear strength to transfer the shear forces.

Section 21.9.8.3 is intended to reduce the possibility of chord buckling in the vicinity of splices and anchorage zones.

R21.10—Foundations

R21.10.1 Scope—It is desirable that inelastic response in beyond-design-basis earthquake shaking occur above the foundations, as repairs to foundations can be extremely difficult and expensive.

R21.10.2 Footings, foundation mats, and pile caps

R21.10.2.2 Tests\textsuperscript{21,30} have demonstrated that flexural members terminating in a footing, slab, or beam (a T-joint) should have their hooks turned inward toward the axis of the member for the joint to be able to resist the flexure in the member forming the stem of the T.

R21.10.2.3 Columns or boundary members supported close to the edge of the foundation, as often occurs near property lines, should be detailed to prevent an edge failure of the footing, pile cap, or mat.

R21.10.2.4 The purpose of 21.10.2.4 is to alert the designer to provide top reinforcement as well as other required reinforcement.

R21.10.3 Grade beams and slabs-on-ground—For seismic loads, slabs-on-ground (soil-supported slabs) might form part of the lateral-force-resisting system and should be designed in accordance with this Code as well as other appropriate standards or guidelines. See R1.1.6.

R21.10.3.2 Grade beams between pile caps or footings can be separate beams beneath the slab-on-ground or can be a thickened portion of the slab-on-ground. The cross-sectional limitation and minimum tie requirements provide reasonable proportions.

R21.10.3.3 Grade beams resisting seismic flexural stresses from column moments should have reinforcing steel details similar to the beams of the frame above the foundation.

R21.10.3.4 Slabs-on-ground often act as diaphragms to hold the building together at the ground level and minimize the effects of out-of-phase ground motion that may occur over the footprint of the building. In these cases, the slab-on-ground should be adequately reinforced and detailed. The design drawings should clearly state that these slabs-on-ground are structural members so as to prohibit sawcutting of the slab.

R21.10.4 Piles, piers, and caissons—Adequate performance of piles and caissons for seismic loadings requires that these provisions be met in addition to other applicable standards or guidelines.

R21.10.4.2 A load path is necessary at pile caps to transfer tension forces from the reinforcing bars in the column or boundary member through the pile cap to the reinforcement of the pile or caisson.

R21.10.4.3 Grouted dowels in a blockout in the top of a precast concrete pile need to be developed, and testing is a practical means of demonstrating strength. Alternatively, reinforcing bars can be cast in the upper portion of the pile, exposed by chipping of concrete and mechanically spliced or welded to an extension.

R21.10.4.4 During earthquakes, piles can be subjected to extremely high flexural demands at points of discontinuity, especially just below the pile cap and near the base of a soft or loose soil deposit. Transverse reinforcement is required in this region to facilitate ductile response. The designer should also consider possible inelastic action in the pile at abrupt changes in soil deposits, such as changes from soft to firm or loose to dense soil layers. Where precast piles are to be used, the potential for the pile tip to be driven to an elevation different than that specified in the design drawings needs to be considered when detailing the pile. If the pile reaches refusal at a shallower depth, a longer length of pile will need to be cut off. If this possibility is not foreseen, the length of transverse reinforcement required by 21.10.4.4 may not be available after the excess pile length is cut off.

R21.10.4.7 Extensive structural damage has often been observed at the junction of batter piles and the building. The pile cap and surrounding structure should be designed for the large forces that can be developed in batter piles.

References


APPENDIX RA—STRUT-AND-TIE MODELS

The commentary of ACI 318-05 is applicable to this chapter.

APPENDIX RB—INTENTIONALLY LEFT BLANK

APPENDIX RC—ALTERNATIVE LOAD AND STRENGTH-REDUCTION FACTORS

The commentary of ACI 318-05, Appendix C, is applicable, except as provided herein.

RC.1—General

RC.1.1 In the 2006 Code, the load and strength-reduction factors in Chapter 9 were revised. The factors in the 2001 Code have been moved to this appendix. This revision has been performed to align with the changes of ACI 318-05, which changed to align with SEI/ASCE 7-02.

RC.2—Required strength

Load Combinations (C-1) to (C-3) involve normal loads and normal loads in combination with severe environmental loads. Similar to ACI 318-05, Appendix C, 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.

Unit load factors are used in Load Combinations (C-4) and (C-5) involving extreme environmental loads. This recognizes that the loads caused by the SSE or the design basis tornado are extreme and are of very low probability.
Load Combinations (C-6) to (C-8) are directed toward 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 (C-8) has unit load factors on all loads because it represents an extremely unlikely combination of events. Load Combination (C-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 (C-8). Load Combination (C-7) is intermediate between (C-6) and (C-8) and is included because it has been specified historically. In Combinations (C-7) and (C-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 $E_0$ and $P_a$ in Combinations (C-6) and (C-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 SSE 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, (C-9), (C-10), and (C-11), are similar to Load Combinations (C-1), (C-2), and (C-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 the design pressure is also used in ACI 318-05.

In applying the load combinations in Section C.2.1 of Appendix C, due regard should be given to sign because the Code recognizes the low likelihood event of a SSE (DBE) occurring while the crane is in use. A probability analysis should be performed to demonstrate that this combination is credible. A probability of 10E-06 or smaller is a reasonable measure to demonstrate incredibility.

**APPENDIX RD—ANCHORING TO CONCRETE**

References to hooked bolts (J- or L-bolt), as discussed in ACI 318-05, have been deleted because these anchors do not have a ductile failure mode, which is strongly recommended for anchors used in nuclear safety-related structures.

**RD.1—Definitions**

- **brittle steel element**
- **ductile steel element**—the 14% elongation should be measured over the gage length specified in the appropriate ASTM standard for the steel.

**five percent fractile**—the determination of the coefficient $K_{05}$ associated with the 5% fractile, $\bar{X} - K_{05}S_x$, depends on the number of tests, $n$, used to compute $\bar{X}$ and $S_x$. Values of $K_{05}$ range, for example, from 1.645 for $n = \infty$, to 2.010 for $n = 40$, and 2.568 for $n = 10$.

**RC.3—Design strength**

In Section C.3.4 of ACI 318-05, $\phi = 0.60$ is used for shear critical members, reflecting the lower ductility of shear-critical members than flexure-critical members. Under cyclic inelastic loading, shear-critical members such as squat shear-walls exhibit hysteresis that is characterized by degradation of stiffness and strength, and pinching. Although structural members in safety-related nuclear structures are detailed typically for inelastic response per conventional building construction, the inelastic deformation demands on components in nuclear structures are not as high as those in conventional special building structures. As such, loss of strength and stiffness due to cyclic inelastic loading in structural members of nuclear structures will be smaller than those in conventional building structures, and $\phi = 0.85$ (as in C.3.2.3) is considered to be adequate for the conditions delineated in section C.3.4 of ACI 318-05. Thus, Section C.3.4 of ACI 318-05 is not incorporated in ACI 349.

Fig. RD.1—Types of fasteners.
RD.2—Scope

RD.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 that is described in the 2001 Code.

RD.2.3 Typical cast-in headed studs and headed bolts with geometries consistent with ANSI/ASME B1.1 D.1 B18.2.1 D.2 and B18.2.6 D.3 have been tested and 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.

RD.2.6 Typical embedment configurations are shown in Fig. RD2.6(a), (b), (c), and (d). These figures also indicate the extent of the embedment within the jurisdiction of this Code.

Fig. RD2.6—(a) Typical embedments for tension loads; (b) typical embedments for compressive loads; (c) typical embedments for shear loads; and (d) typical embedments for combined loads.
**RD.3—General requirements**

**RD.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 D.4 to D.6 discuss nonlinear analysis, using theory of plasticity, for the determination of the strengths of ductile anchor groups.

**RD.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 from 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 satisfactory performance in passing simulated seismic tests. The qualification should be performed consistent with the provisions of this appendix and should be reviewed by a licensed professional engineer experienced in anchor technology. Typical simulated seismic-testing methods are described in Reference D.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. Post-installed anchors qualified to the procedures of ACI 355.2 as Category I for use in cracked concrete are considered acceptable for use in nuclear power plants. 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.

**RD.3.4** The provisions of Appendix D are applicable to normalweight concrete. The design of anchors in heavyweight concrete should be based on testing for the specific heavy weight concrete.

**RD.3.5** A limited number of tests of cast-in-place and post-installed anchors in high-strength concrete indicate that the design procedures contained in this appendix become unconservative, particularly for cast-in anchors, at $f'_c = 11,000$ to $12,000$ psi. Until further tests are available, an upper limit of $f'_c = 10,000$ psi has been imposed in the design of cast-in-place anchors. This is consistent with Chapters 11 and 12. The ACI 355.2 test method does not require testing of post-installed anchors in concrete with $f'_c$ greater than 8000 psi because some post-installed anchors may have difficulty expanding in very high-strength concretes. Because of this, $f'_c$ is limited to 8000 psi in the design of post-installed anchors unless testing is performed.

**RD.3.6.1** The design provisions of ACI 349, Appendix D, 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 yielding 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 reduction in stiffness. For such cases, the design strength is the nominal strength of the steel, multiplied by a strength-reduction factor of 0.75 if load combinations in 9.2 are used or 0.80 if load combinations in Appendix C are used.

The nominal tensile strength of the embedment should be determined based on those portions of the embedment that transmit tension or shear loads into the concrete. 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 strength. This value of 20% is consistent with the value of 20% used in the equation in D.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.

**RD.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 as long as they are stronger than the yield strength of the attachment. This is established with a margin equivalent to that in D.3.6.1. D.3.6.2 is based on attachment yield strength $f_y$, whereas D.3.6.1 uses $f_y$ because attachments are typically of ASTM A 36 material, and the strength is better characterized by the yield strength. The 0.75 factor allows for the actual yield strength versus specified minimum yield strength.

**RD.3.6.3** There are situations where a ductile-failure mode cannot be achieved. It is permissible to design anchors as ductile for one loading but nonductile for the other. 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 D 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.

**RD.3.8** Ductile steel elements are defined in D.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 325D.9 and A 490D.10 anchor materials that have been shown to behave in a ductile manner when used for embedment steel.
RD.3.9 Anchors that incorporate a reduced section (such as threads, notch, or wedge) in the load path (the term “load path” includes the tension load path and the 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 yielding of the unreduced section will occur before failure of the reduced section. Shear failure can be affected significantly by reduced sections within five anchor diameters of the shear plane (many wedge-type anchors). In this case, tests for the evaluation of the shear strength are required. Tests reported in Reference D.4 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.

RD.3.10 The design provisions for impulsive and impactive loads in Appendix F may be used for embedments. Energy can be absorbed by deformation of anchors designed for ductile steel failure.

RD.4—General requirements for strength of anchors

RD.4.1 This section provides requirements for establishing the strength of anchors to concrete. The various types of steel and concrete failure modes for anchors are shown in Fig. RD.4.1(a) and (b). Comprehensive discussions of anchor failure modes are included in References D.11 to D.13. Any model that complies with the requirements of D.4.2 and D.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 methods of D.5.2 and D.6.2 are acceptable. The anchor strength is also dependent on the pullout strength of D.5.3, the side-face blowout strength of D.5.4, and the minimum spacings and edge distances of D.8. The design of anchors for tension recognizes that the strength of anchors is sensitive to appropriate installation; installation requirements are included in D.9. Some post-installed anchors are less sensitive to installation errors and tolerances. This is reflected in varied $\phi$-factors based on the assessment criteria of ACI 355.2.

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 D.4.2. The basic strength cannot be taken greater than the 5% fractile. The number of tests has to be sufficient for statistical validity and should be considered in the determination of the 5% fractile.

RD.4.2 and RD.4.3—D.4.2 and D.4.3 establish the performance factors for which anchor design models are required to be verified. Many possible design approaches exist, and the designer is always permitted to “design by test” using D.4.2 as long as sufficient data are available to verify the model.

RD.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

![Fig. RD.4.1—Failure modes for anchors.](image-url)
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 D.11, D.14, and D.15 provide substantial information on design of such reinforcement. The effect of such supplementary reinforcement is not included in the ACI 355.2 anchor acceptance tests or in the concrete breakout calculation method of D.5.2 and D.6.2. The designer has to rely on other test data and design theories to include the effects of supplementary reinforcement.

For anchors exceeding the limitations of D.4.2.2, or for situations where geometric restrictions limit concrete breakout strength, or both, reinforcement oriented in the direction of load and proportioned to resist the total load within the breakout prism, and fully anchored on both sides of the breakout planes, may be provided instead of calculating concrete breakout strength.

The concrete 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 RD.6.2.1.)

**RD.4.2.2** The method for determining concrete breakout strength included as “considered to satisfy” D.4.2 was developed from the concrete capacity design (CCD) method, D.12, D.13 which was an adaptation of the \( \kappa \) method, D.16, D.17 and is considered to be accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD method predicts the strength of an anchor or group of anchors by using a basic equation for tension, or for shear for a single anchor in cracked concrete, and multiplied by factors that account for the number of anchors, edge distance, spacing, eccentricity and absence of cracking. The limitations on anchor size and embedment length are based on the current range of test data.

The concrete breakout strength calculations are based on a model suggested in the \( \kappa \) method. It is consistent with a breakout prism angle of approximately 35 degrees (Fig. RD.4.2.2(a) and (b)).

**RD.4.4** The \( \phi \)-factors for steel strength are based on using \( f_{ut} \) to determine the nominal strength of the anchor (see D.5.1 and D.6.1) rather than \( f_y \) as used in the design of reinforced concrete members. Although the \( \phi \)-factors for use with \( f_{ut} \) appear low, they result in a level of safety consistent with the use of higher \( \phi \)-factors applied to \( f_y \). The smaller \( \phi \)-factors for shear than for tension do not reflect basic material differences but rather account for the possibility of a non-uniform 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 D.3.6.2). For anchors governed by the more brittle concrete breakout or blowout failure, two conditions are recognized. If supplementary reinforcement is provided to tie the failure prism into the structural member (Condition A), more ductility is present than in the case where such supplementary reinforcement is not present (Condition B). Design of supplementary reinforcement is discussed in RD.4.2.1 and References D.11, D.14, and D.15. Further discussion of strength-reduction factors is presented in RD.4.5.

The strengths of anchors under shear forces are not as sensitive to installation errors and tolerances. Therefore, for shear calculations of all anchors, \( \phi = 0.70 \).

**RD.4.5** As noted in R9.1, the 2006 Code incorporated the load factors of SEI/ASCE 7-02 and the corresponding strength-reduction factors provided in the ACI 318-99 Appendix C into Section 9.2 and 9.3, except that the factor for flexure has been increased. Investigative studies for the \( \phi \)-factors to be used for Appendix D were based on the ACI 349-01 and 9.2 and 9.3 load and strength-reduction factors. The resulting \( \phi \)-factors are presented in D.4.5 for use with the load factors of the 2006 Appendix C. The \( \phi \)-factors for use with the load factors of the ACI 318-99, Appendix C were determined in a manner consistent with the other \( \phi \)-factors of the ACI 318-99 Appendix C. These \( \phi \)-factors are presented in D.4.4 for use with the load factors of 2006 Section 9.2. Since investigative studies for \( \phi \)-factors to be used with Appendix D, for brittle concrete failure modes, were performed for the load and strength-reduction factors now given in Appendix C, the discussion of the selection of these \( \phi \)-factors appears in this section.

Even though the \( \phi \)-factor for plain concrete in Appendix C uses a value of 0.65, the basic factor for brittle concrete failures (\( \phi = 0.75 \)) was chosen based on results of probabilistic
studies\textsuperscript{D,18} that indicated the use of $\phi = 0.65$ with mean values of concrete-controlled failures produced adequate safety levels. Because the nominal resistance expressions used in this appendix and in the test requirements are based on the 5\% fractiles, the $\phi = 0.65$ value would be overly conservative. Comparison with other design procedures and probabilistic studies\textsuperscript{D,18} indicated that the choice of $\phi = 0.75$ was justified. For applications with supplementary reinforcement and more ductile failures (Condition A), the $\phi$-factors are increased. The value of $\phi = 0.85$ is compatible with the level of safety for shear failures in reinforced concrete beams, and has been recommended in the \textit{PCI Design Handbook}\textsuperscript{D,19} and by earlier versions of ACI 349.\textsuperscript{D,20}

\textbf{RD.4.6 Bearing strength}

\textbf{RD.4.6.1} D.4.6.1 prohibits the designer 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 associated with the bearing mode preclude development of the shear-friction mode until after bearing mode failure.\textsuperscript{D,21} As described in RD.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 strength and can therefore be used.

\textbf{RD.4.6.2} For shear lugs, the nominal bearing strength value of $1.3f_y'$ is recommended based on the tests described in Reference D.21 rather than the general provisions of 10.15. The factor of 0.65 corresponds to that used for bearing on concrete in Chapter 9. The factor of 0.70 corresponds to that used for bearing on concrete in Appendix C.

\textbf{RD.5—Design requirements for tensile loading}

\textbf{RD.5.1 Steel strength of anchor in tension}

\textbf{RD.5.1.2} The nominal tensile strength of anchors is best represented by $A_{se}f_{u,ta}$ rather than $A_{se}f_{ya}$ because the large majority of anchor materials do not exhibit a well-defined yield point. The American Institute of Steel Construction (AISC) has based tensile strength of anchors on $A_{se}f_{u,ta}$ since the 1986 edition of their specifications. The use of Eq. (D-3) with Section 9.2 load factors and the $\phi$-factors of D.4.4 give design strengths consistent with the “AISC Load and Resistance Factor Design Specifications for Structural Steel Buildings.”\textsuperscript{D,22}

The limitation of $1.9f_{ya}$ on $f_{u,ta}$ is to ensure that, under service load conditions, the anchor does not exceed $f_{ya}$. The limit on $f_{u,ta}$ of $1.9f_{ya}$ was determined by converting the LRFD provisions to corresponding service load conditions. For Section 9.2, the average load factor of 1.4 (from $1.2D + 1.6L$) divided by the highest $\phi$-factor (0.75 for tension) results in a limit of $f_{u,ta}/f_{ya}$ of 1.55. For consistent results, the service-ability limitation of $f_{u,ta}$ was taken as $1.9f_{ya}$. If the ratio of $f_{u,ta}/f_{ya}$ exceeds this value, the anchoring may be subjected to service loads above $f_{ya}$. Although not a concern for standard structural steel anchors (maximum value of $f_{u,ta}/f_{ya}$ is 1.6 for ASTM A 307\textsuperscript{D,23}), the limitation is applicable to some stainless steels.

\textbf{RD.5.2 Concrete breakout strength of anchors in tension}

\textbf{RD.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 $A_{Ne}/A_{Ne,o}$ and $\psi_{ed,N}$ in Eq. (D-4) and (D-5). Figure RD.5.2.1(a) shows $A_{Ne,o}$ and the development of Eq. (D-6). $A_{Ne}$ is the maximum projected area for a single anchor. Figure RD.5.2.1(b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. Because $A_{Ne}$ is the total projected area for a group of anchors, and $A_{Ne,o}$ is the area for a single anchor, there is no need to include $n$, the number of anchors, in Eq. (D-4) or (D-5). If anchor groups are positioned in such a way that their projected areas overlap, the value of $A_{Ne}$ is required to be reduced accordingly.

\textbf{RD.5.2.2} The basic equation for anchor strength was derived\textsuperscript{D,12-D,14,D,17} assuming a concrete failure prism with an angle of approximately 35 degrees, considering fracture mechanics concepts.

The values of $k_c$ in Eq. (D-7) were determined from a large database of test results in uncracked concrete\textsuperscript{D,12} at the 5\% fractile. The values were adjusted to corresponding $k_c$ values for cracked concrete.\textsuperscript{D,13,D,24} Higher $k_c$ values for post-installed anchors may be permitted, provided they have been determined from product approval testing in accordance with ACI 355.2. For anchors with a deep embedment ($h_{ef} > 11$ in.), test evidence indicates the use of $h_{ef}^{1.5}\textsuperscript{15}$ can be overly conservative for some cases. Often such tests have been with selected aggregates for special applications. An alternative expression (Eq. (D-8)) is provided using $h_{ef}^{53}$ for evaluation of cast-in anchors with 11 in. $\leq h_{ef} < 25$ in. The limit of 25 in. corresponds to the upper range of the test data. This expression can also be appropriate for some undercut post-installed anchors.\textsuperscript{D,4.2} however, should be used with test results to justify such applications.

\textbf{RD.5.2.3} For anchors located less than $1.5h_{ef}$ from three or more edges, the tensile breakout strength computed by the CCD method, which is the basis for Eq. (D-4) to (D-11), gives misleading results.\textsuperscript{D,29} This occurs because the ordinary definition of $A_{Ne}/A_{Ne,o}$ do not correctly reflect the edge effects. This problem is corrected by limiting the value of $h_{ef}$ used in Eq. (D-4) to (D-11) to $c_{a,max}/1.5$, where $c_{a,max}$ is the largest of the influencing edge distances that are less than or equal to the actual $1.5h_{ef}$, but not less than 1/3 of the maximum spacing between anchors for anchor groups. The limit on $h_{ef}$ of not less than 1/3 of the maximum spacing between anchors for anchor groups prevents the designer from using a calculated strength based on individual breakout prisms for a group anchor configuration.

This approach is illustrated in Figure RD.5.2.3. In this example, the proposed limit on the value of $h_{ef}$ to be used in
The critical edge distance for headed studs, headed bolts, expansion anchors, and undercut anchors is $1.5h_{ef}$.

The computations where $h_{ef} = c_{a,max}/1.5$, results in $h_{ef} = h'_{ef} = 4$ in. For this example, this would be the proper value to be used for $h_{ef}$ in computing the resistance even if the actual embedment depth is larger.

The requirement of D.5.2.3 may be visualized by moving the actual concrete breakout surface originating at the actual $h_{ef}$ toward the surface of the concrete perpendicular to the applied tension load. The limit on $h_{ef}$ for use in Eq. (D-4) to...
(D-11) occurs when either the outer boundaries of the failure surface first intersect a free edge or when the intersection of the breakout surface originating between anchors within the group first intersects the surface of the concrete. For the example shown in Fig. RD.5.2.3, Point A shows the controlling intersection.

RD.5.2.4 Figure RD.5.2.4(a) shows a group of anchors that are all in tension but that have 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. RD.5.2.4(b)]. In this case, only the anchors in tension are to be considered in the determination of \( e'_N \). 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.

RD.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 strength of the anchor is further reduced beyond that reflected in \( A_{Ne}/A_{Nea} \). If the smallest side cover distance is greater than 1.5\( h_{ef} \), a complete prism can form and there is no reduction \( (\psi_{ed,N} = 1) \). If the side cover is less than 1.5\( h_{ef} \), the factor \( \psi_{ed,N} \) is required to adjust for the edge effect. \( D.12 \)

RD.5.2.6 The analysis for the determination of crack formation should include the effects of restrained shrinkage (see 7.12.1.2), and should consider all specified load combinations using unfactored loads. The anchor qualification tests of ACI 355.2 require that anchors in cracked concrete zones perform well in a crack that is 0.012 in. wide. If wider cracks are expected, confining reinforcement to control the crack width to approximately 0.012 in. should be provided.

The concrete breakout strengths given by Eq. (D-7) and (D-8) assume cracked concrete (that is, \( \psi_{c,N} = 1.0 \)) with \( \psi_{ed,N} k_c = 24 \) for cast-in-place, and 17 for post-installed (cast-in 40% higher). When the uncracked concrete \( \psi_{c,N} \) factors are applied (1.25 for cast-in, and 1.4 for post-installed), the results are \( \psi_{ed,N} k_c \) factors of 30 for cast-in and 24 for post-installed (25% higher for cast-in). This agrees with field observations and tests that show cast-in anchor strength exceeds that of post-installed for both cracked and uncracked concrete.

When \( k_c \) used in Eq. (D-7) is taken from the ACI 355.2 product evaluation report for post-installed anchors approved for use in both cracked and uncracked concrete, the value of both \( k_c \) and \( \psi_{c,N} \) is based on the ACI 355.2 product evaluation report.

For post-installed anchors approved for use only in uncracked concrete in accordance with ACI 355.2, the value of \( k_c \) in the ACI 355.2 product evaluation report is used in Eq. (D-7), and \( \psi_{c,N} \) shall be 1.0.

RD.5.2.7 The design provisions in D.5 are based on the assumption that the basic concrete breakout strength can be achieved if the minimum edge distance \( c_{amin} \) equals 1.5\( h_{ef} \). However, test results\( D.26 \) indicate that many torque-controlled and displacement-controlled expansion anchors and some undercut anchors require minimum edge distances exceeding 1.5\( h_{ef} \) to achieve the basic concrete breakout strength when tested in uncracked concrete without supple-

mentary reinforcement to control splitting. When a tension load is applied, the resulting tensile stresses at the embedded end of the anchor are added to the tensile stresses induced due to anchor installation, and a splitting failure may occur before reaching the concrete breakout strength defined in D.5.2.1. To account for this potential splitting mode of failure, the basic concrete breakout strength is reduced by a factor \( \psi_{ep,N} \) if \( c_{amin} \) is less than the critical edge distance \( c_{ac} \). If supplementary reinforcement to control splitting is present or if the anchors are located in a region where analysis indicates cracking of the concrete at service loads, then the reduction factor \( \psi_{ep,N} \) is taken as 1.0. The presence of supplementary reinforcement to control splitting does not affect the selection of Condition A or B in D.4.4 or D.4.5.

RD.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.

RD.5.3 Pullout strength of anchor in tension

RD.5.3.2 The pullout strength equations given in D.5.3.4 and D.5.3.5 are only applicable to cast-in headed anchors.\( D.11 \) They are not applicable to expansion and undercut anchors that
use various mechanisms for end anchorage unless the validity of the pullout strength equations are verified by tests.

RD.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.

RD.5.3.4 Equation (D-15) corresponds to the load at which the concrete under the anchor head begins to crush. 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.

RD.5.4 Concrete side-face blowout strength of a headed anchor in tension—The design requirements for side-face blowout are based on the recommendations of Reference D.27. Side-face blowout may control when the anchor is close to an edge (\(c < 0.4d_{ef}\)). 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, and is evaluated by the ACI 355.2 requirements. 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.

RD.6—Design requirements for shear loading

RD.6.1 Steel strength of anchor in shear

RD.6.1.2 The nominal shear strength of anchors is best represented by \(A_{s,ef,\text{uta}}\) for headed stud anchors and \(0.6A_{s,ef,\text{uta}}\) for other anchors rather than a function of \(A_{s,ef,\text{ya}}\) because typical anchor materials do not exhibit a well-defined yield point. The use of Eq. (D-18) and (D-19) with Section 9.2 load factors and the \(\phi\)-factors of D.4.4 give design strengths consistent with the “AISC Load and Resistance Factor Design Specifications for Structural Steel Buildings.”

The limitation of \(1.9f_y\) on \(f_{\text{uta}}\) is to ensure that, under service load conditions, the anchor stress does not exceed \(f_y\). The limit on \(f_{\text{uta}}\) of \(1.9f_y\) was determined by converting the LRFD provisions to corresponding service load conditions as discussed in RD.5.1.2.

RD.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.

RD.6.1.4 The friction force that develops between the base plate and concrete due to the compressive resultant from moment or axial load or both 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.

If the frictional strength is larger than the applied shear force, 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 force. The presence or absence of loads should satisfy Section 9.2.3. Compressive resultsants due to secondary loads should not be considered.

RD.6.2 Concrete breakout strength of anchor in shear

RD.6.2.1 The shear strength equations were developed from the CCD method. They assume a breakout cone angle of approximately 35 degrees (refer to Fig. RD.4.2.2(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 \(A_{Vc}/A_{Vco}\) in Eq. (D-20) and (D-21), and \(\psi_{ec,\text{V}}\) in Eq. (D-21). For anchors far from the edge, D.6.2 usually will not govern. For these cases, D.6.1 and D.6.3 often govern.

Figure RD.6.2.1(a) shows \(A_{Vco}\) and the development of Eq. (D-22), \(A_{Vco}\) 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 RD.6.2.1(b) shows examples of the projected areas for various single anchor and multiple anchor arrangements. \(A_{Vc}\) approximates the full surface area of the breakout cone for the particular arrangement of anchors. Because \(A_{Vc}\) is the total projected area for a group of anchors, and \(A_{Vco}\) is the area for a single anchor, there is no need to include the number of anchors in the equation.

When using Eq. (D-21) for anchor groups loaded in shear, both assumptions for load distribution illustrated in examples on the right side of Fig. RD.6.2.1(b) should be considered because the anchors nearest the edge could fail first or the whole group could fail as a unit with the failure surface originating from the anchors farthest from the edge. 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 this reason, anchors welded to a common plate do not need to consider the failure mode shown in the upper right figure of Fig. RD.6.2.1(b). The PCI Design Handbook approach suggests in Section 6.5.2.2 that the strength 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, D.6.2 allows such a procedure. If the failure surfaces do not intersect, as would generally occur if the anchor spacing \(s\) is equal to or greater than \(1.5e_{\text{ut}}\), then after formation of the near-edge failure surface, the higher strength of the farther anchor would resist most of the load. As shown in the bottom right example in Fig. RD.6.2.1(b), it would be appropriate to consider the full shear strength to be provided by this anchor with its much larger resisting failure surface. No contribution of the anchor near the edge is then considered. Checking the near-edge anchor condition is advisable to preclude undesirable cracking at service load conditions. Further discussion of design for multiple anchors is given in Reference D.11.

For the case of anchors near a corner subjected to a shear load with components normal to each edge, a satisfactory
solution is to check the connection independently for each component of the shear load. 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 D.14.

The detailed provisions of D.6.2.1(a) apply to the case of shear load directed toward an edge. When the shear load is directed away from the edge, the strength will usually be governed by D.6.1 or D.6.3.

The case of shear load parallel to an edge is shown in Fig. RD.6.2.1(c). A special case can arise with shear load parallel to the edge near a corner (refer to Fig. RD.6.2.1(d)). The provisions for shear in the direction of the load should be checked in addition to the parallel-to-edge provisions.

RD.6.2.2 Similar to the concrete breakout tensile strength, the concrete breakout shear capacity does not increase with the failure surface, which is proportional to

![Diagram of anchor group with calculations and notes for shear load at an edge and near a corner.]

**Fig. RD.6.2.1**—(a) Calculation of $A_{Vco}$; (b) Projected area for single anchors and groups of anchors and calculation of $A_{Vc}$; (c) shear force parallel to an edge; and (d) shear force near a corner.
The actual $c_{af} = 12$ in. but two orthogonal edges $c_{af}$ and $h_a$ are $\leq 1.5$ $c_{af}$ therefore the limiting value of $c_{af}$ (shown as $c'_{af}$ in the figure) is the larger of $c_{af,\text{max}}/1.5$, $h_a/1.5$ and one-third of the maximum spacing for an anchor group: $c'_{af} = \max (7/1.5, 8/1.5, 9/3) = 5.33$ in.

Therefore, use $c'_{af} = 5.33$ in. in Eq. (D-21) to (D-28) including the calculation of $A_{af}$:

$$A_{af} = (5 + 9 + 7)(1.5\times5.33) = 168\text{ in.}^2$$

Point A shows the intersection of the assumed failure surface for limiting $c_{af}$ with the concrete surface.

Fig. RD.6.2.4—Shear when anchors are influenced by three or more degrees.

$$(c_{af})^2.$$ Instead, the capacity increases proportionally to $(c_{af})^{1.5}$ due to size effect. The strength is also influenced by the anchor stiffness and the anchor diameter.

The constant, 7, in the shear strength equation was determined from test data reported in Reference D.12 at the 5% fractile adjusted for cracking.

RD.6.2.3 For the special case of cast-in headed bolts continuously welded to an attachment, test data\(^{D.28,D.29}\) show that somewhat higher shear strength 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 D.11, D.14, and D.15.

RD.6.2.4 For anchors influenced by three or more edges where any edge distance is less than $1.5c_{af}$ the shear breakout strength computed by the basic CCD method, which is the basis for Eq. (D-21) through (D-28), gives safe but misleading results. These special cases were studied for the $\kappa$ method\(^{D.17}\) and the problem was pointed out by Lutz.\(^{D.25}\)

Similar to the approach used for tensile breakouts in D.5.2.3, a correct evaluation of the strength is determined if the value of $c_{af}$ to be used in Eq. (D-21) to (D-28) is limited to the maximum of $c_{af,\text{max}}/1.5$ in each direction, $h_a/1.5$, and $1/3$ of the maximum spacing between anchors for anchor groups. The limit on $c_{af}$ of at least $1/3$ of the maximum spacing between anchors for anchor groups prevents the designer using a calculated strength based on individual breakout prisms for a group configuration. This approach is illustrated in Fig. RD.6.2.4. In this example, the proposed limit on the value of $c_{af}$ to be used in the computations where $c_{af} = \text{the largest of } c_{af,\text{max}}/1.5$ in each direction, $h_a/1.5$, results in $c_{af} = 5.33$ in. For this example, this would be the proper value to be used for $c_{af}$ in computing the resistance even if the actual edge distance that the shear is directed toward is larger. The requirement of D.6.2.4 may be visualized by moving the actual concrete breakout surface originating at the actual $c_{af}$ toward the surface of the concrete perpendicular to the applied shear force. The limit on $c_{af}$ for use in Eq. (D-21) to (D-28) occurs when either the outer boundaries on the failure surface first intersect a free edge or when the intersection of the breakout surface originating between anchors within the group first intersects the surface of the concrete. For the example shown in Fig. RD.6.2.4, Point A shows the controlling intersection.

RD.6.2.5 This section provides a modification factor for an eccentric shear force toward an edge on a group of anchors. If the shear force 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 may or may not also cause tension in the anchors, depending on the normal force. Figure RD.6.2.5 defines the term $\psi_{ec,V}$ for calculating the $\psi_{ec,V}$ 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.
RD.6.2.7 Torque-controlled and displacement-controlled expansion anchors are permitted in cracked concrete under pure shear.

RD.6.3 Concrete pryout strength of anchor in shear—Reference D.12 indicates that the pryout shear resistance can be approximated as one to two times the anchor tensile resistance with the lower value appropriate for \( h_{ef} \) less than 2.5 in.

RD.7—Interaction of tensile and shear forces
The shear-tension interaction expression has traditionally been expressed as

\[
\left( \frac{N_{aa}}{N_n} \right)^{1/3} + \left( \frac{V_{aa}}{V_n} \right)^{1/3} \leq 1.0
\]

where \( \varsigma \) varies from 1 to 2. The current trilinear recommendation is a simplification of the expression where \( \varsigma = 5/3 \) (Fig. RD.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 to satisfy D.4.3.

RD.8—Required edge distances, spacings, and thicknesses to preclude splitting failure
The minimum spacings, edge distances, and thicknesses are dependent on the anchor characteristics. Installation forces and torques in post-installed anchors can cause splitting of the surrounding concrete. Such splitting also can be produced in subsequent torquing during connection of attachments to anchors including cast-in anchors. The primary source 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.

RD.8.2 Because the edge cover over a deep embedment close to the edge can have a significant effect on the side-face blowout strength of D.5.4, in addition to the normal concrete cover requirements, the designer may wish to use larger cover to increase the side-face blowout strength.

RD.8.3 Drilling holes for post-installed anchors can cause microcracking. The requirement for a minimum edge distance twice the maximum aggregate size is to minimize the effects of such microcracking.

RD.8.4 Intentionally left blank.

RD.8.5 This minimum thickness requirement is not applicable to through-bolts because they are outside the scope of Appendix D. In addition, splitting failures are caused by the load transfer between the bolt and the concrete. Because through-bolts transfer their load differently than cast-in or expansion and undercut anchors, they would not be subject to the same member thickness requirements. Post-installed anchors should not be embedded deeper than 2/3 of the member thickness.

RD.8.6 The critical edge distance \( c_{ac} \) is determined by the corner test in ACI 355.2. Research has indicated that the corner-test requirements are not met with \( c_{a,min} = 1.5h_{ef} \) for many expansion anchors and some undercut anchors because installation of these types of anchors introduces splitting tensile stresses in the concrete that are increased during load application, potentially resulting in a premature splitting failure. To permit the design of these types of anchors when product-specific information is not available, conservative default values for \( c_{ac} \) are provided.

RD.9—Installation of anchors
Many anchor performance characteristics depend on proper installation of the anchor. Anchor strength and deformations can be assessed by acceptance testing under ACI 355.2. These tests are performed out assuming that the manufacturer’s installation directions will be followed. Certain types of anchors can be sensitive to variations in hole diameter, cleaning conditions, orientation of the axis, magnitude of the installation torque, crack width, and other variables. Some of this sensitivity is indirectly reflected in the assigned \( \phi \)-values for the different anchor categories, which depend in part on the results of the installation safety tests. Gross deviations from the ACI 355.2 acceptance testing results could occur if anchor components are incorrectly exchanged, or if anchor installation criteria and procedures vary from those recommended. Project specifications should require that anchors be installed according to the manufacturer’s recommendations.

RD.10—Structural plates, shapes, and specialty inserts
Design strengths for structural plates, shapes, and specialty inserts are based on the \( \phi \)-values in the AISC-LRFD Steel Manual. The \( \phi \)-value of 0.90 for tension, compression, and bending was established based on SEI/ASCE 7 load combinations. The value of 0.55 for shear is a product of \( \phi = 0.90 \) and \( F_v = 0.6F_y \). For these elements, the same \( \phi \)-factors are used for the load combinations in Section 9.2 and Appendix C of the Code.

RD.11—Shear strength of embedded plates and shear lugs
RD.11.1 Shear lugs—The Code requirements for the design of shear lugs are based on testing reported in
The tests also revealed two distinct response modes:

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 D.4.6. 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. RD.11.1 and a shear stress limited to \( 10\phi K_f f_c' \), with \( \phi \) equal to 0.85.

The anchorage shear strength due to confinement can be taken as \( \phi K_c (N_y - P_a) \), with \( \phi \) 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 anchor. (\( P_a \) is positive for tension and negative for compression). This approach considers the effect of the tension anchors and external loads acting across the initial shear fracture planes (see Fig. RD.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 D.21, 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 0.13\( h \sqrt{f_c'} \); 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 0.13\( h \sqrt{f_c'} \).

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 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 used for determining \( N_{sa} \), 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 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.

**RD.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 (post-bearing 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 D.21.

**References**


D.6. Lotze, D., and Klingner, R.E., “Behavior of Multiple-Anchor Attachments to Concrete from the Perspective of...


APPENDIX RE—THERMAL CONSIDERATIONS

RE.1—Scope

Appendix E gives requirements for the design of concrete nuclear safety-related structures subjected to thermal loading (T_a and T_o).

RE.1.1 Appendix E 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 outdoor atmospheric temperatures that depend on 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 normally 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;
linear temperature distribution for use in design of the section. When the temperature distribution is non-
linear and the section is predicted to crack, the nonlinear
for a rectangular section. The line
variations, as per this appendix, indicate lesser amounts of
sections of this Code should be provided even if the calcu-
lations for the section, and deflections of the structural members.
The gradient temperature distribution effect and the difference
between mean temperature distribution and base temperature.

RE.3—General design requirements

RE.3.1 Figure RE.1 illustrates a technique for considering
the gradient temperature distribution effect and the difference
between mean temperature distribution and base temperature.
The gradient temperature distribution is represented by $\Delta T$
in Fig. RE.1-III, which is acceptable for cracked sections. The difference
between the mean temperature distribution and the base temperature is $T_m - T_b$.

RE.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 E.3.4.

RE.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.

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 members.

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 strength of the member to
support mechanical loading.

In addition, deflection of structural members 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 resolved 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 member based on full
uncracked cross sections. Although fairly easy to
perform, such an analysis may be overly conservative
for moments and forces 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 nonlinear analysis and
iterative solutions not readily available to the designer; 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.

RE.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 SP-27-E.1.

RE.4—Concrete temperatures

The concrete temperatures given in this section are identical to those given by ACI 359. The temperature limits given in E.4.1 and E.4.2 are conservative. Experience with specific types of concrete indicates that higher temperatures may be allowed without loss of significant compressive strength. The observed effects, however, are dependent on the type of cement and aggregate used in the mixture, and adequate test data do not exist to establish higher temperature limits for inclusion in the Code. Available data indicate that the decrease in tensile strength and modulus of elasticity may be greater than the decrease in compressive strength. Therefore, in applications where the tensile strength and modulus of elasticity may be important, closer examination of the test data is recommended.

Because the provisions of E.4.1 and E.4.2 are considered conservative, E.4.3 provides a mechanism by which project-specific acceptance criteria may be developed. Projects where higher temperature limits are needed to assure appropriate structural behavior, and/or to gain regulatory acceptance, may undertake testing programs for their specific concrete design. Testing must simulate the actual, expected operating conditions for the intended use. Such testing must include a sufficient number of specimens so that a pre-established probabilistic acceptance criterion may be satisfied.

It should be noted that temperature limits given in this section relate to concrete surface—not the air temperature, pipe surface temperature, or the temperature at some internal point.

**Fig. RE.1—Temperature distributions.**
In the absence of a measured concrete surface temperature, some extrapolation by a rational method (for example, heat transfer analysis) may be needed if the specified limits are approached. If temperature limits in local areas are exceeded and visual examinations reveal concrete deteriorations, appropriate corrective action should be taken before returning the plant to service.

References

**APPENDIX RF—SPECIAL PROVISIONS FOR IMPULSIVE AND IMPACTIVE EFFECTS**

**RF.1—Scope**

**RF.1.2** While the provisions of this appendix apply to those structural members directly affected by the impactive and impulsive loadings, vibratory effects at points away from the location of impact should also be considered.

**RF.2—Dynamic strength increase**

Because of the rapid strain rates that occur in structural members 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 direct functions of the strain rates involved, as indicated in Table RF.2 and References F.1 and F.2.

Dynamic increase factors given in Table RF.2 are based on tests with specified concrete strengths $f'_c$ of 4000 to 6000 psi and may not be used for high-strength concrete.

The limitation on the DIF is provided in Reference F.3.

**RF.3—Deformation**

**RF.3.1** The ductility ratio is used in conjunction with total deformation consisting of both shear and flexural displacements.

**RF.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 member, during which the load is approximately constant. For example, referring to Fig. RF.3.2, the impulsive loading, which attains a maximum value $F_1$, has the approximately constant value $F_2$ during a time $\Delta t$, where $\Delta t$ is equal to or greater than the fundamental period of the structural member. $R_{m1}$ denotes the resistance required by the impulsive loading with peak value $F_1$ that acts before the time interval $\Delta t$.

Section F.3.2 requires that the minimum available resistance for the impulsive load be the larger of the values $R_{m1}$ and $R_{m2} = 1.2F_2$, and stipulates that this value is applicable to the load combinations that include impulsive loads in Chapter 9 or Appendix C.

This section emphasizes by referencing Section F.8 that the calculation of available resistance or margin in a particular structural element should consider the strength required for other loads that may be acting concurrently with the impulsive load.

**RF.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 F.3.4. It should be noted that the compression reinforcement contributes to the ductility of a structural member, by enabling a large angle-change to take place before general crushing failure of the concrete occurs, thereby increasing the deflection that the structural member 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. In certain cases, however, the position of the neutral axis of a structural member may result in the so-called compression reinforcement being actually in tension when the section reaches its nominal strength. In such cases, the section should be evaluated to determine the effectiveness of the compression reinforcement’s contribution to the ductility of the structural member.

The equation for ductility, $\mu_d = 0.05(\rho - \rho')$, is based on test data given in References F.4 and F.5 and is widely accepted in engineering practice. The coefficient of 0.05 was chosen instead of 0.1 given in Reference F.5 to provide an additional margin of safety against overestimating ductility. However, available data indicate that the 0.05 factor may be too conservative.

---

**Table RF.2—Dynamic increase factors**

<table>
<thead>
<tr>
<th>Material</th>
<th>Dynamic increase factor (DIF)</th>
<th>But not more than</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Reinforcing steel</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grade 40</td>
<td>1.1 + 0.0723 (log$SR + 3.3$)</td>
<td>1.20</td>
</tr>
<tr>
<td>Grade 50</td>
<td>1.05 + 0.08 (log$SR + 3.0$)</td>
<td>1.15</td>
</tr>
<tr>
<td>Grade 60</td>
<td>1.0 + 0.02625 (log$SR + 5.9$)</td>
<td>1.10</td>
</tr>
<tr>
<td>** Prestressing steel**</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Axial and flexural compression</td>
<td>0.9 + 0.1 (log$SR + 5.0$)</td>
<td>1.25</td>
</tr>
<tr>
<td>Shear</td>
<td>$[0.90 + 0.1 (logSR + 5.0)]^{1/2}$</td>
<td>1.10</td>
</tr>
</tbody>
</table>

*where $SR = \text{strain rate, in./in./s, and DIF} \geq 1.0.$*
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 rotation, with an upper limit based on the rotational capacity specified in Section F.3.4.

Reference F.7 presents a rational method for obtaining a conservative estimate of the displacement at ultimate rotation of a reinforced concrete slab subjected to a concentrated load.

It is likely that the upper limit of 10 specified for the case when the permissible ductility ratios are established using the \( \mu_u = 0.05/(\rho - \rho') \) equation is too restrictive for two-way slabs. Therefore, the Code permits the designer, in accordance with Section F.1.3, to use higher limits if sufficient justification can be provided.

**RF.3.4** The rotational capacity \( r_u \) of any yield hinge can be expressed by

\[
r_u = \psi_u D_h \tag{RF-1}
\]

in which

\[
\psi_u = \frac{\varepsilon_u}{c} \tag{RF-2}
\]

Reference F.8, based upon testing simply supported beams with concentrated loads, suggests that the ultimate concrete compressive strain be given by

\[
\varepsilon_u = 0.003 + \frac{0.5}{z} \tag{RF-3}
\]

and the effective plastic hinge zone dimension be given by

\[
D_h = \frac{d}{2} \left[ 1 + \left( 1.14 \sqrt[3]{\frac{z}{d}} - 1 \right) \left( 1 - \frac{q-q'}{q_b} \right) \frac{d}{16.2} \right] \tag{RF-4}
\]

The steel reinforcement indexes are

\[
q = \frac{\rho f_y}{f'_{c}}
\]

\[
q' = \frac{\rho' f_y}{f'_{c}}
\]

All the test data from which Eq. (RF-3) and (RF-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 than 20 in. because 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

\[
\frac{q-q'}{q_b} \leq 0.5 \tag{RF-5}
\]

In this case, it can be shown\(^ {F.8} \) that within practical limits for \( z \) and \( d \), the rotations obtained from Eq. (RF-1) through (RF-4) can be conservatively estimated by

\[
r_u = (0.0065 \frac{d}{c}) \tag{RF-6}
\]

The ultimate rotation results reported in Reference F.8 for beams that satisfy Eq. (RF-5) are conservatively estimated by Eq. (RF-6). The ratio of test results to calculated results has a mean of 1.47 and a standard deviation of 0.49. Equation (RF-6) generally results in rotations in the range from 0.025 to 0.075 radians (1.4 to 4.3 degrees) when applied to beams that satisfy the requirements of Eq. (RF-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. (RF-6) may result in greater rotations.

**RF.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 F.1.4 and F.1.5. Therefore, the upper limit of ductility has been conservatively limited to 3.0 to minimize the permanent deformation due to these loads.

**RF.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\(^ {F.1,F.2,F.5,F.9} \).

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%.

**RF.3.7** This section specifies the ductility ratios for reinforced concrete members 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 resisted 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\(^ {F.1} \) 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 force is resisted by stirrups or bent bars, with the remainder of the shear force being resisted by concrete.
RF.3.8 and RF.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 (refer to Fig. RF.3.8). 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 F.1.

When flexure controls the design, the ductility ratio is to be as specified in Sections F.3.3 or F.3.4. Section F.3.8b defines that a design with axial load less than or equal to \(0.1 f'_c A_g\) or \(1/3\) of that which produces balanced strain conditions, can be considered a flexural failure. The limits of \(0.1 f'_c A_g\), or \(1/3\) that which produces balanced strain conditions, whichever is smaller, represent a magnitude of load below which axial effects on ductility are negligible.

RF.4—Requirements to assure ductility

The provisions to assure ductility are parallel to appropriate sections of Chapter 21 of ACI 318-05.

RF.5—Shear strength

The shear strength for concrete beams and columns is determined in accordance with Sections 11.1 and 11.5 of the main body of this Code, which were evaluated by Joint ACI-ASCE Committee 326 on shear and diagonal tension\(^{F.10}\) 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 318-05 practice. Examples would be checking of reaction shear at supported edges for slabs under local or distributed loads.

The shear strength 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 \(4 \sqrt{f'_c}\) limit taken from ACI 318-05. 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 (diagonal) tension. This criterion was taken from the nuclear containment code.\(^{F.11}\)

The Code recognizes the possible conservatism of the punching shear equation contained in Section 11.11. Therefore, the provisions of Section F.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 (such as References F.12 and F.13) suggesting alternate punching shear relationships for two-way slabs based on the percentage of flexural reinforcement. In particular, Reference F.13 suggests that the punching shear capacity \(P_v\) be taken as the lesser of

\[
P_v = \frac{\rho f_y d^2 (1 - 0.59 \frac{f'_c}{f_y})}{(0.2 - 0.9 \frac{c}{L})} \quad \text{(RF-7)}
\]

or

\[
P_v = \frac{20(c_v + d) d (100 \rho)^{0.25} \sqrt{f'_c}}{0.75 + 4 \frac{c_v}{L_v}} \quad \text{(RF-8)}
\]

where \(c_v = \sqrt{A_p}\) where \(A_p\) is the area over which the \(P_v\) is applied; and \(L_v\) may be taken as infinity in most impulse and impact cases.\(^{F.13}\) For impactive and impulsive loads, the dynamic increase factors (DIFs) of Section F.2 should be used with Eq. (RF-7) and (RF-8) and the results of these equations should be reduced by the appropriate \(\phi\)-factor. With these modifications, Eq. (RF-7) and (RF-8) can be substituted for the shear provisions of this Code for those specific situations where these relationships can be shown to be applicable.

RF.6—Impulsive effects

Three methods are identified as being acceptable for the 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 structure 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 F.5, F.14, and F.15, and summarized briefly as follows.

The equivalent mass $M_e$, load $F_e$, and elastic stiffness $K_e$ are determined on the basis of an assumed deformed shape function $\phi(x,y)$ for the structure as follows

$$M_e = \int \int m \phi^2(x,y) dx dy$$

$$F_e = \int \int p \phi^2(x,y) dx dy$$

$$K_e = \frac{F}{F_t}$$

where

$m$ = mass per unit area;
$p$ = $p(x,y) = \text{pressure}$;
$F_t$ = resultant force; and
$K$ = value of $F_t$ to cause unit deflection at point of application of resultant force.

The $\phi(x,y)$ 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 load been applied statically. Exceptions may occur for very fundamental mode shape or as the deformed shape had the

RF.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 member. 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 member opposite to the face of impact.
- **Scabbing**—Ejection of material from the back face of the impacted structural member opposite to the face of impact.
- **Spalling**—Ejection of material from the front face of the impacted structural member (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 instead of scabbing to define the ejection of materials from the back face). The previous definitions, however, are consistently used in this Code.

If a structural member must act as a missile barrier, then it is necessary that the member be sufficiently thick so as to prevent perforation and the provisions of Section F.7.2.1 must be met. However, if the structural member is not required to stop the missile and local perforation is permissible and does not impair the required function of the structural member, then the provisions of Section F.7.2.1 are not mandatory.

The provisions of Section F.7.2.1 do not preclude scabbing of concrete off the rear face of the structural member. 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. 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 member, 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 the use of 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, F.20 the Bechtel formulas, F.21 and the Stone and Webster formulas F.22 appear to be in reasonable agreement with the available published pertinent test dataF.23-F.25 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 F.20 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 suggestedF.19-F.21 for accounting for missile deformability.

Test data in the range of interest are rapidly becoming available.F.22-F.25 Sufficient data, however, are not available to adequately define the degree of scatter on perforation or scabbing thickness. For higher missile velocities, however, 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 or she 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 formulas, the range of parameters used in the tests should be considered.

**RF.8—Impactive and impulsive loads**

In cases of impulsive and impactive loading where a structural member is expected to deform beyond its elastic limits, the usefulness of load combination equations presented in Chapter 9 and Appendix C 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 and live load, which may be present as the structural member deforms beyond its effective yield point (corresponding to $X_y$ [Fig. RF.8]).

If the energy balance method is used, only the energy represented by Area A in Fig. RF.8, 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

$$\mu' = \frac{X_m - X_x}{X_y - X_x} = \frac{\mu_d X_y - X_x}{X_y - X_x}$$

where $\mu_d$ 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 $R_m - R_s$.

Instead of a more rigorous analysis, seismic forces can be conservatively treated as equivalent static loads in the analysis for determining the adequacy of the structural member for the impactive and impulsive loading.

**References**


The changes summarized in this section reflect only the technical changes from ACI 349-01 to ACI 349-06. The editorial changes are not identified due to the numerous changes required to update this Code to the ACI 318-05. Major format changes included moving notation summaries at the beginning of each chapter to Chapter 2.

CHAPTER 1—GENERAL REQUIREMENTS
Technical revisions made to Chapter 1 of the 2006 Code and Commentary:
- The minimum specified concrete compressive strength was added per ACI 318-05: Revision to 1.1.1.
- Provisions excluding piles, piers, and soil-supported slabs were removed. Revision to 1.1.5 and 1.1.6.
- 10CFR830 was included to address Department of Energy facilities. Revision to 1.5.
- New subsection was introduced to express the committee’s concern that using high-strength concrete may require the user to verify the applicability of equations/formulations. Revision to R1.1.1.
- New subsection was introduced to provide additional guidance for designing slabs on ground; a reference to PTI was included. Revision to R1.1.6.
- Clarification was added to address a USNRC concern on inspector’s qualifications. Revision to R1.3.
- Reference to ANSI/ANS-6.4 was added. Revision to R1.4.
- Reference to ANSI/ASME NQA-2 was deleted as it is no longer applicable. Revision to R1.5.

CHAPTER 2—NOTATION AND DEFINITIONS
Technical revisions made to Chapter 2 of the 2006 Code and Commentary:
- In accordance with the revised format of ACI 318-05, all notation from various chapters and appendixes were moved to Chapter 2. The notation section was revised to eliminate multiple definitions for a single variable, and a notation protocol was established and implemented. Renumbered 2.1 to 2.2. New section 2.1 and R2.1.
- Various definitions were revised to conform to ACI 318-05. New definitions unique to the nuclear industry were revised and added (see below) to agree with regulatory nomenclature.
  - Authority Having Jurisdiction replaced Regulatory Authority.
  - Design basis tornado (DBT); specification, construction; specification, design; and specification, technical were added. Revision to 2.2.

CHAPTER 3—MATERIALS
Technical revisions made to Chapter 3 of the 2006 Code and Commentary:
- The following sections were revised to conform to ACI 318-05: 3.2.1, 3.3.2, 3.5.1, 3.5.3.2, 3.5.4.1, and 3.5.6.1.
- Added requirements for epoxy-coated steel wire and welded wire reinforcement. Revision to 3.5.3.8.

CHAPTER 4—DURABILITY REQUIREMENTS
Technical revisions made to Chapter 4 of the 2006 Code:
- The following sections were revised to conform to ACI 318-05: 4.1.1, Table 4.2.2, Table 4.2.3, Table 4.3.1, R4.4, and R4.4.2.
- The type of member was clarified. Revision to Table 4.4.1.
- References were updated. Revision to R4—References.

CHAPTER 5—CONCRETE QUALITY, MIXING, AND PLACING
Technical revisions made to Chapter 5 of the 2006 Code:
- The following sections were revised to conform to ACI 318-05: 5.1.1, 5.2.3, 5.3.2.1, Table 5.3.2.1, Table 5.3.2.2, 5.4.1, 5.6.3.3, 5.6.5.1, 5.6.5.2, 5.6.5.3, and R5.6.5.
- Section 5.6.1 was added to conform to ACI 318-05. Renumbered the existing sections in 5.6 accordingly.
- The frequency of testing changed to address the regulations of the U.S. Nuclear Regulatory Commission in their Regulatory Guide 1.142, Rev. 2. Revision to 5.6.2.1.
- References were updated. Revision to R5—References.

CHAPTER 7—DETAILS OF REINFORCEMENT
Technical revisions made to Chapter 7 of the 2006 Code:
- The following sections were revised to conform to ACI 318-05: 7.1.4, 7.4, 7.5.2.2, 7.6.7, 7.7.1, 7.7.2, 7.7.3, 7.7.5.1, 7.7.7, 7.10.4.5, 7.10.5.6, 7.13.2.1, 7.13.2.2, and 7.13.2.3.
- References were updated. Revision to R7—References.

CHAPTER 8—ANALYSIS AND DESIGN—GENERAL CONSIDERATIONS
Technical revisions made to Chapter 8 of the 2006 Code:
- The following sections were revised to conform to ACI 318-05: 8.1.3, 8.3.3, 8.3.4, 8.4.1, 8.4.3, and 8.11.6.
- Appendix B of ACI 318-05 was not adopted; 8.1.2 was intentionally left blank. Revision to 8.1.2.
- The use of burned clay or concrete tile fillers was not adopted; 8.11.5 was intentionally left blank. Revision to 8.11.5.
- All references to the working stress design method have been deleted. R8.1 was deleted. Revision to R8.3.
- Section R8.6 no longer applies; section deleted.

CHAPTER 9—STRENGTH AND SERVICEABILITY REQUIREMENTS
Technical revisions made to Chapter 9 of the 2006 Code and Commentary:
- The following sections were revised to conform to ACI 318-05: 9.1.6, 9.2.8, and 9.3.
- \( L_p, R, \) and \( S \) introduced new terms to agree with ACI 318-05 and load \( C_{cr} \) added to distinguish it from \( L \). Revision to 2.1 and 9.1.1.1.
• “Crane loads” was deleted as it is separately defined. Revision 9.1.3 and R9.1.

• The load combinations were revised to reflect changes in ASCE 7-02 and ACI 318-05. The number of load combinations was reduced to 9: Eq. (9-1) to (9-5) are similar to ACI 318-05; and Eq. (9-6) to (9-9) were modified to reflect conditions unique to nuclear facilities. Revision to 9.2.1 and R9.2.

• Updated the referenced load combinations, according to the revision of 9.2.1. Revision to 9.2.2, 9.2.5, 9.2.6, 9.2.7, 9.5.1.3, and 9.5.2.4.

• The paragraph was modified to accommodate fractional load factors, keeping the basic concept the same. Revision to 9.2.3.

• Crane load is treated as a separate load; \( L \) was revised to \( C_{cr} \). Revision to 9.2.4.

• Crane load need not be combined with SSE (DBE). Revision to 9.2.9.

• Because of the change in \( \phi \)-factor for shear and torsion from 0.85 to 0.75, a 10% reduction in SSE (DBE) load was allowed for future nuclear facilities. Revision to 9.2.10.

• Because thick squat shearwalls are prevalent in nuclear facilities, the committee decided to delete 9.3.4; section deleted.

• A clarification was added for crane load, \( C_{cr} \), and temperature load, \( T_o \). Revision to R9.

• An introductory sentence was added concerning the creation of the new Appendix C; discussion was added to clarify the load factor for \( R_p \); and the reference to “ongoing industry efforts” was deleted as it is historical and no longer pertinent. Revision to R9.1.

• New paragraphs explaining the types of load categories were added. Revisions to R9.1.1.1, R9.1.1.2, R9.1.1.3, and R9.1.1.4.

• New section was added explaining the change in \( \phi \)-factors. Revision to R9.3.

• The provision of Section 9.4 of ACI 318-05 does not apply for nuclear structures; explanation has been provided. Revision to R9.4.

• References were updated. Revision to R9—References.

CHAPTER 10—FLEXURE AND AXIAL LOAD
Technical revisions made to Chapter 10 of the 2006 Code and Commentary:
• The following sections were revised to conform to ACI 318-05: 10.2.2, 10.3.3, 10.3.4, 10.5, 10.6.4, 10.6.7, 10.7, 10.12.2, and 10.15.3.

• Section 10.3.5 was added to conform to ACI 318-05. Renumbered the remaining sections in 10.3 accordingly.

• References were updated.

CHAPTER 11—SHEAR AND TORSION
Technical revisions made to Chapter 11 of the 2006 Code and Commentary:
• The following sections were revised to conform to ACI 318-05: 11.1.1, 11.1.2.1, 11.1.3, 11.3.3, 11.4, 11.5, 11.6, 11.8, 11.9.1, 11.9.3, 11.10.1, 11.10.6, 11.10.9, 11.12.2.2, 11.12.3, 11.12.4, and 11.12.6.

CHAPTER 12—DEVELOPMENT AND SPLICES OF REINFORCEMENT
Technical revisions made to Chapter 12 of the 2006 Code:
• The following sections were revised to conform to ACI 318-05: 12.2, 12.3, 12.5, 12.9, 12.10.5, 12.11.3, 12.11.4, 12.13, 12.14.3.

• Mechanical splice was revised to develop specified tensile strength. Revision to 12.14.3.2 and R12.14.3.2.

• Provisions for the qualification of mechanical splices in ACI 349-01, 12.14.3.4.1 are deleted; section deleted.

• Welded splice was revised to develop specified tensile and compression strength. Revision to 12.14.3.4.

• Splices not meeting the requirements of 12.14.3.2 or 12.14.3.4 are not permitted. Revision to 12.14.3.5, R12.14.3.5, and 12.15.4.

• The calculation for factored load stress in a bar was deleted. Stagger revised to 30 in., similar to 12.5.5. Revision to 12.14.3.7 and R12.14.3.7.

CHAPTER 13—TWO-WAY SLAB SYSTEMS
Technical revisions made to Chapter 13 of the 2006 Code and Commentary:
• The following sections were revised to conform to ACI 318-05: 13.3.7, 13.3.8, 13.6.1.6, and 13.6.4.2.

CHAPTER 14—WALLS
Technical revisions made to Chapter 14 of the 2006 Code:
• The following section was revised to conform to ACI 318-05: 14.2.2.

• Cover requirement was aligned with Code provisions in Chapter 7. Revision to 14.3.4.

• Added new section on alternative design of slender walls. Added 14.8.

CHAPTER 15—FOOTINGS
Technical revisions made to Chapter 15 of the 2006 Code and Commentary:
• The following sections were revised to conform to ACI 318-05: 15.4.4.2, 15.5.1, 15.5.3, 15.5.4, 15.8.3, and 15.8.3.3.

CHAPTER 16—PRECAST CONCRETE
Technical revisions made to Chapter 16 of the 2006 Code and Commentary:
• The following section was revised to conform to ACI 318-05: 16.6.2.3.

CHAPTER 17—COMPOSITE CONCRETE
Technical revisions made to Chapter 17 of the 2006 Code and Commentary:
• The following sections were revised to conform to ACI 318-05: 17.2.6, 17.5.3, 17.6.1, and 17.6.2.

• Section 17.5.2 was added to conform to ACI 318-05. Renumbered the existing sections in 17.5 accordingly.
CHAPTER 18—PRESTRESSED CONCRETE
Technical revisions made to Chapter 18 of the 2006 Code and Commentary:
- The following sections were revised to conform to ACI 318-05: 18.1.3, 18.1.4, 18.2.2, 18.2.5, 18.3.2, 18.3.3, 18.3.4, 18.3.5, 18.4.2, 18.4.4, 18.6.1, 18.6.2, 18.7.2, 18.8, 18.9.2.2, 18.9.3, 18.9.4, 18.10.1, 18.10.2, 18.10.4, 18.11.1, 18.12.2, 18.12.3, 18.13, 18.16, and 18.17.
- Sections 18.14 and 18.15 have been left blank to keep section numbering consistent with ACI 318-05. The remaining sections were renumbered accordingly.
- Added new section on external post-tensioning. Added 18.22.

CHAPTER 19—SHELLS
Technical revisions made to Chapter 19 of the 2006 Code and Commentary:
- The following section was revised to conform to ACI 318-05: 19.2.8.

CHAPTER 20—STRENGTH EVALUATION OF EXISTING STRUCTURES
Technical revisions made to Chapter 20 of the 2006 Code and Commentary:
- The limits on the increase to strength reduction factors were removed. The section was changed to require the engineer to determine the appropriate factors for the evaluation. Revision to 20.2.5, R20.2.5, 20.4.2, and R20.4.2.
- References were updated. Revision to R20—References.

CHAPTER 21—PROVISIONS FOR SEISMIC DESIGN
Technical revisions made to Chapter 21 of the 2006 Code and Commentary:
- The content and format of Chapter 21 of ACI 349-06 was substantially revised to follow that of Chapter 21 of ACI 318-05. The following changes state the technical exceptions to new information provided from ACI 318-05.
- Text on welded splices from ACI 318-05 was modified. Revision to 21.2.6 and 21.2.7.
- The strength multiplier was increased from (6/5) to (7/5). Revision to 21.4.2.2.
- Section 21.4.2.3 was deleted; section removed.
- Sections 21.6 was not adopted; left blank. Revision to 21.6.
- Multiplier of 1.25 was decreased to 1.10. Revision to 21.7.2.3.
- Boundary elements are not required for walls in which the extreme fiber strains are small: squat walls. Revision to 21.7.6.1.
- A limiting strain of 0.002 is used as the boundary-element-strain threshold. Revision to 21.7.6.2.
- Section 21.7.6.3 was not adopted; left blank. Revision to 21.7.6.3.
- Limits were reduced. Revision to 21.7.6.5.
- Section 21.8 was not adopted; left blank. Revision to 21.8.
- Sections 21.11, 21.12, and 21.13 of ACI 318-05 were not adopted.
- The committee decided to write a complete commentary for this chapter. This commentary is based on the commentary to Chapter 21 of ACI 318-05.

APPENDIX A—STRUT-AND-TIE MODELS
Technical revisions made to Appendix A of the 2006 Code and Commentary:
- This is a new Appendix that matches the new Appendix in ACI 318-05, Strut-and-Tie Models. Revision to Appendix A.

APPENDIX B—INTENTIONALLY LEFT BLANK
Technical revisions made to Appendix B of the 2006 Code and Commentary:
- Appendix B from ACI 318-05 was not adopted; left blank. Revision to Appendix B.

APPENDIX C—ALTERNATIVE LOAD AND STRENGTH-REDUCTION FACTORS
Technical revisions made to Appendix C of the 2006 Code and Commentary:
- This is a new appendix to the Code and Commentary. Appendix C from ACI 318-05 was adopted. Revision to Appendix C.
- Section C.2 presents the load factors, $\phi$-factors, and load combinations of ACI 349-01, Chapter 9, and a separate term for the crane loading, $C_{cr}$, was introduced.
- Provisions C.2.8 and C.2.9 were added from ACI 318-05.
- Sections C.3.4 and C.3.5 were not adopted from ACI 318-05.

APPENDIX D—ANCHORING TO CONCRETE
Technical revisions made to Appendix D of the 2006 Code and Commentary:
- Appendix D was previously Appendix B.
- Added reference to D.12. Revision to D.2.2.
- Added reference to Appendix C. Revision to D.3.2.
- Clarification. Revision to D.3.3.
- Clarification. Revision to D.3.6.1.
- Clarification. Revision to D.6.1.4.
- Clarification. Revision to D.10.1 and D.10.2.
- Clarification. Revision to D.11.3.
- References were updated. Revision to RD—References.

APPENDIX E—THERMAL CONSIDERATIONS
Technical revisions made to Appendix E of the 2006 Code and Commentary:
- Appendix E was previously Appendix A.
- Existing definitions were revised and new definitions were added: short term, long term, local, elevated or differential temperature, and local area. Revisions to E.2.
- Added a new section on temperature limitations. Added E.4 and RE.4.
APPENDIX F—SPECIAL PROVISIONS FOR IMPULSIVE AND IMPACTIVE EFFECTS

Technical revisions made to Appendix F of the 2006 Code and Commentary:

- Appendix F was previously Appendix C.
- Added limiting statement on DIF. Revision to F.2.1 and RF.2.
- References were updated. Revision to RF—References.

APPENDIX G—SI METRIC EQUIVALENT OF U.S. CUSTOMARY UNITS

Technical revisions made to Appendix G of the 2006 Code and Commentary:

- Appendix G was previously Appendix D.
- The metric equivalents have been updated to agree with ACI 318M-05.
As ACI begins its second century of advancing concrete knowledge, its original chartered purpose remains “to provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading knowledge.” In keeping with this purpose, ACI supports the following activities:

· Technical committees that produce consensus reports, guides, specifications, and codes.

· Spring and fall conventions to facilitate the work of its committees.

· Educational seminars that disseminate reliable information on concrete.

· Certification programs for personnel employed within the concrete industry.

· Student programs such as scholarships, internships, and competitions.

· Sponsoring and co-sponsoring international conferences and symposia.

· Formal coordination with several international concrete related societies.

· Periodicals: the *ACI Structural Journal* and the *ACI Materials Journal*, and *Concrete International*.

Benefits of membership include a subscription to *Concrete International* and to an ACI Journal. ACI members receive discounts of up to 40% on all ACI products and services, including documents, seminars and convention registration fees.

As a member of ACI, you join thousands of practitioners and professionals worldwide who share a commitment to maintain the highest industry standards for concrete technology, construction, and practices. In addition, ACI chapters provide opportunities for interaction of professionals and practitioners at a local level.

American Concrete Institute  
38800 Country Club Drive  
Farmington Hills, MI 48331  
U.S.A.  
Phone: 248-848-3700  
Fax: 248-848-3701  
www.concrete.org
The AMERICAN CONCRETE INSTITUTE

was founded in 1904 as a nonprofit membership organization dedicated to public service and representing the user interest in the field of concrete. ACI gathers and distributes information on the improvement of design, construction and maintenance of concrete products and structures. The work of ACI is conducted by individual ACI members and through volunteer committees composed of both members and non-members.

The committees, as well as ACI as a whole, operate under a consensus format, which assures all participants the right to have their views considered. Committee activities include the development of building codes and specifications; analysis of research and development results; presentation of construction and repair techniques; and education.

Individuals interested in the activities of ACI are encouraged to become a member. There are no educational or employment requirements. ACI's membership is composed of engineers, architects, scientists, contractors, educators, and representatives from a variety of companies and organizations.

Members are encouraged to participate in committee activities that relate to their specific areas of interest. For more information, contact ACI.

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