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Building Code Requirements for Structural Concrete Reinforced with Glass Fiber-Reinforced Polymer (GFRP) Bars—Code and Commentary

Reported by ACI Committee 440

ACI CODE-440.11-22

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Building Code Requirements for Structural Concrete Reinforced with Glass Fiber-Reinforced Polymer (GFRP) Bars—Code and Commentary

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An ACI Standard

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This Code was developed by an ANSI-approved consensus process and addresses structural systems, members, and connections, including cast-in-place, precast, nonprestressed, and composite construction. The "Building Code Requirements for Structural Concrete Reinforced with Glass Fiber-Reinforced Polymer (GFRP) Bars" ("Code") provides minimum requirements for the materials, design, and detailing of structural concrete buildings and, where applicable, nonbuilding structures reinforced with GFRP bars that conform to the requirements of ASTM D7957-22. Among the subjects covered are: design and construction for strength, serviceability, and durability; load combinations, load factors, and strength reduction factors; structural analysis methods; deflection limits; development and splicing of reinforcement; construction document information; field inspection and testing; and methods to evaluate the strength of existing structures.

Keywords: admixtures; aggregates; beam-column frame; beams (supports); cements; columns (supports); combined stress; composite construction (concrete to concrete); compressive strength; concrete; construction documents; continuity (structural); cover; curing; deflections; durability; flexural strength; floors; footings; formwork (construction); GFRP reinforcement; inspection; joints (junctions); joists; load tests (structural); loads (forces); mixture proportioning; modulus of elasticity; moments; piles; placing; precast concrete; quality control; reinforced concrete; roofs; serviceability; shear strength; spans; splicing; strength analysis; stresses; structural analysis; structural design; structural integrity; structural walls; T-beams; torsion; walls; water.

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PREFACE

This Code was developed by an ANSI-approved consensus process and addresses structural systems, members, and connections, including cast-in-place, precast, nonprestressed, and composite construction. The "Building Code Requirements for Structural Concrete Reinforced with Glass Fiber-Reinforced Polymer (GFRP) Bars" ("Code") provides minimum requirements for the materials, design, and detailing of structural concrete buildings and, where applicable, nonbuilding structures reinforced with GFRP bars that conform to the requirements of ASTM D7957-22, "Standard Specification for Solid Round Glass Fiber Reinforced Polymer Bars for Concrete Reinforcement." Among the subjects covered are: design and construction for strength, serviceability, and durability; load combinations, load factors, and strength reduction factors; structural analysis methods; deflection limits; development and splicing of reinforcement; construction document information; field inspection and testing; and methods to evaluate the strength of existing structures.

This Code covers the design of reinforced concrete members that are reinforced entirely with GFRP; the design of "hybrid" members with mixed types of reinforcement is outside the scope of this Code. However, this Code can be used for the design of GFRP-reinforced concrete members that are part of a structure that also includes members that are not reinforced with GFRP. In such a situation, this Code would cover the design of the GFRP-reinforced concrete members, and other suitable standards (such as ACI 318) would cover the design of other types of members in the structure. This Code covers the design of GFRP-reinforced concrete members in a structure assigned to Seismic Design Category (SDC) A. This Code also covers the design of GFRP-reinforced concrete members not designated as part of the seismic-force-resisting system in Seismic Design Categories B and C. This initial version of the Code, which has been developed from the body of GFRP-reinforced concrete research that has been published over the past 30 years, does not cover GFRP-reinforced concrete members in any structure assigned to Seismic Design Categories D, E, and F although subsequent editions of this Code are expected to incorporate additional SDCs as further research becomes available. Other topics that are not addressed in this version of the Code but are expected to be covered in subsequent editions include prestressed construction, lightweight concrete, shotcrete, connections of precast members, diaphragms, deep beams, drilled piers and caissons, brackets and corbels, methods for designing discontinuity regions using strut-and-tie theory where section-based methods do not apply, shear friction, and anchoring to concrete.

This Code is dependent on ACI 318-19 and adheres to the chapter and section numbering of ACI 318-19, with the exception of Chapter 15 in which language and numbering is dependent on ACI 318M-14. This Code does not include several chapters that are addressed in ACI 318-19, specifically Chapter 12: Diaphragms, Chapter 17: Anchoring to Concrete, Chapter 18: Earthquake-Resistant Structures, and Chapter 23: Strut-and-Tie Method. These chapters have been identified as "Not Addressed" in this version of the Code, but are expected to be included in future versions of this Code as additional research becomes available. This Code also does not include Chapter 14: Plain Concrete from ACI 318-19 which has been identified as "Not Applicable" because it is not related to design with GFRP reinforcement and is not expected to be included in future versions of this Code, as ACI 318 is the applicable standard. Within chapters, the terms "out of scope" and "not applicable" are used for numbered section headings from ACI 318-19 that are not covered by this Code, while the term "intentionally left blank" is used as a place holder to maintain consistency with section numbering in situations where ACI 318-19 includes a numbered provision that is not also in this Code.

For ease of use, language in common with ACI 318 has been reproduced in this document. Provisions that are identical to ACI 318-19 are denoted with an equal sign ("="). Accordingly, this Code follows the organizational philosophy of ACI 318, which is to present all design and detailing requirements for structural systems or for individual members in chapters devoted to those individual subjects, and to arrange the chapters in a manner that generally follows the process and chronology of design and construction. Information and procedures that are common to the design of multiple members are located in utility chapters.

Uses of the Code include adoption by reference in a general building code. The Code is written in a format that allows such reference without change to its language. Therefore, background details or suggestions for carrying out the requirements or intent of the Code provisions cannot be included within the Code itself. The Commentary is provided for this purpose. This Code can supplement a current International Code Council (ICC) building code, supplement the codes governing new and existing structures of a local jurisdiction authority, or act as a stand-alone code in a locality that has not adopted an existing building code.

Some considerations of the committee in developing the Code are discussed in the Commentary, with emphasis given to the explanation of differences in design between GFRP-reinforced concrete and steel-reinforced concrete. For example, GFRP bars do not yield; rather, they are linear elastic until failure. Design procedures in this Code account for this difference from the traditional steel-reinforced concrete design procedures adopted in ACI 318, and approach design from the perspective of deformability (the ability of a member to undergo large displacements prior to failure) rather than from the steel-reinforced concrete design focus on ductility. Consequently, this Code permits GFRP-reinforced concrete flexural members to have either tension-controlled or compression-controlled failure modes.

Furthermore, GFRP bars possess high tensile strength only in the direction of the reinforcing fibers, which affects shear strength, dowel action, and bond performance; thus, design equations for shear strength and development length are necessarily different from the equations used for steel reinforcement in ACI 318, although the design procedures themselves are similar. Other significant differences from ACI 318 occur in serviceability design for deflection and crack control, as the stiffness of GFRP reinforcement can be as small as one-fourth that of steel reinforcement. Because the mechanical and bond properties of

GFRP bars are more negatively impacted at elevated temperatures than are steel bars, and reports from ASTM E119 fire tests on GFRP-reinforced concrete members are not yet available, this Code is only applicable where fire-resistance ratings are not required or where approved by the building official under the alternative means and methods provisions of 1.10.1. Recommendations for increasing the fire resistance of GFRP-reinforced concrete members have been included in the Commentary. Much of the research data referenced in the Commentary is cited for the user desiring greater detail on this subject. Other documents that provide suggestions for carrying out the requirements of the Code are also cited in the Commentary.

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CODE

CHAPTER 1—GENERAL

1.1—Scope of ACI CODE-440.11-22

=**1.1.1** This chapter addresses (a) through (h):

(a) General requirements of this Code
(b) Purpose of this Code
(c) Applicability of this Code
(d) Interpretation of this Code
(e) Definition and role of the building official and the licensed design professional
(f) Construction documents
(g) Testing and inspection
(h) Approval of special systems of design, construction, or alternative construction materials

1.2—General

1.2.1 ACI CODE-440.11, "Building Code Requirements for Structural Concrete Reinforced with Glass Fiber-Reinforced Polymer (GFRP) Bars," is hereafter referred to as "this Code."

=**1.2.2** In this Code, the general building code refers to the building code adopted in a jurisdiction. When adopted, this Code forms part of the general building code.

=**1.2.3** The official version of this Code is the English language version, using inch-pound units, published by the American Concrete Institute.

=**1.2.4** In case of conflict between the official version of this Code and other versions of this Code, the official version governs.

1.2.5 This Code provides minimum requirements for the materials, design, construction, and strength evaluation of GFRP-reinforced concrete members and systems in any structure designed and constructed under the requirements of the general building code.

=**1.2.6** Modifications to this Code that are adopted by a particular jurisdiction are part of the laws of that jurisdiction, but are not a part of this Code.

=**1.2.7** If no general building code is adopted, this Code provides minimum requirements for the materials, design,

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CHAPTER R1—GENERAL

R1.1—Scope of ACI CODE-440.11-22

R1.1.1 This Code includes provisions for the design of nonprestressed glass fiber-reinforced polymer (GFRP)-reinforced concrete used for structural purposes. This Code does not address concrete prestressed with GFRP. This Code does not cover any applications of steel reinforcement of concrete. The design of structural concrete reinforced with steel is governed by ACI 318. This Code covers the design of reinforced concrete members that are reinforced entirely with GFRP; the design of "hybrid" members with mixed types of reinforcement is outside the scope of this Code. However, this Code can be used for the design of GFRP-reinforced concrete members that are part of a structure that also includes members that are not reinforced with GFRP. Steel reinforcement may be present in GFRP-reinforced concrete members designed using this Code, but the steel reinforcement should not be considered as part of the reinforcement for that member for the purposes of strength or serviceability calculations.

This Code is a dependent code on ACI 318-19. This chapter includes a number of provisions that explain where this Code applies and how it is to be interpreted.

R1.2—General

=**R1.2.2** The American Concrete Institute recommends that this Code be adopted in its entirety.

R1.2.3 Committee 440 develops the Code in English, using inch-pound units. Based on that version, Committee 440 approved a version in English using SI units.

=**R1.2.5** This Code provides minimum requirements and exceeding these minimum requirements is not a violation of the Code.

The licensed design professional may specify project requirements that exceed the minimum requirements of this Code.

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construction, and strength evaluation of members and systems in any structure within the scope of this Code.

1.3—Purpose

1.3.1 The purpose of this Code is to provide for public health and safety by establishing minimum requirements for strength, stability, serviceability, durability, and integrity of GFRP-reinforced concrete structures.

=**1.3.2** This Code does not address all design considerations.

=**1.3.3** Construction means and methods are not addressed in this Code.

1.4—Applicability

1.4.1 This Code shall apply to GFRP-reinforced concrete structures designed and constructed under the requirements of the general building code.

=**1.4.2** Provisions of this Code shall be permitted to be used for the assessment, repair, and rehabilitation of existing structures.

=**1.4.3** Applicable provisions of this Code shall be permitted to be used for structures not governed by the general building code.

1.4.4 Intentionally left blank.

=**1.4.5** This Code shall apply to the design of slabs cast on stay-in-place, noncomposite steel decks.

1.4.6 Intentionally left blank.

1.4.7 This Code does not apply to the design and installation of concrete piles, drilled piers, and caissons embedded in ground, except as provided in (a) and (b):

(a) For portions of deep foundation members in air or water, or in soil incapable of providing adequate lateral restraint to prevent buckling throughout their length

(b) For precast concrete piles supporting structures assigned to Seismic Design Categories A and B

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R1.3—Purpose

R1.3.1 This Code provides a means of establishing minimum requirements for the design and construction of GFRP-reinforced concrete, as well as for acceptance of design and construction of GFRP-reinforced concrete structures by the building officials or their designated representatives.

This Code does not provide a comprehensive statement of all duties of all parties to a contract or all requirements of a contract for a project constructed under this Code.

=**R1.3.2** The minimum requirements in this Code do not replace sound professional judgment or the licensed design professional's knowledge of the specific factors surrounding a project, its design, the project site, and other specific or unusual circumstances to the project.

R1.4—Applicability

=**R1.4.2** Specific provisions for assessment, repair, and rehabilitation of existing concrete structures are provided in ACI 562-19. Existing structures in ACI 562 are defined as structures that are complete and permitted for use.

R1.4.3 Structures such as underground utility structures and sea walls involve design and construction requirements that are not specifically addressed by this Code. Many Code provisions, however, may be applicable for these structures if approved by the authority having jurisdiction.

R1.4.5 In its most basic application, the noncomposite steel deck serves as a form, and the concrete slab is designed to resist all loads, while in other applications the concrete slab may be designed to resist only the superimposed loads.

R1.4.7 The design and installation of concrete piles fully embedded in the ground is regulated by the general building code.

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1.4.8 This Code does not apply to design and construction of slabs-on-ground unless the slab transmits vertical loads from other portions of the structure to the soil.

=**1.4.9** This Code does not apply to the design and construction of tanks and reservoirs.

1.4.10 This Code does not apply to composite design slabs cast on stay-in-place composite steel deck.

1.4.11 This Code does not apply to the design and construction of GFRP-reinforced lightweight concrete members and systems.

1.4.12 This Code does not apply to the design and construction of concrete prestressed with GFRP.

1.4.13 This Code does not apply to the design and construction of members for structures classified as Seismic Design Categories D through F, or to the design and construction of members that are part of the lateral-load-resisting system for structures classified as Seismic Design Categories B or C.

1.5—Interpretation

=**1.5.1** The principles of interpretation in this section shall apply to this Code as a whole unless otherwise stated.

=**1.5.2** This Code consists of chapters including text, headings, tables, figures, footnotes to tables and figures, and referenced standards.

=**1.5.3** The Commentary consists of a preface, introduction, commentary text, tables, figures, and cited publications. The Commentary is intended to provide contextual information, but is not part of this Code, does not provide binding requirements, and shall not be used to create a conflict with or ambiguity in this Code.

=**1.5.4** This Code shall be interpreted in a manner that avoids conflict between or among its provisions. Specific provisions shall govern over general provisions.

=**1.5.5** This Code shall be interpreted and applied in accordance with the plain meaning of the words and terms used. Specific definitions of words and terms in this Code shall be used where provided and applicable, regardless of whether other materials, standards, or resources outside of this Code provide a different definition.

1.5.6 The following words and terms in this Code shall be interpreted in accordance with (a) through (i):

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R1.4.8 ACI 360R presents information on the design of steel-reinforced concrete slabs-on-ground, primarily industrial floors and the slabs adjacent to them. Information from ACI 360R can be used in conjunction with the guidelines found in ACI 440.1R for the design of slabs-on-ground that do not transmit vertical loads or from other portions of the structure to the soil.

R1.4.11 Lightweight concrete has been excluded due to a lack of experimental data on the behavior of GFRP-reinforced concrete members made with lightweight concrete.

R1.5—Interpretation

=**R1.5.4** General provisions are broad statements, such as a building needs to be serviceable. Specific provisions, such as explicit reinforcement distribution requirements for crack control, govern over the general provisions.

=**R1.5.5** ACI Concrete Terminology (CT-21) is the primary resource to help determine the meaning of words or terms that are not defined in the Code. Dictionaries and other reference materials commonly used by licensed design professionals may be used as secondary resources.

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(a) The word "shall" is always mandatory.
(b) Provisions of this Code are mandatory even if the word "shall" is not used.
(c) Words used in the present tense shall include the future.
(d) The word "and" indicates that all of the connected items, conditions, requirements, or events shall apply.
(e) The word "or" indicates that the connected items, conditions, requirements, or events are alternatives, at least one of which shall be satisfied.
(f) The term "Not addressed" refers to chapters that are not included in this version of the Code but are expected to be included in future versions of this Code.
(g) The term "Not applicable" refers to chapters and section headings in ACI 318 that are not related to design with GFRP reinforcement.
(h) The term "Out of scope" refers to numbered section headings from ACI 318 that are not covered by this Code.
(i) The term "Intentionally left blank" is used as a place holder to maintain consistency with section numbering in ACI 318.

=**1.5.7** In any case in which one or more provisions of this Code are declared by a court or tribunal to be invalid, that ruling shall not affect the validity of the remaining provisions of this Code, which are severable. The ruling of a court or tribunal shall be effective only in that court's jurisdiction, and shall not affect the content or interpretation of this Code in other jurisdictions.

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=**R1.5.7** This Code addresses numerous requirements that can be implemented fully without modification if other requirements in this Code are determined to be invalid. This severability requirement is intended to preserve this Code and allow it to be implemented to the extent possible following legal decisions affecting one or more of its provisions.

=**1.5.8** If conflicts occur between provisions of this Code and those of standards and documents referenced in Chapter 3, this Code shall apply.

1.6—Building official

=**1.6.1** All references in this Code to the building official shall be understood to mean persons who administer and enforce this Code.

=**1.6.2** Actions and decisions by the building official affect only the specific jurisdiction and do not change this Code.

=**1.6.3** The building official shall have the right to order testing of any materials used in concrete construction to determine if materials are of the quality specified.

R1.6—Building official

=**R1.6.1** Building official is defined in 2.3.

=**R1.6.2** Only the American Concrete Institute has the authority to alter or amend this Code.

1.7—Licensed design professional

=**1.7.1** All references in this Code to the licensed design professional shall be understood to mean the engineer in either 1.7.1.1 or 1.7.1.2.

=**1.7.1.1** The licensed design professional responsible for, and in charge of, the structural design work.

=**1.7.1.2** A specialty engineer to whom a specific portion of the structural design work has been delegated subject to the conditions of (a) and (b).

R1.7—Licensed design professional

=**R1.7.1** Licensed design professional is defined in 2.3.

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(a) The authority of the specialty engineer shall be explicitly limited to the delegated design work.

(b) The portion of design work delegated shall be well defined such that responsibilities and obligations of the parties are apparent.

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R1.7.1.2(b) A portion of the design work may be delegated to a specialty engineer during the design phase or to the contractor in the construction documents. An example of design work delegated to a specialty engineer or contractor is precast concrete.

1.8—Construction documents and design records

=**1.8.1** The licensed design professional shall provide in the construction documents the information required in Chapter 26 and that required by the jurisdiction.

=**1.8.2** Calculations pertinent to design shall be filed with the construction documents if required by the building official. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. Model analysis shall be permitted to supplement calculations.

R1.8—Construction documents and design records

=**R1.8.1** The provisions of Chapter 26 for preparing project drawings and specifications are, in general, consistent with those of most general building codes. Additional information may be required by the building official.

=**R1.8.2** Documented computer output is acceptable instead of manual calculations. The extent of input and output information required will vary according to the specific requirements of individual building officials. However, if a computer program has been used, only skeleton data should normally be required. This should consist of sufficient input and output data and other information to allow the building official to perform a detailed review and make comparisons using another program or manual calculations. Input data should be identified as to member designation, applied loads, and span lengths. The related output data should include member designation and the shears, moments, and reactions at key points in the span. For column design, it is desirable to include moment magnification factors in the output where applicable.

The Code permits model analysis to be used to supplement structural analysis and design calculations. Documentation of the model analysis should be provided with the related calculations. Model analysis should be performed by an individual having experience in this technique.

1.9—Testing and inspection

=**1.9.1** Concrete materials shall be tested in accordance with the requirements of Chapter 26.

=**1.9.2** Concrete construction shall be inspected in accordance with the general building code and in accordance with Chapter 26.

=**1.9.3** Inspection records shall include information required in Chapter 26.

1.10—Approval of special systems of design, construction, or alternative construction materials

=**1.10.1** Sponsors of any system of design, construction, or alternative construction materials 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 building official or to a board of examiners appointed by the building offi-

R1.10—Approval of special systems of design, construction, or alternative construction materials

R1.10.1 New methods of design, new materials, and new uses of materials should undergo a period of development before being covered in a code. Hence, good systems or components might be excluded from use by implication if means were not available to obtain acceptance.

For special systems considered under this section, specific tests, load factors, deflection limits, and other pertinent

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cial. This board shall be composed of competent engineers and shall have authority to investigate the data so submitted, require tests, and formulate rules governing design and construction of such systems to meet the intent of this Code. These rules, when approved by the building official and promulgated, shall be of the same force and effect as the provisions of this Code.

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requirements should be set by the board of examiners and should be consistent with the intent of the Code.

The provisions of this section do not apply to model tests used to supplement calculations under 1.8.2 or to strength evaluation of existing structures under Chapter 27.

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CHAPTER 2—NOTATION AND TERMINOLOGY

2.1—Scope

2.1.1 This chapter defines notation and terminology used in this Code.

2.2—Notation

A_1 = loaded area for consideration of bearing strength, mm^2

A_2 = area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having its upper base equal to the loaded area. The sides of the pyramid, cone, or tapered wedge shall be sloped one vertical to two horizontal, mm^2

A_{ch} = cross-sectional area of a member measured to the outside edges of transverse reinforcement, mm^2

A_{cp} = area enclosed by outside perimeter of concrete cross section, mm^2

A_f = area of GFRP longitudinal tension reinforcement, mm^2

$A_{f,min}$ = minimum area of GFRP flexural reinforcement, mm^2

$A_{f,provided}$ = provided area of GFRP reinforcement to resist flexure, mm^2

$A_{f,required}$ = required area of GFRP reinforcement to resist flexure, mm^2

$A_{f\ell}$ = total area of GFRP longitudinal reinforcement to resist torsion, mm^2

$A_{f\ell,min}$ = minimum area of GFRP longitudinal reinforcement to resist torsion, mm^2

A_{ft} = area of one leg of a closed GFRP stirrup, or tie resisting torsion within spacing s, mm^2

A_{fv} = area of GFRP shear reinforcement within spacing s, mm^2

$A_{fv,min}$ = minimum area of GFRP shear reinforcement within spacing s, mm^2

A_{fw} = total area of GFRP longitudinal reinforcement, mm^2

A_g = gross area of concrete section, mm^2. For a hollow section, A_g is the area of the concrete only and does not include the area of the void(s)

A_o = gross area enclosed by torsional shear flow path, mm^2

A_{oh} = area enclosed by centerline of the outermost closed transverse torsional reinforcement, mm^2

a = depth of equivalent rectangular stress block, mm

B_n = nominal bearing strength, N

COMMENTARY

CHAPTER R2—NOTATION AND TERMINOLOGY

R2.2—Notation

A_b = nominal area of an individual bar, mm^2

A_{fb} = total area of GFRP longitudinal tension reinforcement for which the design rupture strain in the extreme tension layer of the GFRP longitudinal reinforcement occurs simultaneously with crushing of the concrete in the extreme compression fiber of the cross section, mm^2

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B_u = factored bearing load, N
b = dimension of the column section perpedicular to the direction under consideration, mm
b_1 = dimension of the critical section b_o measured in the direction of the span for which moments are determined, mm
b_2 = dimension of the critical section b_o measured in the direction perpendicular to b_1, mm
b_f = effective flange width, mm
b_o = perimeter of critical section for two-way shear in slabs and footings, mm
b_{slab} = effective slab width, mm
b_t = width of that part of cross section containing the closed stirrups resisting torsion, mm
b_v = width of cross section at contact surface being investigated for horizontal shear, mm
b_w = web width or diameter of circular section, mm
C_E = environmental reduction factor
C_m = factor relating actual moment diagram to an equivalent uniform moment diagram
c = distance from extreme compression fiber to neutral axis, mm
c_1 = dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm
c_2 = dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to c_1, mm
c_b = lesser of: (a) the distance from center of a bar to nearest concrete surface, and (b) one-half the center-to-center spacing of bars being developed, mm
c_c = clear cover of reinforcement, mm
D = effect of service dead load
D_s = effect of superimposed dead load
D_w = effect of self-weight dead load of the concrete structural system
d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm
d_{agg} = nominal maximum size of coarse aggregate, mm
d_b = nominal diameter of bar, mm
d_c = thickness of concrete cover measured from extreme tension fiber to center of bar location closest thereto, mm
d_{pile} = diameter of pile at footing base, mm
E = effect of horizontal and vertical earthquake-induced forces
E_c = modulus of elasticity of concrete, MPa
E_{cb} = modulus of elasticity of beam concrete, MPa
E_{cs} = modulus of elasticity of slab concrete, MPa

COMMENTARY

b = width of compression face of member, mm
c_{bal} = distance from extreme compression fiber to neutral axis at the balanced condition, mm
d_{pa} = depth of drop panel or insulation protecting GFRP reinforcement from fire, mm

CODE

COMMENTARY

E_f = modulus of elasticity of GFRP reinforcement, MPa

EI = flexural stiffness of member, mm^2-N

$(EI)_{eff}$ = effective flexural stiffness of member, mm^2-N

F = effect of service load due to fluids with well-defined pressures and maximum heights

f_c' = specified compressive strength of concrete, MPa

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

f_f = tensile stress in the GFRP reinforcement at factored moment M_u, MPa

f_{fb} = design tensile strength of bent portion of GFRP reinforcement, MPa; see 20.2.2.4

f_{fb}^* = guaranteed tensile strength of bent portion of GFRP reinforcement, MPa

f_{fd} = tensile design strength in the GFRP longitudinal reinforcement for columns corresponding to a strain of 0.01, MPa

f_{fr} = tensile stress in the GFRP reinforcement required to develop the full nominal sectional capacity, MPa

f_{fs} = tensile stress in GFRP reinforcement at service loads, MPa

$f_{fs,sus}$ = tensile stress in GFRP longitudinal reinforcement due to sustained service loads, MPa

f_{ft} = design tensile strength of GFRP transverse reinforcement, MPa; see 20.2.2.6

f_{fu} = design tensile strength of GFRP longitudinal reinforcement, MPa; see 20.2.2.3

f_{fu}^* = guaranteed tensile strength of GFRP longitudinal reinforcement, MPa

f_r = modulus of rupture of concrete, MPa

H = effect of service load due to lateral earth pressure, ground water pressure, or pressure of bulk materials, N

h = overall thickness, height, or depth of member, mm

I = moment of inertia of section about centroidal axis, mm^4

I_b = moment of inertia of gross section of beam about centroidal axis, mm^4

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

I_e = effective moment of inertia for calculation of deflection, mm^4

I_{e+} = effective moment of inertia at location of maximum positive moment for calculation of deflection, mm^4

I_{e1-} = effective moment of inertia at location of maximum negative moment at the near end of the span for calculation of deflection, mm^4

I_{e2-} = effective moment of inertia at location of maximum negative moment at the far end of the span for calculation of deflection, mm^4

I_f = moment of inertia of GFRP reinforcement about centroidal axis of member cross section, mm^4

CODE

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

I_s = moment of inertia of gross section of slab about centroidal axis, mm^4

k = effective length factor for compression members

k_b = bond-dependent coefficient

k_{cr} = ratio of the depth of the elastic cracked section neutral axis to the effective depth

L = effect of service live load

L_r = effect of service roof live load

ℓ = span length of beam or one-way slab; clear projection of cantilever, mm

ℓ_1 = length of span in direction that moments are being determined, measured center-to-center of supports, mm

ℓ_2 = length of span in direction perpendicular to ℓ_1, measured center-to-center of supports, mm

ℓ_a = additional embedment length beyond centerline of support or point of inflection, mm

ℓ_c = length of compression member, measured center-to-center of the joints, mm

ℓ_d = development length in tension of bar, mm

ℓ_{dc} = development length in compression of bars, mm

ℓ_{dh} = development length in tension of bar with a standard hook, measured from outside end of hook, point of tangency, toward critical section, mm

ℓ_{ext} = straight extension at the end of a standard hook, mm

ℓ_n = length of clear span measured face-to-face of supports, mm

ℓ_{st} = tension lap splice length, mm

ℓ_u = unsupported length of column or wall, mm

ℓ_w = length of entire wall, or length of wall segment or wall pier considered in direction of shear force, mm

M_1 = lesser factored end moment on a compression member, N-mm

M_{1ns} = factored end moment on a compression member at the end at which M_1 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm

M_{1s} = factored end moment on compression member at the end at which M_1 acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm

COMMENTARY

K_t = torsional stiffness of member; moment per unit rotation

$k_{cr,rect}$ = ratio of depth of elastic cracked section neutral axis to the effective depth for a rectangular cross section

ℓ_{in} = length of insulated area, mm

ℓ_{pa} = length of GFRP reinforcement protected from fire, mm

ℓ_{un} = length of GFRP reinforcement at anchorage not exposed to fire, mm

CODE

M_2 = greater factored end moment on a compression member. If transverse loading occurs between supports, M_2 is taken as the largest moment occurring in member. Value of M_2 is always positive, N-mm

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

M_{2ns} = factored end moment on compression member at the end at which M_2 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm

M_{2s} = factored end moment on compression member at the end at which M_2 acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, N-mm

M_a = maximum moment in member due to service loads at stage deflection is calculated, N-mm

M_c = factored moment amplified for the effects of member curvature used for design of compression member, N-mm

M_{cr} = cracking moment, N-mm

M_n = nominal flexural strength at section, N-mm

M_s = moment due to total service loads, N-mm

M_{sc} = factored slab moment that is resisted by the column at a joint, N-mm

$M_{s,sus}$ = moment due to sustained service loads, N-mm

M_u = factored moment at section, N-mm

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, N

P_c = critical buckling load, N

P_n = nominal axial compressive strength of member, N

$P_{n,max}$ = maximum nominal axial compressive strength of a member, N

P_{nt} = nominal axial tensile strength of member, N

$P_{nt,max}$ = maximum nominal axial tensile strength of member, N

P_o = nominal axial strength at zero eccentricity, N

P_u = factored axial force; to be taken as positive for compression and negative for tension, N

p_{cp} = outside perimeter of concrete cross section, mm

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

Q = stability index for a story

q_u = factored load per unit area, N/m^2

R = cumulative load effect of service rain load

r = radius of gyration of cross section, mm

S = effect of service snow load

COMMENTARY

n_f = ratio of modulus of elasticity of GFRP bars to modulus of elasticity of concrete

$P\delta$ = secondary moment due to individual member slenderness, N-mm

$P\Delta$ = secondary moment due to lateral deflection, N-mm

CODE

S_n = nominal moment, shear, axial, torsional, or bearing strength

s = center-to-center spacing of items, such as longitudinal reinforcement or transverse reinforcement, mm

s_w = clear distance between adjacent webs, mm

T = cumulative effects of service temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete

T_{cr} = cracking torsional moment, N-mm

T_n = nominal torsional moment strength, N-mm

T_t = total test load, N

T_{th} = threshold torsional moment, N-mm

T_u = factored torsional moment at section, N-mm

t_f = thickness of flange, mm

U = strength of a member or cross section required to resist factored loads or related internal moments and forces in such combinations as stipulated in this Code

V_c = nominal shear strength provided by concrete, N

V_f = nominal shear strength provided by GFRP shear reinforcement, N

V_n = nominal shear strength, N

V_{nh} = nominal horizontal shear strength, N

V_u = factored shear force at section, N

V_{us} = factored horizontal shear in a story, N

v_c = stress corresponding to nominal two-way shear strength provided by concrete, MPa

v_n = equivalent concrete stress corresponding to nominal two-way shear strength of slab or footing, MPa

v_u = maximum factored two-way shear stress calculated around the perimeter of a given critical section, MPa

v_{uv} = factored shear stress on the slab critical section for two-way action due to gravity loads without moment transfer, MPa

W = effect of wind load

w_c = density, unit weight, of normalweight concrete, kg/m^3

w_u = factored load per unit length of beam or one-way slab, N/mm

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to tension face, mm

α_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

β = ratio of long to short dimensions: clear spans for two-way slabs, sides of column, concentrated load or reaction area; or sides of a footing

β_1 = factor relating depth of equivalent rectangular compressive stress block to depth of neutral axis

β_b = ratio of area of reinforcement cut off to total area of tension reinforcement at section

COMMENTARY

t = wall thickness of hollow section, mm

CODE

β_{cr} = ratio of distance from elastic cracked section neutral axis to extreme tension fiber to distance from elastic cracked section neutral axis to centroid of tensile reinforcement

β_{dns} = ratio used to account for reduction of stiffness of columns due to sustained axial loads

β_{ds} = ratio of maximum factored sustained shear within a story to the maximum factored shear in that story associated with the same load combination

γ = parameter to account for the variation in stiffness along the length of the flexural member

γ_f = factor used to determine the fraction of M_{sc} transferred by slab flexure at slab-column connections

γ_s = factor used to determine the portion of reinforcement located in center band of footing

γ_v = factor used to determine the fraction of M_{sc} transferred by eccentricity of shear at slab-column connections

Δ_1 = maximum deflection, during first load test, measured 24 hours after application of the full test load, mm

Δ_2 = maximum deflection, during second load test, measured 24 hours after application of the full test load. Deflection is measured relative to the position of the structure at the beginning of the second load test, mm

Δ_o = relative lateral deflection between the top and bottom of a story due to V_{us}, mm

δ = moment magnification factor used to reflect effects of member curvature between ends of a compression member

δ_s = moment magnification factor used for frames not braced against sidesway, to reflect lateral drift resulting from lateral and gravity loads

ε_{ft} = net tensile strain in extreme layer of GFRP longitudinal tension reinforcement at nominal strength, excluding strains due to creep, shrinkage, and temperature

ε_{fu} = design rupture strain of GFRP reinforcement; see 20.2.2.5

ε_{fu}^* = guaranteed rupture strain of GFRP longitudinal reinforcement

ϕ = strength reduction factor

λ_s = factor used to modify shear strength based on the effects of member depth, commonly referred to as the size effect factor

λ_Δ = multiplier used for additional deflection due to long-term effects

ξ = time-dependent factor for sustained load

ρ_f = ratio of A_f to bd

COMMENTARY

ε_c = strain in concrete at extreme compression fiber

ε_{cu} = maximum usable strain at extreme concrete compression fiber

ϕ_K = stiffness reduction factor

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CODE

ρ_{fb} = GFRP reinforcement ratio producing balanced strain conditions

$\rho_{f\ell}$ = ratio of area of distributed GFRP longitudinal reinforcement to gross concrete area perpendicular to that reinforcement

ρ_{ft} = ratio of area of distributed GFRP transverse reinforcement to gross concrete area perpendicular to that reinforcement

ρ_s = ratio of volume of spiral reinforcement to total volume of core confined by the spiral, measured out-to-out of spirals

ψ_t = factor used to modify development length for casting location in tension

COMMENTARY

ρ_{fv} = ratio of the area of GFRP shear reinforcement to the product of web width and GFRP shear reinforcement spacing

θ = angle between compression diagonal and the tension chord of the members

2.3—Terminology

admixture—material other than water, aggregate, cementitious materials, and fiber reinforcement used as an ingredient, which is added to grout, mortar, or concrete, either before or during its mixing to modify the freshly mixed, setting, or hardened properties of the mixture.

aggregate—granular material, such as sand, gravel, crushed stone, and iron blast-furnace slag, or recycled aggregates including crushed hydraulic cement concrete, used with a cementing medium to form concrete or mortar.

alternative cement—an inorganic cement that can be used as a complete replacement for portland cement or blended hydraulic cement, and that is not covered by applicable specifications for portland or blended hydraulic cements.

anchor—an element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads to the concrete.

anchor bolt— a bolt or stud, headed or threaded, cast in place, grouted in place, or drilled and fastened into existing concrete either by expansion or by chemical adhesives.

anchorage length—length over which force is transferred by bond stress between the GFRP reinforcement and the concrete.

area of GFRP—the nominal cross-sectional area of the GFRP reinforcement calculated using the nominal bar diameter.

balanced moment—moment capacity at simultaneous crushing of concrete and rupture of the extreme layer of GFRP tension reinforcement.

balanced reinforcement—an amount and distribution of GFRP reinforcement in a flexural member such that in strength design, the extreme layer of GFRP tensile reinforce-

R2.3—Terminology

ACI Concrete Terminology (CT-21) is the primary resource for the meaning of terms that are not defined for general use in this Code.

aggregate—The definition of recycled materials in ASTM C33 is very broad and is likely to include materials that would not be expected to meet the intent of the provisions of this Code for use in structural concrete. Use of recycled aggregates, including crushed hydraulic-cement concrete in structural concrete, requires additional precautions. Refer to 26.4.1.2.1(c).

alternative cements—alternative cements are described in the references listed in R26.4.1.1.1(b). Refer to 26.4.1.1.1(b) for precautions when using these materials in concrete covered by this Code.

CODE

ment reaches its design rupture strain simultaneously with the concrete in compression reaching its assumed crushing strain of 0.003.

bar—a long, slender structural element used to reinforce concrete. In this Code, bars are normally composed of GFRP.

beam—member subjected primarily to flexure and shear, with or without axial force or torsion; beams in a moment frame that forms part of the lateral-force-resisting system are predominantly horizontal members; a girder is a beam.

bent bar—a GFRP reinforcing bar factory formed to a prescribed bent shape. (See also **hook**, **hooked bar**, **stirrup**, and **tie**.)

boundary element—portion along wall and diaphragm edge, including edges of openings, strengthened by longitudinal and transverse reinforcement.

building official—term used to identify the authority having jurisdiction or individual charged with administration and enforcement of provisions of the building code. Such terms as building commissioner or building inspector are variations of the title, and the term "building official" as used in this Code, is intended to include those variations, as well as others that are used in the same sense.

cementitious materials—materials that have cementing value if used in grout, mortar, or concrete, including portland cement, blended hydraulic cements, expansive cement, fly ash, raw or calcined natural pozzolans, slag cement, and silica fume, but excluding alternative cements.

collector—element that acts in axial tension or compression to transmit forces between a diaphragm and a vertical element of the lateral-force-resisting system.

column—member, usually vertical or predominantly vertical, used primarily to support axial compressive load, but that can also resist moment, shear, or torsion. Columns used as part of a lateral-force-resisting system resist combined axial load, moment, and shear. See also **moment frame**.

column capital—enlargement of the top of a concrete column located directly below the slab or drop panel that is cast monolithically with the column.

compliance requirements—construction-related code requirements directed to the contractor to be incorporated into construction documents by the licensed design professional, as applicable.

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

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

compression-controlled strain limit—net tensile strain of $\mathbf{0.8}\varepsilon_{fu}$ which corresponds to a GFRP reinforcement ratio of $\mathbf{1.4}\rho_{fb}$.

COMMENTARY

bent bar—GFRP bars are not field bent.

cementitious materials—Cementitious materials permitted for use in this Code are addressed in 26.4.1.1. Fly ash, raw or calcined natural pozzolan, slag cement, and silica fume are considered supplementary cementitious materials.

compliance requirements—Although primarily directed to the contractor, the compliance requirements are also commonly used by others involved with the project.

CODE

concrete—mixture of portland cement or any other cementitious material, fine aggregate, coarse aggregate, and water, with or without admixtures.

concrete, lightweight—concrete containing lightweight aggregate and having an equilibrium density, as determined by ASTM C567, between 1440 and 1840 kg/m^3.

concrete, nonprestressed—reinforced concrete with at least the minimum amount of nonprestressed GFRP reinforcement and no prestressed reinforcement.

concrete, normalweight—concrete containing only coarse and fine aggregates that conform to ASTM C33 and having a density greater than 2160 kg/m^3.

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

concrete, prestressed—reinforced concrete in which internal stresses have been introduced by prestressed reinforcement to reduce potential tensile stresses in concrete resulting from loads, and for two-way slabs, with at least the minimum amount of prestressed reinforcement.

concrete, reinforced—structural concrete reinforced with at least the minimum amount of nonprestressed reinforcement, prestressed reinforcement, or both, as specified in this Code.

concrete, steel-reinforced—structural concrete reinforced entirely with steel reinforcement conforming to requirements of Chapter 20 in ACI 318-19.

concrete strength, specified compressive (f_c')—compressive strength of concrete used in design and evaluated in accordance with provisions of this Code, MPa; wherever the quantity f_c' is under a radical sign, the square root of numerical value only is intended, and the result has units of MPa.

connection—region of a structure that joins two or more members; a connection also refers to a region that joins members of which one or more is precast.

construction documents—written and graphic documents and specifications prepared or assembled for describing the location, design, materials, and physical characteristics of the elements of a project necessary for obtaining a building permit and construction of the project.

continuous closed stirrup—stirrup manufactured by wrapping continuous, wet fibers around a jig or mandrel to form the desired closed shape and eliminate the need for lapped open stirrups or hooks.

continuous closed tie—tie manufactured by wrapping continuous, wet fibers around a jig or mandrel to form the desired closed shape and eliminate the need for lapped open ties.

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.

cover, specified concrete—distance between the outermost surface of embedded reinforcement and the closest outer surface of the concrete.

creep rupture—breakage of a material under sustained loading at stresses less than the tensile strength.

COMMENTARY

concrete, normalweight—normalweight concrete typically has a density (unit weight) between 2160 and 2560 kg/m^3 and is normally taken as 2320 and 2400 kg/m^3.

CODE

crosstie—a continuous GFRP reinforcing bar having standard hooks with a bend not less than 90 degrees with at least a $\mathbf{12}d_b$ extension at both ends. The hooks shall engage peripheral longitudinal bars.

cutoff point—point where reinforcement is terminated.

deformability—the ability of a member to undergo large displacements prior to failure.

design information—project-specific information to be incorporated into construction documents by the licensed design professional, as applicable.

design load combination—combination of factored loads and forces.

development length—length of embedded reinforcement required to develop the design strength of reinforcement at a critical section.

discontinuity—abrupt change in geometry or loading.

dowel—(1) a reinforcing bar intended to transmit tension, compression, or shear through a construction joint; (2) smooth bar acting as a load transfer device between concrete slabs where expansion and contraction movement along the main dowel axis is not inhibited.

drop panel—projection below the slab used to reduce the amount of negative reinforcement over a column or the minimum required slab thickness, and to increase the slab shear strength.

durability—ability of a structure or member to resist deterioration that impairs performance or limits service life of the structure in the relevant environment considered in design.

effective depth of section—distance measured from extreme compression fiber to centroid of longitudinal tension reinforcement.

effective stiffness—stiffness of a structural member accounting for cracking, creep, and other nonlinear effects.

embedment length—length of embedded reinforcement provided beyond a critical section.

embedments—items embedded in concrete, excluding reinforcement as defined in Chapter 20. Reinforcement or anchors connected to the embedded item to develop the strength of the assembly, are considered part of the embedment.

embedments, pipe—embedded pipes, conduits, and sleeves.

environmental reduction factor—a factor applied to guaranteed GFRP reinforcing bar material properties in design equations to account for potential change in material properties resulting from exposure to the concrete environment.

extreme tension reinforcement—layer of reinforcement that is the farthest from the extreme compression fiber.

fiber content—the amount of fiber present in a composite.

fiber roving—parallel bundle of continuous fibers with little or no twist.

COMMENTARY

fiber content is typically measured by the fiber volume fraction, which is the ratio of the volume of the fibers to the volume of the composite; alternatively, fiber content can also be measured by the fiber weight fraction, which is the ratio of the weight of the fibers to the weight of the composite.

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fiber-reinforced polymer (FRP) bar—composite material formed into a long, slender structural shape suitable for the internal reinforcement of concrete and consisting primarily of longitudinal unidirectional fibers bound and shaped by a rigid polymer resin material.

finite element analysis—a numerical modeling technique in which a structure is divided into a number of discrete elements for analysis.

fire-protected embdement zone—in fire protection, the region at the end of the member that is not directly exposed to fire or is protected from exposure to fire by additional cover concrete or externally applied insulation.

GFRP-reinforced concrete—concrete in which GFRP reinforcement is used as internal reinforcement.

GFRP reinforcement—glass fiber reinforced polymer reinforcement meeting the requirements of ASTM D7957.

GFRP structural profile—structural GFRP shape of constant cross section, manufactured by the pultrusion process.

head—a separate piece of any shape firmly attached to the end of a bar, or a protuberance of the bar itself at the end, used to anchor the reinforcing bar in concrete.

headed GFRP bars—GFRP reinforcing bars with heads attached at one or both ends.

helical wrapping—a surface treatment for GFRP reinforcing bars consisting of a glass roving or other fiber, which is applied by a stationary winding operation as the GFRP reinforcing bar is simultaneously pulled in the longitudinal direction during manufacture. (See also **surface enhancement**).

insert—anything other than reinforcement that is rigidly positioned within a concrete form for permanent embedment in the hardened concrete.

inspection—observation, verification, and required documentation of the materials, installation, fabrication, erection, or placement of components and connections to determine compliance with construction documents and referenced standards.

inspection, continuous—the full-time observation, verification, and required documentation of work in the area where the work is being performed.

inspection, periodic—part-time or intermittent observation, verification, and required documentation of work in the area where the work is being performed.

isolation joint—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.

joint—portion of structure common to intersecting members.

lap splice—a connection of reinforcing bars made by lapping the ends of bars.

COMMENTARY

CODE

licensed design professional—an individual who is licensed to practice structural design as defined by the statutory requirements of the professional licensing laws of the state or jurisdiction in which the project is to be constructed, and who is in responsible charge of the structural design.

load—forces or other actions that result from the weight of all building materials, occupants, and their possessions, environmental effects, differential movement, and restrained dimensional changes; permanent loads are those loads in which variations over time are rare or of small magnitude; all other loads are variable loads.

load, dead—(a) the weights of the members, supported structure, and permanent attachments or accessories that are likely to be present on a structure in service; or (b) loads meeting specific criteria found in the general building code; without load factors.

load, factored—load, multiplied by appropriate load factors.

load, live—(a) load that is not permanently applied to a structure but is likely to occur during the service life of the structure (excluding environmental loads); or (b) loads meeting specific criteria found in the general building code; without load factors.

load, self-weight dead—weight of the structural system, including the weight of any bonded concrete topping.

load, service—all loads, static or transitory, imposed on a structure or element thereof, during the operation of a facility, without load factors.

load, superimposed dead—dead loads other than the self-weight that are present or considered in the design.

load effects—forces and deformations produced in structural members by applied loads or restrained volume changes.

load path—sequence of members and connections designed to transfer the factored loads and forces in such combinations as are stipulated in this Code, from the point of application or origination through the structure to the final support location or the foundation.

mat—an assembly of reinforcement composed of two or more layers of bars placed at angles to each other and secured together.

mechanical anchorage—any mechanical device capable of developing the specified strength of the reinforcement without damage to the concrete.

modular ratio—the ratio of modulus of elasticity of reinforcement to that of concrete.

modulus of elasticity—ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material.

moment frame—frame in which beams, slabs, columns, and joints resist forces predominantly through flexure, shear, and axial force; beams or slabs are predominantly horizontal or nearly horizontal; columns are predominantly vertical or nearly vertical.

negative reinforcement—reinforcement for negative moment.

COMMENTARY

licensed design professional—May also be referred to as "registered design professional" in other documents; a licensed design professional in responsible charge of the design work is often referred to as "the engineer of record" (EOR).

loads—Numerous definitions for loads are given as the Code contains requirements that are to be met at various load levels. The terms "dead load" and "live load" refer to the unfactored, sometimes called "service" loads specified or defined by the general building code. Service loads (loads without load factors) are to be used where specified in the Code to proportion or investigate members for adequate serviceability. Loads used to proportion a member for adequate strength are defined as factored loads. Factored loads are service loads multiplied by the appropriate load factors for required strength except Wind and Earthquake which are already specified as strength loads in ASCE/SEI 7. The factored load terminology clarifies where the load factors are applied to a particular load, moment, or shear value as used in the Code provisions.

load effects—Stresses and strains are directly related to forces and deformations and are considered load effects.

CODE

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

one-way construction—members designed to be capable of supporting all loads through bending in a single direction; see also **two-way construction**.

pedestal—member with a ratio of height-to-least lateral dimension less than or equal to 3 used primarily to support axial compressive load; for a tapered member, the least lateral dimension is the average of the top and bottom dimensions of the smaller side.

percentage of reinforcement—ratio of nominal cross-sectional area of reinforcement to the effective cross-sectional area of a member, expressed as a percentage.

pullout failure—failure mode in which the reinforcement pulls out of the concrete without development of the required design strength.

pultrusion—continuous process for manufacturing composites that have a constant cross-sectional shape. The process consists of pulling a fiber-reinforcing material through a resin impregnation bath and through a shaping die, where the resin is subsequently cured.

reinforcement, bond-critical—in fire performance, GFRP reinforcement which relies upon the bond of the GFRP bars to concrete for strength in zones directly exposed to fire.

reinforcement, non-bond-critical—in fire performance, GFRP reinforcement that does not rely upon the bond of the GFRP bars to concrete in zones directly exposed to fire for strength of the member in fire.

reinforcement, nonprestressed—bonded reinforcement that is not prestressed.

reliability index—measure of the probability of failure.

roof live load—a load on a roof produced: (a) during maintenance by workers, equipment, and materials, and (b) during the life of the structure by movable objects, such as planters or other similar small decorative appurtenances that are not occupancy related; or loads meeting specific criteria found in the general building code; without load factors.

sand-coated bar—GFRP bar to which a sand coating has been applied to increase bond strength.

Seismic Design Category—classification assigned to a structure based on its occupancy category and the severity of the design earthquake ground motion at the site, as defined by the general building code. Also denoted by the abbreviation SDC.

seismic-force-resisting system—portion of the structure designed to resist earthquake effects required by the general building code using the applicable provisions and load combinations.

service temperature—highest ambient temperature expected to be experienced by a structure or structural member under intended occupancy and use.

COMMENTARY

one-way construction—Joists, beams, girders, and some slabs and foundations are considered one-way construction.

reinforcement, non-bond-critical—In most cases, non-bond-critical reinforcement is the reinforcement that is anchored in a fire-protected embedment zone. Spiral reinforcement is an example of non-bond-critical reinforcement that does not rely on anchorage in a fire-protected embedment zone, as the continuity of the spiral eliminates reliance on bond with the concrete to develop strength.

reliability index— larger reliability index values indicate lower probability of failure.

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shear cap—projection below the slab used to increase the slab shear strength.

shotcrete—mortar or concrete placed pneumatically by high velocity projection from a nozzle onto a surface.

slip—movement occurring between reinforcement and concrete, indicating loss of bond.

spacing—center-to-center distance between adjacent items, such as longitudinal reinforcement or transverse reinforcement.

spacing, clear—least dimension between the outermost surfaces of adjacent items.

span length—distance between supports.

specialty engineer—a licensed design professional to whom a specific portion of the design work has been delegated.

spiral reinforcement—continuously wound reinforcement in the form of a cylindrical helix.

squat wall—reinforced concrete wall with clear height to horizontal length ratio less than 2.

standard hooked bar—a GFRP reinforcing bar with the end factory formed into a hook of prescribed geometry to provide anchorage.

stirrup—GFRP reinforcement used to resist shear and torsion forces in a member; typically bars, either single leg or factory formed into L, U, C or rectangular shapes and located perpendicular to longitudinal reinforcement. See also **tie.**

strength, design—nominal strength multiplied by a strength reduction factor **ϕ**.

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shotcrete—Terms such as "gunite" and "sprayed concrete" are sometimes used to refer to shotcrete.

stirrup—The term "stirrup" is usually applied to transverse reinforcement in beams or slabs and the term "ties" to transverse reinforcement in compression members. Figures R2.3a and R2.3b illustrate C-shaped and U-shaped transverse reinforcement.

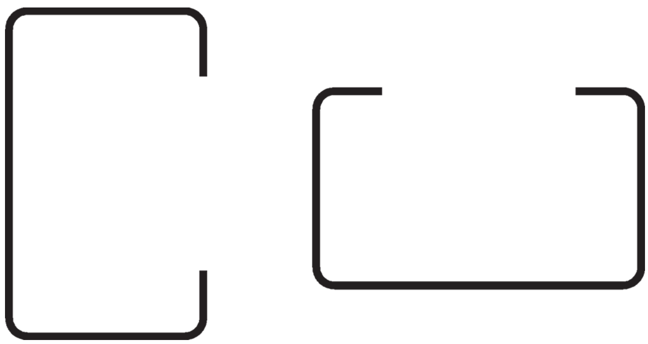

Fig. R2.3a—C-shaped transverse reinforcement.

Fig. R2.3b—U-shaped transverse reinforcement.

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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.

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

structural diaphragm—member, such as a floor or roof slab, that transmits forces acting in the plane of the member to vertical elements of the lateral-force-resisting system. A structural diaphragm may include chords and collectors as part of the diaphragm.

structural integrity—ability of a structure through strength, redundancy, deformability, and detailing of reinforcement to redistribute stresses and maintain overall stability if localized damage or significant overstress occurs.

structural system—interconnected members designed to meet performance requirements.

structural truss—assemblage of reinforced concrete members subjected primarily to axial forces.

structural wall—wall proportioned to resist combinations of shears, moments, and axial forces in the plane of the wall; a shear wall is a structural wall.

structural wall, ordinary reinforced concrete—a wall complying with Chapter 11.

strut—intact concrete that carries the compressive forces between diagonal tension cracks.

surface enhancement—treatment applied or created during manufacture of GFRP reinforcing bars, in the form of sand coating, spiral winding, machined grooves, or other methods, or combinations thereof to enhance bond strength of GFRP reinforcement.

tension-controlled section—a cross section in which the extreme layer of the GFRP tensile reinforcement ruptures before the concrete crushes.

tie—reinforcing bar enclosing longitudinal reinforcement; a continuously wound transverse bar in the form of a circle, rectangle, or other polygonal shape without reentrant corners enclosing longitudinal reinforcement. See also **stirrup**.

transition section—a cross section in which the net tensile strain in the extreme GFRP tension reinforcement at nominal strength is between $0.8\varepsilon_{fu}$ and ε_{fu}.

two-way construction—members designed to be capable of supporting loads through bending in two directions; some slabs and foundations are considered two-way construction. See also **one-way construction**.

COMMENTARY

strength, nominal—Nominal or specified values of material strengths and dimensions are used in the calculation of nominal strength. The subscript n is used to denote the nominal strengths; for example, nominal axial load strength P_n, nominal moment strength M_n, and nominal shear strength V_n. For additional discussion on the concepts and nomenclature for strength design, refer to the Commentary of Chapter 22.

strength, required—The subscript u is used only to denote the required strengths; for example, required axial load strength P_u, required moment strength M_u, and required shear strength V_u, calculated from the applied factored loads and forces. The basic requirement for strength design may be expressed as follows: design strength ≥ required strength; for example, $\phi P_n \geq P_u$; $\phi M_n \geq M_u$; $\phi V_n \geq V_u$. For additional discussion on the concepts and nomenclature for strength design, refer to the Commentary of Chapter 22.

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wall—a vertical element designed to resist axial load, lateral load, or both, with a horizontal length-to-thickness ratio greater than 3, used to enclose or separate spaces.

wall segment—portion of wall bounded by vertical or horizontal openings or edges.

wall segment, horizontal—segment of a structural wall, bounded vertically by two openings or by an opening and an edge.

wall segment, vertical—segment of a structural wall, bounded horizontally by two openings or by an opening and an edge; wall piers are vertical wall segments.

water-cementitious materials ratio—ratio of mass of water, excluding that absorbed by the aggregate, to the mass of cementitious materials in a mixture, stated as a decimal.

work—the entire construction or separately identifiable parts thereof that are required to be furnished under the construction documents.

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CHAPTER 3—REFERENCED STANDARDS

3.1—Referenced standards

Standards, or specific sections thereof, cited in this Code, including Annex, Appendixes, or Supplements where prescribed, are referenced without exception in this Code, unless specifically noted. Cited standards are listed in the following with their serial designations, including year of adoption or revision.

3.1.1

ACI 301M-16—Metric Specifications for Structural Concrete

ACI 318-19—Building Code Requirements for Reinforced Concrete and Commentary

ACI SPEC-440.5-22—Construction with Glass Fiber-Reinforced Polymer Reinforcing Bars—Specification

3.1.2 *American Society of Civil Engineers*

ASCE/SEI 7-16—Minimum Design Loads for Buildings and Other Structures, Sections 2.3.2, Load Combinations Including Flood Loads; and 2.3.3, Load Combinations Including Atmospheric Ice Loads

3.1.3 *ASTM International*

ASTM C31/C31M-19—Standard Practice for Making and Curing Concrete Test Specimens in the Field

ASTM C33/C33M-18—Standard Specification for Concrete Aggregates

ASTM C39/C39M-18—Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens

ASTM C42/C42M-18a—Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

ASTM C94/C94M-18—Standard Specification for Ready-Mixed Concrete

ASTM C150/C150M-19a—Standard Specification for Portland Cement

ASTM C172/C172M-17—Standard Practice for Sampling Freshly Mixed Concrete

ASTM C173/C173M-16—Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method

ASTM C192/C192M-18—Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory

ASTM C231/C231M-17a—Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method

ASTM C260/C260M-10a(2016)—Standard Specification for Air-Entraining Admixtures for Concrete

ASTM C469/C469M-14—Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression

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CHAPTER R3—REFERENCED STANDARDS

R3.1—Referenced standards

In this Code, references to standard specifications or other material are to a specific edition of the cited document. This is done by using the complete serial designation for the referenced standard including the title that indicates the subject and year of adoption. All standards referenced in this Code are listed in this chapter, with the title and complete serial designation. In other sections of the Code, referenced standards are abbreviated to include only the serial designation without a title or date. These abbreviated references correspond to specific standards listed in this chapter.

R3.1.1

Article 4.2.3 of ACI 301M is referenced for the method of mixture proportioning cited in 26.4.3.1(b).

R3.1.2 *American Society of Civil Engineers*

The two specific sections of ASCE/SEI 7 are referenced for the purposes cited in 5.3.9 and 5.3.10.

R3.1.3 *ASTM International*

The ASTM standards listed are the latest editions at the time the code provisions for the corresponding version of ACI 318 were adopted. ASTM standards are revised frequently relative to the revision cycle for the Code. Current and historical editions of the referenced standards can be obtained from ASTM International. Use of an edition of a standard other than that referenced in the Code obligates the user to evaluate if any differences in the nonconforming edition are significant to use of the standard.

Many of the ASTM standards are combined standards as denoted by the dual designation, such as ASTM C31/C31M. For simplicity, these combined standards are referenced without the metric (M) designation within the text of the Code and Commentary. In this provision, however, the complete designation is given because that is the official designation for the standard.

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ASTM C494/C494M-17—Standard Specification for Chemical Admixtures for Concrete

ASTM C595/C595M-19—Standard Specification for Blended Hydraulic Cements

ASTM C618-19—Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete

ASTM C685/C685M-17a—Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing

ASTM C845/C845M-18—Standard Specification for Expansive Hydraulic Cement

ASTM C989/C989M-18a—Standard Specification for Slag Cement for Use in Concrete and Mortars

ASTM C1012/C1012M-18b—Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution

ASTM C1017/C1017M-13$^{\varepsilon1}$—Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete

ASTM C1077-17—Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Testing Agency Evaluation

ASTM C1157/C1157M-17—Standard Performance Specification for Hydraulic Cement

ASTM C1240-15—Standard Specification for Silica Fume Used in Cementitious Mixtures

ASTM C1580-15—Standard Test for Water-Soluble Sulfate in Soil

ASTM C1602/C1602M-18—Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete

ASTM C1797-17—Standard Specification for Ground Calcium Carbonate and Aggregate Mineral Fillers for use in Hydraulic Cement Concrete

ASTM D516-16—Standard Test Method for Sulfate Ion in Water

ASTM D4130-15—Standard Test Method for Sulfate Ion in Brackish Water, Seawater, and Brines

ASTM D7957/D7957M-22—Standard Specification for Solid Round Glass Fiber Reinforced Polymer Bars for Concrete Reinforcement

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CHAPTER 4—STRUCTURAL SYSTEM REQUIREMENTS

4.1—Scope

4.1.1 This chapter shall apply to design of GFRP-reinforced concrete in structures or portions of structures defined in Chapter 1.

4.2—Materials

4.2.1 Design properties of concrete shall be selected to be in accordance with Chapter 19.

4.2.2 Design properties of GFRP reinforcement shall be selected to be in accordance with Chapter 20.

4.3—Design loads

4.3.1 Loads and load combinations considered in design shall be in accordance with Chapter 5.

4.4—Structural system and load paths

4.4.1 The structural system shall include (a) through (g), as applicable:

(a) Floor construction and roof construction, including one-way and two-way slabs
(b) Beams and joists
(c) Columns
(d) Walls
(e) Diaphragms
(f) Foundations
(g) Joints, connections, and anchors as required to transmit forces from one component to another

COMMENTARY

CHAPTER R4—STRUCTURAL SYSTEM REQUIREMENTS

R4.1—Scope

This chapter introduces structural system requirements. Requirements more stringent than the Code provisions may be desirable for unusual construction or construction where enhanced performance is appropriate. The Code and Commentary must be supplemented with sound engineering knowledge, experience, and judgment.

R4.2—Materials

Chapter 3 identifies the referenced standards permitted for design. Chapters 19 and 20 establish properties of concrete and GFRP reinforcement permitted for design. Chapter 26 presents construction requirements for concrete materials, proportioning, and acceptance of concrete.

R4.3—Design loads

R4.3.1 The provisions in Chapter 5 are based on ASCE/SEI 7. The design loads include, but are not limited to, dead loads, live loads, snow loads, wind loads, earthquake effects, prestressing effects, crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete, and predicted unequal settlement of supports. Other project-specific loads may be specified by the licensed design professional.

R4.4—Structural system and load paths

R4.4.1 Structural concrete design has evolved from emphasizing the design of individual members to designing the structure as an entire system. A structural system consists of structural members, joints, and connections, each performing a specific role or function. A structural member may belong to one or more structural systems, serving different roles in each system and having to meet all the detailing requirements of the structural systems of which they are a part. Joints and connections are locations common to intersecting members or are items used to connect one member to another, but the distinction between members, joints, and connections can depend on how the structure is idealized. Throughout this chapter, the term "members" often refers to GFRP-reinforced concrete members, joints, and connections.

Although the Code is written considering that a structural system comprises these members, many alternative arrangements are possible because not all GFRP-reinforced concrete member types are used in all building structural systems. The selection types of the members to use in a specific project and the role or roles these member types play is made by the licensed design professional complying with requirements of the Code.

This Code does not cover the requirements for, or design of, GFRP-reinforced concrete diaphragms due to a lack of published research on this topic. This Code does cover the requirements for GFRP-reinforced concrete one-way and two-way slabs if the GFRP reinforcement is not relied upon to transfer the lateral forces from diaphragm action. A structural system may have steel- reinforced concrete diaphragms interacting with other GFRP-reinforced concrete members.

4.4.2 Design of GFRP-reinforced concrete members including joints and connections given in 4.4.1 shall be in accordance with Chapters 7 through 11, 13, 15, and 16.

R4.4.2 In the chapter for each type of structural member, requirements follow the same general sequence and scope, including general requirements, design limits, required strength, design strength, reinforcement limits, reinforcement detailing, and other requirements unique to the type of member.

4.4.3 It shall be permitted to design a structural system comprising structural members not in accordance with 4.4.1 and 4.4.2, provided the structural system is approved in accordance with 1.10.1.

R4.4.3 Some materials, structural members, or systems that may not be recognized in the prescriptive provisions of the Code may still be acceptable if they meet the intent of the Code. Section 1.10.1 outlines the procedures for obtaining approval of alternative materials and systems.

4.4.4 The structural system shall be designed to resist the factored loads in load combinations given in 4.3 without exceeding the appropriate member design strengths, considering one or more continuous load paths from the point of load application or origination to the final point of resistance.

R4.4.4 The design should be based on members and connections that provide design strengths not less than the strengths required to transfer the loads along the load path. The licensed design professional may need to study one or more alternative paths to identify weak links along the sequence of elements that constitute each load path.

4.4.5 Structural systems shall be designed to accommodate anticipated volume change and differential settlement.

R4.4.5 The effects of column and wall creep and shrinkage, restraint of creep and shrinkage in long roof and floor systems, volume changes caused by temperature variation, as well as potential damage to supporting members caused by these volume changes should be considered in design. Reinforcement, closure strips, or expansion joints are common ways of accommodating these effects. Minimum shrinkage and temperature reinforcement controls cracking to an acceptable level in many concrete structures of ordinary proportions and exposures.

Differential settlement or heave may be an important consideration in design. Geotechnical recommendations to allow for nominal values of differential settlement and heave are not normally included in design load combinations for ordinary building structures.

4.4.6 *Seismic-force-resisting system*

R4.4.6 *Seismic-force-resisting system*

4.4.6.1 Every structure shall be assigned to a Seismic Design Category in accordance with the general building code or as determined by the building official in areas without a legally adopted building code. GFRP-reinforced concrete members designated as part of the seismic-force-resisting system in structures assigned to Seismic Design Categories B, C, D, E, and F are not covered by this code.

R4.4.6.1 Design requirements in the Code are based on the seismic design category to which the structure is assigned. In general, the seismic design category relates to seismic risk level, soil type, occupancy, and building use. Assignment of a building to a seismic design category is under the jurisdiction of a general building code rather than this Code. In the absence of a general building code, ASCE/SEI 7 provides the assignment of a building to a seismic design category.

4.4.6.2 Structural systems designated as part of the seismic-force-resisting system shall be restricted to those systems designated by the general building code or as determined by the building official in areas without a legally adopted building code.

R4.4.6.2 The general building code prescribes, through ASCE/SEI 7, the types of structural systems permitted as part of the seismic-force-resisting system based on considerations such as seismic design category and building height. Other systems can be used if approved by the building official.

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4.4.6.3 GFRP-reinforced concrete members in structural systems assigned to Seismic Design Category A shall satisfy the applicable requirements of this Code.

4.4.6.4 Intentionally left blank.

4.4.6.5 GFRP-reinforced concrete members assumed not to be part of the seismic-force-resisting system shall be permitted in structures assigned to Seismic Design Category B or C, subject to the requirements of 4.4.6.5.1 and 4.4.6.5.2. GFRP-reinforced concrete members are not permitted in structures assigned to Seismic Design Categories D, E, and F.

4.4.6.5.1 In structures assigned to Seismic Design Category B or C, the effects of those structural members on the response of the system shall be considered and accommodated in the structural design.

4.4.6.5.2 In structures assigned to Seismic Design Category B or C, the consequences of damage to those structural members shall be considered.

4.4.7 *Diaphragms*—Out of scope.

4.5—Structural analysis

4.5.1 Analytical procedures shall satisfy compatibility of deformations and equilibrium of forces.

4.5.2 The methods of analysis given in Chapter 6 shall be permitted.

4.6—Strength

4.6.1 Design strength of a member and its joints and connections, in terms of moment, axial force, shear, torsion, and bearing, shall be taken as the nominal strength S_n multiplied by the applicable strength reduction factor ϕ.

4.6.2 Structures and structural members shall have design strength at all sections, ϕS_n, greater than or equal to the required strength U calculated for the factored loads and forces in such combinations as required by this Code or the general building code.

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R4.4.6.3 Structures assigned to Seismic Design Category A are subject to the lowest seismic hazard.

R4.5—Structural analysis

The role of analysis is to estimate the internal forces and deformations of the structural system and to establish compliance with the strength, serviceability, and stability requirements of the Code. The use of computers in structural engineering has made it feasible to perform analysis of complex structures. The Code requires that the analytical procedure used meets the fundamental principles of equilibrium and compatibility of deformations as provided in Chapter 6.

R4.6—Strength

The basic requirement for strength design may be expressed as follows:

$$\text{design strength} \geq \text{required strength}$$

$$\phi S_n \geq U$$

In the strength design procedure, the level of safety is provided by a combination of factors applied to the loads and strength reduction factors ϕ applied to the nominal strengths.

The strength of a member or cross section, calculated using standard assumptions and strength equations, along with nominal values of material strengths and dimensions, is referred to as nominal strength and is generally designated S_n. Design strength or usable strength of a member or cross section is the nominal strength reduced by the applicable strength reduction factor ϕ. The purpose of the strength reduction factor is to account for the probability of understrength due to variations of in-place material strengths and dimensions, the effect of simplifying assumptions in the

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design equations, the degree of ductility, potential failure mode of the member, the required reliability, and significance of failure and existence of alternative load paths for the member in the structure.

This Code, or the general building code, prescribes design load combinations, also known as factored load combinations, which define the way different types of loads are multiplied (factored) by individual load factors and then combined to obtain a factored load ***U***. The individual load factors and additive combination reflect the variability in magnitude of the individual load effect, the probability of simultaneous occurrence of various load effects, and the assumptions and approximations made in the structural analysis when determining required design strengths.

A typical design approach, where linear analysis is applicable, is to analyze the structure for individual unfactored load cases, and then combine the individual unfactored load cases in a factored load combination to determine the design load effects. Where effects of loads are nonlinear—for example, in foundation uplift—the factored loads are applied simultaneously to determine the nonlinear, factored load effect. The load effect relevant for strength design includes moments, shears, axial forces, torsions, bearing forces, and punching shear stresses. Sometimes, design displacements are determined for factored load effects. The load effects relevant for service design include stresses and deflections.

In the course of applying these principles, the licensed design professional should be aware that providing more strength than required does not necessarily lead to a safer structure because doing so may change the potential failure mode. For example, increasing longitudinal reinforcement area beyond that required for moment strength as derived from analysis without increasing transverse reinforcement could increase the probability of a shear failure occurring prior to a flexural failure.

4.7—Serviceability

4.7.1 Evaluation of performance at service load conditions shall consider reactions, moments, torsions, shears, and axial forces induced by creep, shrinkage, temperature change, axial deformation, restraint of attached structural members, and foundation settlement.

=**4.7.2** For structures, structural members, and their connections, the requirements of 4.7.1 shall be deemed to be satisfied if designed in accordance with the provisions of the applicable member chapters.

R4.7—Serviceability

Serviceability refers to the ability of the structural system or structural member to provide appropriate behavior and functionality under the actions affecting the system. Serviceability requirements address issues such as deflections and cracking, among others. Creep-rupture failure is addressed under sustained service loads. Serviceability considerations for vibrations are discussed in R6.6.3.2.2 and R24.1.

Except as stated in Chapter 24, service-level load combinations are not defined in this Code but are discussed in Appendix C of ASCE/SEI 7-16. Appendixes to ASCE/SEI 7 are not considered mandatory parts of the standard.

4.8—Durability

=**4.8.1** Concrete mixtures shall be designed in accordance with the requirements of 19.3.2 and 26.4, considering applicable environmental exposure to provide required durability.

4.8.2 Reinforcement shall be protected in accordance with 20.5.

R4.8—Durability

=The environment where the structure will be located will dictate the exposure category for materials selection, design details, and construction requirements to minimize potential for premature deterioration of the structure caused by environmental effects. Durability of a structure is also

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impacted by the level of preventative maintenance, which is not addressed in the Code.

Chapter 19 provides requirements for protecting concrete against major environmental causes of deterioration.

4.9—Sustainability

4.9.1 The licensed design professional shall be permitted to specify in the construction documents sustainability requirements in addition to strength, serviceability, and durability requirements of this Code.

4.9.2 The strength, serviceability, and durability requirements of this Code shall take precedence over sustainability considerations.

R4.9—Sustainability

The Code provisions for strength, serviceability, and durability are minimum requirements to achieve a safe and durable concrete structure. The Code permits the owner or the licensed design professional to specify requirements higher than the minimums mandated in the Code. Such optional requirements can include higher strengths, more restrictive deflection limits, enhanced durability, and sustainability provisions.

4.10—Structural integrity

4.10.1 *General*

4.10.1.1 Reinforcement and connections shall be detailed to tie the structure together effectively and to improve overall structural integrity.

4.10.2 *Minimum requirements for structural integrity*

4.10.2.1 Structural members and their connections shall be in accordance with structural integrity requirements in Table 4.10.2.1.

Table 4.10.2.1—Minimum requirements for structural integrity

Member type	Section
One-way cast-in-place slabs	7.7.7
Two-way slabs	8.7.4.2
Cast-in-place beam	9.7.7
One-way joist system	9.8.1.6

R4.10—Structural integrity

R4.10.1 *General*

R4.10.1.1 It is the intent of the structural integrity requirements to improve redundancy and deformability through detailing of GFRP reinforcement and connections so that, in the event of damage to a major supporting element or an abnormal loading, the resulting damage will be localized and the structure will have a higher probability of maintaining overall stability.

Integrity requirements for selected structural member types are included in the corresponding member chapter in the sections noted.

R4.10.2 *Minimum requirements for structural integrity*

Structural members and their connections referred to in this section include only member types that have specific requirements for structural integrity. Notwithstanding, detailing requirements for other member types address structural integrity indirectly. Such is the case for detailing of one-way slabs as provided in 7.7.

4.11—Fire resistance and elevated service temperature

4.11.1 Structural concrete reinforced with GFRP bars shall not be permitted where fire-resistance ratings are required except where the structural fire resistance has been shown to be adequate by calculations or tests and approved by the building official.

R4.11—Fire resistance and elevated service temperature

R4.11.1 The performance of GFRP-reinforced concrete elements at high temperatures relies primarily on the GFRP reinforcement-concrete bond strength being maintained (Hajiloo and Green 2018; Hajiloo et al. 2017, 2019; Nigro et al. 2011). Table R20.5.1.3.1 provides the fire-resistance ratings for the concrete cover specified in Table 20.5.1.3.1 for non-bond-critical GFRP reinforcement. Achieving non-bond-critical GFRP reinforcement requires specific detailing for anchorage. This commentary provides guidance on

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detailing to obtain non-bond-critical GFRP reinforcement in members to prevent anchorage failure during a fire event.

In beams and slabs, specific GFRP reinforcement detailing for anchorage will usually consist of providing adequate embedment length into the support to anchor the GFRP reinforcement or using additional concrete cover or insulation near the supports to reduce the temperature of the GFRP reinforcement near the supports during a fire event. During the fire event, the maximum bar stress due to the service load combination of $1.0D + 1.0L$ should be less than $0.3f_{fu}$ and the average bar temperature over the fire-protected embedment length should be less than 99°C for the required fire-resistance duration. Ideally, the bond development length corresponding to 1.3 times the maximum bar stress due to the full service loads ($1.0D + 1.0L$) should be embedded into the support (that is, in an area not directly exposed to fire). The 1.3 factor accounts for increased stress in the bars during the fire event as a result of extensive deformations in the slabs under combined load and fire effects (Hajiloo et al. 2019). Instead of development length calculations, embedment into the support of the larger of 300 mm or $20d_b$ is conservative for No. 32 and smaller bars with $f_c' \geq 28$ MPa and maximum bar stresses due to full service loads ($1.0D + 1.0L$) less than 240 MPa. If this embedment length into the support or unexposed length ℓ_{un}, cannot be achieved, additional protection can be provided at the ends of GFRP reinforcement near supports by increasing the concrete cover using a haunch or drop panel (Fig R4.11.1a and R4.11.1b) or insulating the concrete (Fig R4.11.1c). In these figures, ℓ_{pa} is the fire-protected embedment length of GFRP reinforcement and is measured from the end of the reinforcing bars to the end of the haunch, drop panel or insulation and d_{pa} is the depth of the haunch, drop panel or insulation. Table R4.11.1 provides the suggested fire-protected embedment length and depth of haunch, drop panel or insulation based on unexposed length at the ends of the GFRP reinforcement for the case of $f_c' \geq 28$ MPa, clear cover c_c at least 38 mm, and a maximum bar stress due to full service loads ($1.0D + 1.0L$) less than 240 MPa. Tests have shown that a 38 mm insulation layer can keep the temperature at the GFRP reinforcing bars with 38 mm of clear concrete cover below 99°C for 2 hours of fire exposure (Williams et al. 2008; Adelzadeh et al. 2012).

Any tensile GFRP reinforcement splice in a member will necessarily result in a bond-critical area under fire. Figure R4.11.1d shows one option for protecting positive moment splice regions by adding insulation under the splice region. For $f_c' \geq 28$ MPa, clear cover c_c at least 38 mm, and a maximum bar stress due to full service loads ($1.0D + 1.0L$) less than 240 MPa, the length of the insulated area (ℓ_{in}) should be the larger of 900 mm or $60d_b$, and extend at least 75 mm beyond each end of the spliced GFRP reinforcement.

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Table R4.11.1—Haunch, drop panel, or insulation for protection of GFRP reinforcement near supports*

ℓ_{un}, mm	ℓ_{pa}, mm	d_{pa}, mm
100	Max(560 or 30d_b)	50
150	Max(510 or 28d_b)	50
200	Max(410 or 25d_b)	50
250	Max(360 or 22d_b)	50
Max(300 or 20d_b)	—	—

*For 2-hour fire exposure. Assumes clear cover ≥ 38 mm, $f_c' \geq 28$ MPa, and maximum bar stress due to $1.0D + 1.0L < 240$ MPa.

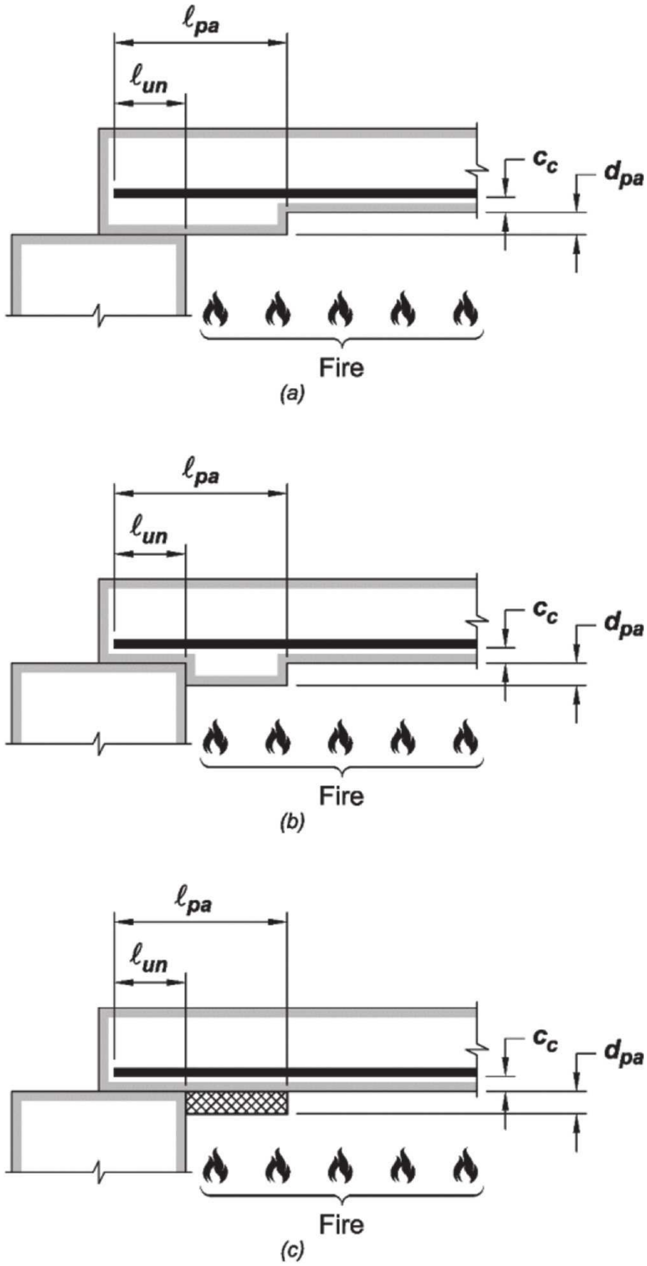

Fig R4.11.1a-c—Protection of GFRP reinforcement near supports.

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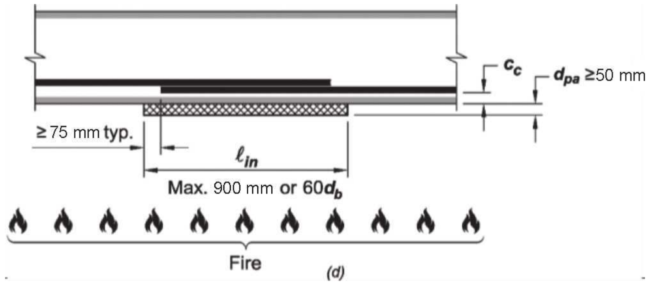

Fig R4.11.1d—Insulation at spliced GFRP reinforcement.

If insulation is used to control GFRP bar temperature, the insulation should be at least 50 mm thick and the insulation material should be tested for application on concrete in accordance with ASTM E119 to verify that the insulated concrete surface temperature does not exceed 150°C for the duration of the required fire-resistance rating. The increased cover or insulation used to control GFRP bar temperature should extend for the full width of the beam or slab. The length of the fire-protected embedment zone ℓ_{pa} and ℓ_{in} should extend at least ℓ_d as determined from 25.4.2.1 using an f_{fr} of 1.3 times the service bar stress at $1.0D + 1.0L$ at the ends of straight bars, ℓ_d as determined from 25.4.3.1 at the ends of longitudinal hooked bars, and the greatest of the splice length plus 150 mm, 900 mm, and $60d_b$ at splices.

Shear reinforcement consisting of lap-spliced GFRP stirrups will necessarily result in a bond-critical area under fire. The maximum stress at ultimate in the transverse reinforcement due to $1.2D + 1.6L$ should be less than $0.45f_{fu}$. In general, this stress limit will be satisfied by 20.2.2.6, which limits the strain in the transverse reinforcement to 0.005 at ultimate. For example, using GFRP bars with f_{fu} = 830 MPa and E_f = 62,000 MPa, the 0.005 strain limit at ultimate corresponds to a stress of $0.375f_{fu}$ under full factored loads. Assuming a lower bound average load factor of 1.4 (corresponding to a live-to-dead load ratio of 1), the stress under service loads would be no larger than $0.27f_{fu}$. Smaller moduli of elasticity and larger bar strengths would result in stresses attaining an even smaller percentage of f_{fu}. As with longitudinal bars, the average temperature of the GFRP transverse reinforcement in the bond-critical areas should be less than 99°C for the required fire-resistance duration and the insulation should meet the same thickness and test requirements as that for the longitudinal reinforcement protection.

GFRP reinforcement in compression is ineffective at high temperatures due to loss of stiffness of the polymer and buckling of the fibers due to the low polymer modulus; hence, GFRP bars in compression under fire conditions should not be considered in the compressive strength of the member.

The temperatures of the GFRP reinforcement in vertical elements (columns and walls) during fire can be controlled by a combination of concrete cover, insulation, and protective coatings, such as dry wall. Instead of detailed calculations, an insulation system that keeps the concrete surface temperature below 150°C can be considered an indirect method to keep GFRP bar temperatures below 99°C. Examples of fire

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insulation systems for columns as well as numerical models for concrete columns under fire can be found in Bisby et al. (2005) and Cree et al. (2012).

4.11.2 Intentionally left blank.

4.11.3 GFRP bars shall not be used in environments with a service temperature higher than –3°C below the glass transition temperature of the bar, as determined in accordance with the requirements of ASTM D7957.

R4.11.3 GFRP bars can lose bond with concrete if used in conditions that have service temperatures approaching the glass transition temperature of the bar, due to softening of the resin (Xian and Karbhari 2007). ASTM D7957 requires GFRP bars to have a minimum mean glass transition temperature of 100°C. GFRP bars with glass transition temperatures in excess of 120°C are commercially available.

4.12—Requirements for specific types of construction

R4.12—Requirements for specific types of construction

This section contains requirements that are related to specific types of construction. Additional requirements that are specific to member types appear in the corresponding member chapters.

4.12.1 *Precast concrete systems*

R4.12.1 *Precast concrete systems*

4.12.1.1 Design of precast concrete 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.

All requirements in the Code apply to precast systems and members unless specifically excluded. In addition, some requirements apply specifically to precast concrete. This section contains specific requirements for precast systems. Other sections of this Code also provide specific requirements for precast systems.

Precast systems differ from monolithic systems in that the type of restraint at supports, the location of supports, and the induced stresses in the body of the member vary during fabrication, storage, transportation, erection, and the final interconnected configuration. Consequently, the member design forces to be considered may differ in magnitude and direction with varying critical sections at various stages of construction. For example, a precast flexural member may be simply supported for dead load effects before continuity at the supporting connections is established and may be a continuous member for live or environmental load effects due to the moment continuity created by the connections after erection.

4.12.1.2 Design, fabrication, and construction of precast members and their connections shall include the effects of tolerances.

R4.12.1.2 For guidance on including the effects of tolerances, refer to the *PCI Design Handbook* (PCI MNL 120).

4.12.1.3 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.

4.12.1.4 Where system behavior requires in-plane loads to be transferred between the members of a precast floor or wall system, (a) and (b) shall be satisfied:

(a) In-plane load paths shall be continuous through both connections and members.

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(b) Where tension loads occur, a load path of reinforcement, with or without splices, shall be provided.

=**4.12.1.5** Distribution of forces that act perpendicular to the plane of precast members shall be established by analysis or test.

4.12.2 *Prestressed concrete systems*—Out of scope

4.12.3 *Composite concrete flexural members*

=**4.12.3.1** This Code shall apply to composite concrete flexural members as defined in Chapter 2.

=**4.12.3.2** Individual members shall be designed for all critical stages of loading.

=**4.12.3.3** Members shall be designed to support all loads introduced prior to full development of design strength of composite members.

4.12.4 *Structural plain concrete system*—Not applicable

4.13—Construction and inspection

=**4.13.1** Specifications for construction execution shall be in accordance with Chapter 26.

=**4.13.2** Inspection during construction shall be in accordance with Chapter 26 and the general building code.

4.14—Strength evaluation of existing structures

=**4.14.1** Strength evaluation of existing structures shall be in accordance with Chapter 27.

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R4.12.1.5 Concentrated and line loads can be distributed among members, provided the members have sufficient torsional stiffness and shear can be transferred across joints. Torsionally stiff members such as hollow-core or solid slabs will provide better load distribution than torsionally flexible members such as double tees with thin flanges. The actual distribution of the load depends on many factors, discussed in detail in LaGue (1971), Johnson and Ghadiali (1972), Pfeifer and Nelson (1983), Stanton (1987, 1992), *PCI Manual for the Design of Hollow Core Slabs and Walls* (PCI MNL 126), Aswad and Jacques (1992), and the *PCI Design Handbook* (PCI MNL 120). Large openings can cause significant changes in distribution of forces.

R4.12.3 *Composite concrete flexural members*

This section addresses structural concrete members, either precast or cast-in-place, consisting of concrete cast at different times intended to act as a composite member when loaded after concrete of the last stage of casting. All requirements in the Code apply to these members unless specifically excluded. In addition, some requirements apply specifically to composite concrete flexural members. This section contains requirements that are specific to these elements and are not covered in the applicable member chapters.

R4.13—Construction and inspection

=Chapter 26 has been organized to collect into one location the design information, compliance requirements, and inspection provisions from the Code that should be included in construction documents. There may be other information that should be included in construction documents that is not covered in Chapter 26.

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CHAPTER 5—LOADS

CHAPTER R5—LOADS

5.1—Scope

=**5.1.1** This chapter shall apply to selection of load factors and combinations used in design, except as permitted in Chapter 27.

5.2—General

=**5.2.1** Loads shall include self-weight; applied loads; and effects of prestressing, earthquakes, restraint of volume change, and differential settlement.

=**5.2.2** Loads and Seismic Design Categories (SDCs) shall be in accordance with the general building code, or determined by the building official.

=**5.2.3** Live load reductions shall be permitted in accordance with the general building code or, in the absence of a general building code, in accordance with ASCE/SEI 7.

R5.2—General

R5.2.1 Provisions in the Code are associated with dead, live, wind, and earthquake loads such as those recommended in ASCE/SEI 7. The commentary to Appendix C of ASCE/SEI 7 provides service-level wind loads W_a for serviceability checks; however, these loads are not appropriate for strength design. Although this Code does not cover the use of GFRP reinforcement for prestressed concrete members, GFRP-reinforced concrete members may be present in structures that include prestressing effects and are possibly subject to loads from such effects.

If the service loads specified by the general building code differ from those of ASCE/SEI 7, the general building code governs. However, if the nature of the loads contained in a general building code differs considerably from ASCE/SEI 7 loads, some provisions of this Code may need modification to reflect the difference.

R5.2.2 Seismic Design Categories (SDCs) in this Code are adopted directly from ASCE/SEI 7. Similar designations are used by the International Building Code (2012 IBC) and the National Fire Protection Association (NFPA 5000 2012). The BOCA National Building Code (BOCA 1999) and "The Standard Building Code" (SBC 1999) used seismic performance categories. The "Uniform Building Code" (IBCO 1997) relates seismic design requirements to seismic zones.

Design requirements for earthquake-resistant structures in this Code are determined by the SDC to which the structure is assigned. In general, the SDC relates to seismic hazard level, soil type, occupancy, and building use. Assignment of a building to an SDC is under the jurisdiction of the general building code rather than this Code. This code does not cover the design of GFRP-reinforced concrete members designated as part of the seismic-force resisting system in structures assigned to Seismic Design Categories B, C, D, E, or F.

5.3—Load factors and combinations

5.3.1 Required strength U for building structures shall be at least equal to the effects of factored loads in Table 5.3.1, with exceptions and additions in 5.3.3 through 5.3.10.

R5.3—Load factors and combinations

R5.3.1 The required strength U is expressed in terms of factored loads, or related internal moments and forces. Factored loads are the loads specified in the general building code multiplied by appropriate load factors. If the load effects such as internal forces and moments are linearly related to the loads, the required strength U may be expressed in terms of load effects multiplied by the appropriate load factors

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Table 5.3.1—Load combinations

Load combination	Equation	Primary load
$U = 1.4D$	(5.3.1a)	D
$U = 1.2D + 1.6L + 0.5(L_r$ or S or $R)$	(5.3.1b)	L
$U = 1.2D + 1.6(L_r$ or S or $R) + (1.0L$ or $0.5W)$	(5.3.1c)	L_r or S or R
$U = 1.2D + 1.0W + 1.0L + 0.5(L_r$ or S or $R)$	(5.3.1d)	W
$U = 1.2D + 1.0E + 1.0L + 0.2S$	(5.3.1e)	E
$U = 0.9D + 1.0W$	(5.3.1f)	W
$U = 0.9D + 1.0E$	(5.3.1g)	E

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with the identical result. If the load effects are nonlinearly related to the loads, such as frame $\boldsymbol{P}\Delta$ effects (Rogowsky and Wight 2010), the loads are factored before determining the load effects. Typical practice for foundation design is discussed in R13.2.6.1. Nonlinear finite element analysis using factored load cases is discussed in R6.9.3.

The factor assigned to each load is influenced by the degree of accuracy to which the load effect usually can be calculated and the variation that might be expected in the load during the lifetime of the structure. Dead loads, because they are more accurately determined and less variable, are assigned a lower load factor than live loads. Load factors also account for variability in the structural analysis used to calculate moments and shears.

The Code gives load factors for specific combinations of loads. In assigning factors to combinations of loading, some consideration is given to the probability of simultaneous occurrence. While most of the usual combinations of loadings are included, it should not be assumed that all cases are covered.

Due regard is to be given to the sign (positive or negative) in determining $\boldsymbol{U}$ for combinations of loadings, as one type of loading may produce effects of opposite sense to that produced by another type. The load combinations with $\mathbf{0.9}\boldsymbol{D}$ are included for the case where a higher dead load reduces the effects of other loads. The loading case may also be critical for tension-controlled column sections. In such a case, a reduction in compressive axial load or development of tension with or without an increase in moment may result in a critical load combination.

Consideration should be given to various combinations of loading to determine the most critical design condition. This is particularly true when strength is dependent on more than one load effect, such as strength for combined flexure and axial load or shear strength in members with axial load.

If unusual circumstances require greater reliance on the strength of particular members than circumstances encountered in usual practice, some reduction in the stipulated strength reduction factors ϕ or increase in the stipulated load factors may be appropriate for such members.

Rain load $\boldsymbol{R}$ in Eq. (5.3.1b), (5.3.1c), and (5.3.1d) should account for all likely accumulations of water. Roofs should be designed with sufficient slope or camber to ensure adequate drainage accounting for any long-term deflection of the roof due to the dead loads. If deflection of roof members may result in ponding of water accompanied by increased deflection and additional ponding, the design should ensure that this process is self-limiting.

Model building codes and design load references refer to earthquake forces at the strength level, and the corresponding load factor is 1.0 (ASCE/SEI 7; BOCA [1999]; SBC [1999]; UBC [ICBO 1997]; 2012 IBC). In the absence of a general building code that prescribes strength level earthquake effects, a higher load factor on $\boldsymbol{E}$ would be required. The load effect $\boldsymbol{E}$ in model building codes and design load reference standards includes the effect of both horizontal and

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vertical ground motions (as $\boldsymbol{E_h}$ and $\boldsymbol{E_v}$, respectively). The effect for vertical ground motions is applied as an addition to or subtraction from the dead load effect (***D***), and it applies to all structural elements, whether part of the seismic force-resisting system or not, unless specifically excluded by the general building code.

Structures other than buildings may require load factors and combinations different from those given in Table 5.3.1. For such structures, the appropriate load factors and combinations may be obtained from relevant codes and standards such as ASCE/SEI 7 and AASHTO LRFD for bridge structures.

5.3.2 The effect of one or more loads not acting simultaneously shall be investigated.

5.3.3 The load factor on live load ***L*** in Eq. (5.3.1c), (5.3.1d), and (5.3.1e) shall be permitted to be reduced to 0.5 except for (a), (b), or (c):

(a) Garages

(b) Areas occupied as places of public assembly

(c) Areas where ***L*** is greater than 4.8 kN/m^2

R5.3.3 The load modification factor in this provision is different than the live load reductions based on the loaded area that may be allowed in the general building code. The live load reduction, based on loaded area, adjusts the nominal live load ($\boldsymbol{L_0}$ in ASCE/SEI 7) to ***L***. The live load reduction, as specified in the general building code, can be used in combination with the 0.5 load factor specified in this provision.

5.3.4 If applicable, ***L*** shall include (a) through (f):

(a) Concentrated live loads

(b) Vehicular loads

(c) Crane loads

(d) Loads on handrails, guardrails, and vehicular barrier systems

(e) Impact effects

(f) Vibration effects

5.3.5 If wind load ***W*** is based on service-level loads, **1.6*W*** shall be used in place of **1.0*W*** in Eq. (5.3.1d) and (5.3.1f), and **0.8*W*** shall be used in place of **0.5*W*** in Eq. (5.3.1c).

R5.3.5 In ASCE/SEI 7-05, wind loads are consistent with service-level design; a wind load factor of 1.6 is appropriate for use in Eq. (5.3.1d) and (5.3.1f) and a wind load factor of 0.8 is appropriate for use in Eq. (5.3.1c). ASCE/SEI 7-16 prescribes wind loads for strength-level design and the wind load factor is 1.0. Design wind speeds for strength-level design are based on storms with mean recurrence intervals of 300, 700, and 1700 years, depending on the risk category of the structure. The higher load factors in 5.3.5 apply where service-level wind loads corresponding to a 50-year mean recurrence interval are used for design.

5.3.6 The structural effects of forces due to restraint of volume change and differential settlement ***T*** shall be considered in combination with other loads if the effects of ***T*** can adversely affect structural safety or performance. The load factor for ***T*** shall be established considering the uncertainty associated with the likely magnitude of ***T***, the probability that the maximum effect of ***T*** will occur simultaneously with other applied loads, and the potential adverse consequences if the effect of ***T*** is greater than assumed. The load factor on ***T*** shall not have a value less than 1.0.

R5.3.6 Several strategies can be used to accommodate movements due to volume change and differential settlement. Restraint of such movements can cause significant member forces and moments, such as tension in slabs and shear forces and moments in vertical members. Forces due to ***T*** effects are not commonly calculated and combined with other load effects. Rather, designs rely on successful past practices using compliant structural members and ductile connections to accommodate differential settlement and volume change movement while providing the needed

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resistance to gravity and lateral loads. Expansion joints and construction closure strips are used to limit volume change movements based on the performance of similar structures. Shrinkage and temperature reinforcement, which may exceed the required flexural reinforcement, is commonly proportioned based on gross concrete area rather than calculated force.

Where structural movements can lead to damage of nonductile elements, calculation of the predicted force should consider the inherent variability of the expected movement and structural response.

A long-term study of the volume change behavior of precast concrete buildings (Klein and Lindenberg 2009) recommends procedures to account for connection stiffness, thermal exposure, member softening due to creep, and other factors that influence T forces.

Fintel et al. (1986) provides information on the magnitudes of volume change effects in tall structures and recommends procedures for including the forces resulting from these effects in design.

5.3.7 If fluid load ***F*** is present, it shall be included in the load combination equations of 5.3.1 in accordance with (a), (b), (c) or (d):

(a) If ***F*** acts alone or adds to the effects of ***D***, it shall be included with a load factor of 1.4 in Eq. (5.3.1a).

(b) If ***F*** adds to the primary load, it shall be included with a load factor of 1.2 in Eq. (5.3.1b) through (5.3.1e).

(c) If the effect of ***F*** is permanent and counteracts the primary load, it shall be included with a load factor of 0.9 in Eq. (5.3.1g).

(d) If the effect of ***F*** is not permanent but, when present, counteracts the primary load, ***F*** shall not be included in Eq. (5.3.1a) through (5.3.1g).

5.3.8 If lateral earth pressure ***H*** is present, it shall be included in the load combination equations of 5.3.1 in accordance with (a), (b), or (c):

(a) If ***H*** acts alone or adds to the primary load effect, it shall be included with a load factor of 1.6.

(b) If the effect of ***H*** is permanent and counteracts the primary load effect, it shall be included with a load factor of 0.9.

(c) If the effect of ***H*** is not permanent but, when present, counteracts the primary load effect, ***H*** shall not be included.

R5.3.8 The required load factors for lateral pressures from soil, water in soil, and other materials, reflect their variability and the possibility that the materials may be removed. The commentary of ASCE/SEI 7 includes additional useful discussion pertaining to load factors for ***H***.

5.3.9 If a structure is in a flood zone, the flood loads and the appropriate load factors and combinations of ASCE/SEI 7 shall be used.

R5.3.9 Areas subject to flooding are defined by flood hazard maps, usually maintained by local governmental jurisdictions.

5.3.10 If a structure is subjected to forces from atmospheric ice loads, the ice loads and the appropriate load factors and combinations of ASCE/SEI 7 shall be used.

R5.3.10 Ice buildup on a structural member increases the applied load and the projected area exposed to wind. ASCE/SEI 7 provides maps of probable ice thicknesses due to freezing rain, with concurrent 3-second gust speeds, for a 50-year return period.

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5.3.11 Intentionally left blank.

5.3.12 Intentionally left blank.

5.3.13 Intentionally left blank.

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CHAPTER 6—STRUCTURAL ANALYSIS

6.1—Scope

=**6.1.1** This Chapter shall apply to methods of analysis, modeling of members and structural systems, and calculation of load effects.

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CHAPTER R6—STRUCTURAL ANALYSIS

R6.1—Scope

The provisions of this chapter apply to analyses used to determine load effects for design.

Section 6.2 provides general requirements that are applicable for analysis procedures.

Section 6.2.4 directs the licensed design professional to specific analysis provisions that are not contained in this chapter. Section 6.2.4.1 identifies analysis provisions that are specific to two-way slabs.

Section 6.3 addresses modeling assumptions used in establishing the analysis model.

Section 6.4 prescribes the arrangements of live loads that are to be considered in the analysis.

Section 6.5 provides a simplified method of analysis for continuous beams and one-way slabs that can be used in place of a more rigorous analysis when the stipulated conditions are satisfied.

Section 6.6 includes provisions for a comprehensive first-order analysis. The effect of cracked sections and creep are included in the analysis through the use of effective stifnesses.

Section 6.7 includes provisions for an elastic second-order analysis. Inclusion of the effects of cracking and creep is required. Inelastic analyses are not addressed in this Code.

Section 6.9 includes provisions for the use of the finite element method.

6.2—General

=**6.2.1** Members and structural systems shall be permitted to be modeled in accordance with 6.3.

=**6.2.2** All members and structural systems shall be analyzed for the maximum effects of loads including the arrangements of live load in accordance with 6.4.

6.2.3 Methods of analysis permitted by this chapter shall be (a) through (e). Redistribution of moments calculated in accordance with (a) through (e) is not permitted.

(a) The simplified method for analysis of continuous beams and one-way slabs for gravity loads in 6.5
(b) Linear elastic first-order analysis in 6.6
(c) Linear elastic second-order analysis in 6.7
(d) Intentionally left blank
(e) Finite element analysis in 6.9

R6.2—General

R6.2.3 GFRP reinforcement is linear elastic until failure; plastic hinge regions associated with moment redistribution do not form.

A first-order analysis satisfies the equations of equilibrium using the original undeformed geometry of the structure. When only first-order results are considered, slenderness effects are not accounted for. Because these effects can be important, 6.6 provides procedures to calculate both individual member slenderness ($P\delta$) effects and sidesway ($P\Delta$) effects for the overall structure using the first-order results.

A second-order analysis satisfies the equations of equilibrium using the deformed geometry of the structure. If the second-order analysis uses nodes along compression members, the analysis accounts for slenderness effects due to lateral deformations along individual members, as well as sidesway of the overall structure. If the second-order analysis uses nodes at the member intersections only, the analysis captures the sidesway effects for the overall structure but neglects individual member slenderness effects. In this case, the moment magnifier method (6.6.4) is used to determine individual member slenderness effects.

6.2.4 Additional analysis methods that are permitted include 6.2.4.1.

6.2.4.1 Two-way slabs shall be permitted to be analyzed for gravity loads in accordance with (a) or (b):

(a) Direct design method for slabs

(b) Equivalent frame method for slabs

R6.2.4.1 ACI 318 Code editions from 1971 to 2014 contained provisions for use of the direct design method and the equivalent frame method. These methods are well-established and are covered in available texts. These provisions for gravity load analysis of two-way slabs have been removed from ACI 318 because they are considered to be only two of several analysis methods currently used for the design of two-way slabs. The direct design method and the equivalent frame method of ACI 318M-14, however, may still be used for the analysis of two-way slabs for gravity loads, with the exception that ACI 318M-14 Section 8.10.4.3 does not apply to GFRP-reinforced concrete two-way slabs.

6.2.4.2 Intentionally left blank.

6.2.4.3 Intentionally left blank.

6.2.4.4. Intentionally left blank.

6.2.5 *Slenderness effects*

6.2.5.1 Slenderness effects shall be permitted to be neglected if (a) or (b) is satisfied:

(a) For columns not braced against sidesway

$$k\ell_u/r \leq 17 \tag{6.2.5.1a}$$

(b) For columns braced against sidesway

$$k\ell_u/r \leq 29 + 12(M_1/M_2) \tag{6.2.5.1b}$$

and

$$k\ell_u/r \leq 35 \tag{6.2.5.1c}$$

where $\boldsymbol{M_1/M_2}$ is negative if the column is bent in single curvature, and positive for double curvature.

If bracing elements resisting lateral movement of a story have a total stiffness of at least 12 times the gross lateral stiffness of the columns in the direction considered, it shall be permitted to consider columns within the story to be braced against sidesway.

R6.2.5 *Slenderness effects*

R6.2.5.1 Second-order effects in many structures are negligible. In these cases, it is unnecessary to consider slenderness effects, and compression members, such as columns, walls, or braces, can be designed based on forces determined from first-order analyses. Slenderness effects can be neglected in both braced and unbraced systems, depending on the slenderness ratio ($\boldsymbol{k\ell_u/r}$) of the member.

$\boldsymbol{M_1/M_2}$ is negative if bent in single curvature and positive if bent in double curvature.

The primary design aid to estimate the effective length factor $\boldsymbol{k}$ is the Jackson and Moreland Alignment Charts (Fig. R6.2.5.1), which provide a graphical determination of $\boldsymbol{k}$ for a column of constant cross section in a multi-bay frame (ACI SP-17(09); Column Research Council 1966).

The slenderness ratio limit given in Eq. (6.2.5.1a) is based on a study conducted by Mirmiran et al. (2001) that recognized that the use of low-stiffness GFRP bars makes columns more susceptible to slenderness effects. A parametric study considering more than 11,000 columns was conducted with different reinforcement ratios, modular ratios, strength ratios, compressive/tensile strength ratios, yielding response of reinforcing bars, slenderness ratios, end eccentricities, and eccentricity ratios. Based on the parametric study, the slenderness ratio limit of 22 used for steel-reinforced concrete columns bent in single curvature has been reduced to 17 for GFRP-reinforced concrete columns.

The slenderness ratio limits given in Eq. (6.2.5.1b) and (6.2.5.1c) for columns braced against sidesway are based on Eq. (6.6.4.5.1), allowing for a 5% increase in moments due

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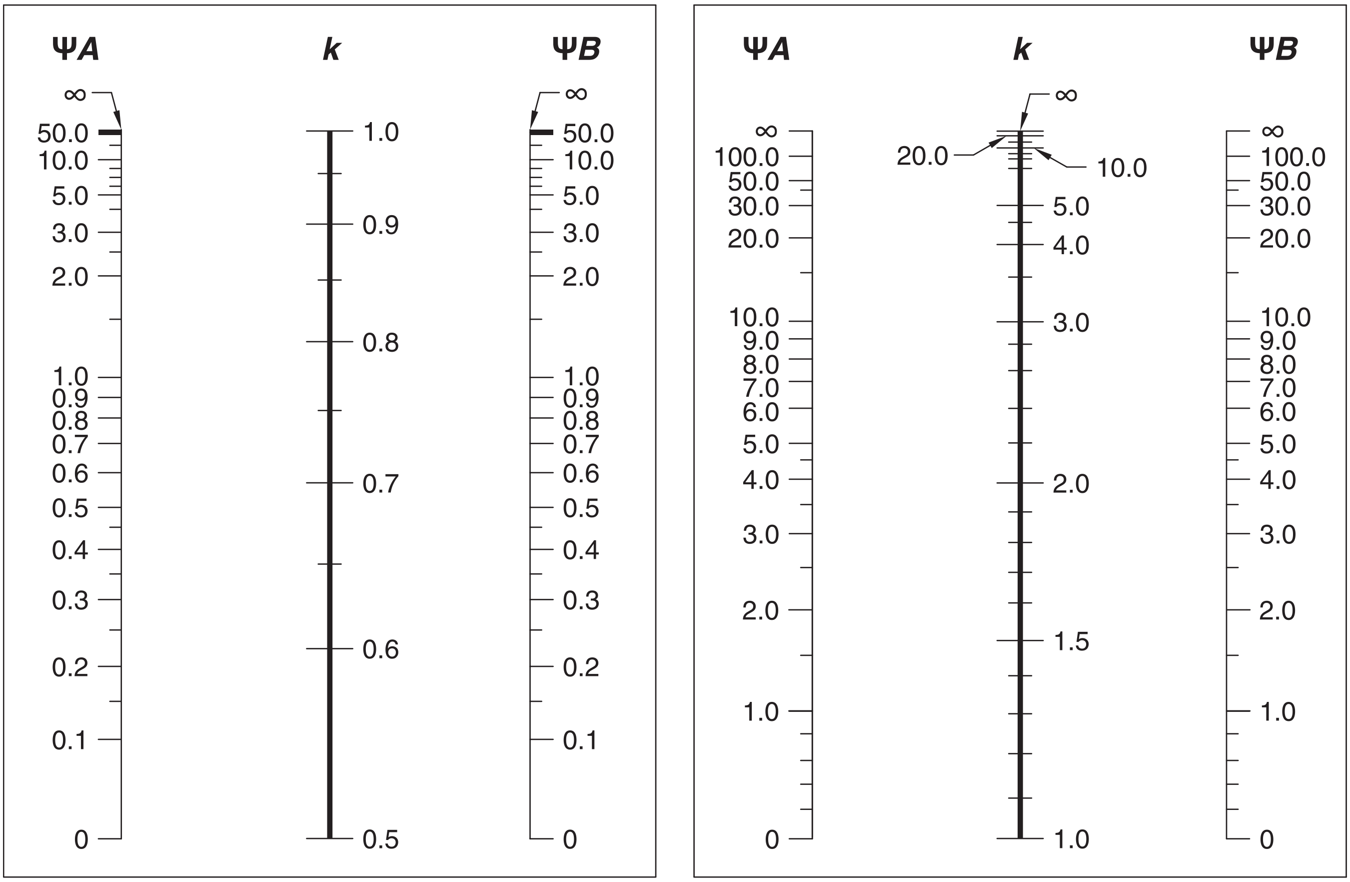

(a)
Nonsway frames

(b)
Sway frames

Ψ = ratio of $\sum(\boldsymbol{EI}/\ell_c)$ of all columns to $\sum(\boldsymbol{EI}/\ell)$ of beams in a plane at one end of a column

ℓ = span length of of beam measured center to center of joints

Fig. R6.2.5.1—Effective length factor **k**.

to slenderness to be neglected, and are lower than the corresponding limits in ACI 318 for steel-reinforced concrete (Jawaheri Zadeh and Nanni 2013, 2017). Jawaheri Zadeh and Nanni (2017) compared the stiffness of steel-reinforced and GFRP-reinforced concrete columns at service level when both steel and GFRP behave linearly. They retraced the provisions and assumptions of ACI 318 in their work, which accounted for the modulus of elasticity of tensile GFRP reinforcement and its reduced effectiveness in compression. The flexural stiffness of GFRP-reinforced concrete columns may be approximated as 60% of steel-reinforced concrete columns, both for first-order analysis and second-order effects. However, this value is only recommended if the frame is subject to gravity loads (nonsway frame). The use of GFRP-reinforced concrete columns as part of the lateral-force-resisting system requires evaluation and validation by seismic investigations and was not covered in this study.

The stiffness of the lateral bracing is considered based on the principal directions of the framing system. Bracing

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⁼**6.2.5.2** The radius of gyration, $\boldsymbol{r}$, shall be permitted to be calculated by (a), (b), or (c):

(a) $$r = \sqrt{\frac{I_g}{A_g}} \quad (6.2.5.2)$$

(b) 0.30 times the dimension in the direction stability is being considered for rectangular columns

(c) 0.25 times the diameter of circular columns

6.2.5.3 Unless slenderness effects are neglected as permitted by 6.2.5.1, the design of columns, restraining beams, and other supporting members shall be based on the factored forces and moments considering second-order effects in accordance with 6.6.4 or 6.7. $\boldsymbol{M_u}$ including second-order effects shall not exceed $\mathbf{1.4}\boldsymbol{M_u}$ due to first-order effects.

6.3—Modeling assumptions

6.3.1 *General*

⁼**6.3.1.1** Relative stiffnesses of members within structural systems shall be based on reasonable and consistent assumptions. The assumptions shall be consistent throughout each analysis.

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elements in typical building structures consist of shear walls or lateral braces. Torsional response of the lateral-force-resisting system due to eccentricity of the structural system can increase second-order effects and should be considered.

R6.2.5.3 Design considering second-order effects may be based on the moment magnifier approach (MacGregor et al. 1970; MacGregor 1993; Ford et al. 1981) or an elastic second-order analysis. Figure R6.2.5.3 is intended to assist designers with application of the slenderness provisions of the Code.

End moments in compression members, such as columns, walls, or braces, should be considered in the design of adjacent flexural members. In nonsway frames, the effects of magnifying the end moments need not be considered in the design of adjacent beams. In sway frames, the magnified end moments should be considered in designing the adjoining flexural members.

If the weight of a structure is high in proportion to its lateral stiffness, excessive $\boldsymbol{P}\Delta$ effects, where secondary moments are more than 25% of the primary moments, may result. The $\boldsymbol{P}\Delta$ effects will eventually introduce singularities into the solution to the equations of equilibrium, indicating physical structural instability (Wilson 1997). Analytical research (MacGregor and Hage 1977) on reinforced concrete frames showed that the probability of stability failure increases rapidly when the stability index $\boldsymbol{Q}$, defined in 6.6.4.4.1, exceeds 0.2, which is equivalent to a secondary-to-primary moment ratio of 1.25. According to ASCE/SEI 7, the maximum value of the stability coefficient $\boldsymbol{\theta}$, which is close to the ACI stability coefficient $\boldsymbol{Q}$, is 0.25. The value 0.25 is equivalent to a secondary-to-primary moment ratio of 1.33. Hence, the upper limit of 1.4 on the secondary-to-primary moment ratio was chosen.

R6.3—Modeling assumptions

R6.3.1 *General*

R6.3.1.1 Separate analyses with different stiffness assumptions may be performed for different objectives such as to check serviceability and strength criteria or to bound the demands on elements where stiffness assumptions are critical.

Ideally, the member stiffnesses $\boldsymbol{E_cI}$ and $\boldsymbol{GJ}$ should reflect the degree of cracking that has occurred along each member. However, the complexities involved in selecting different

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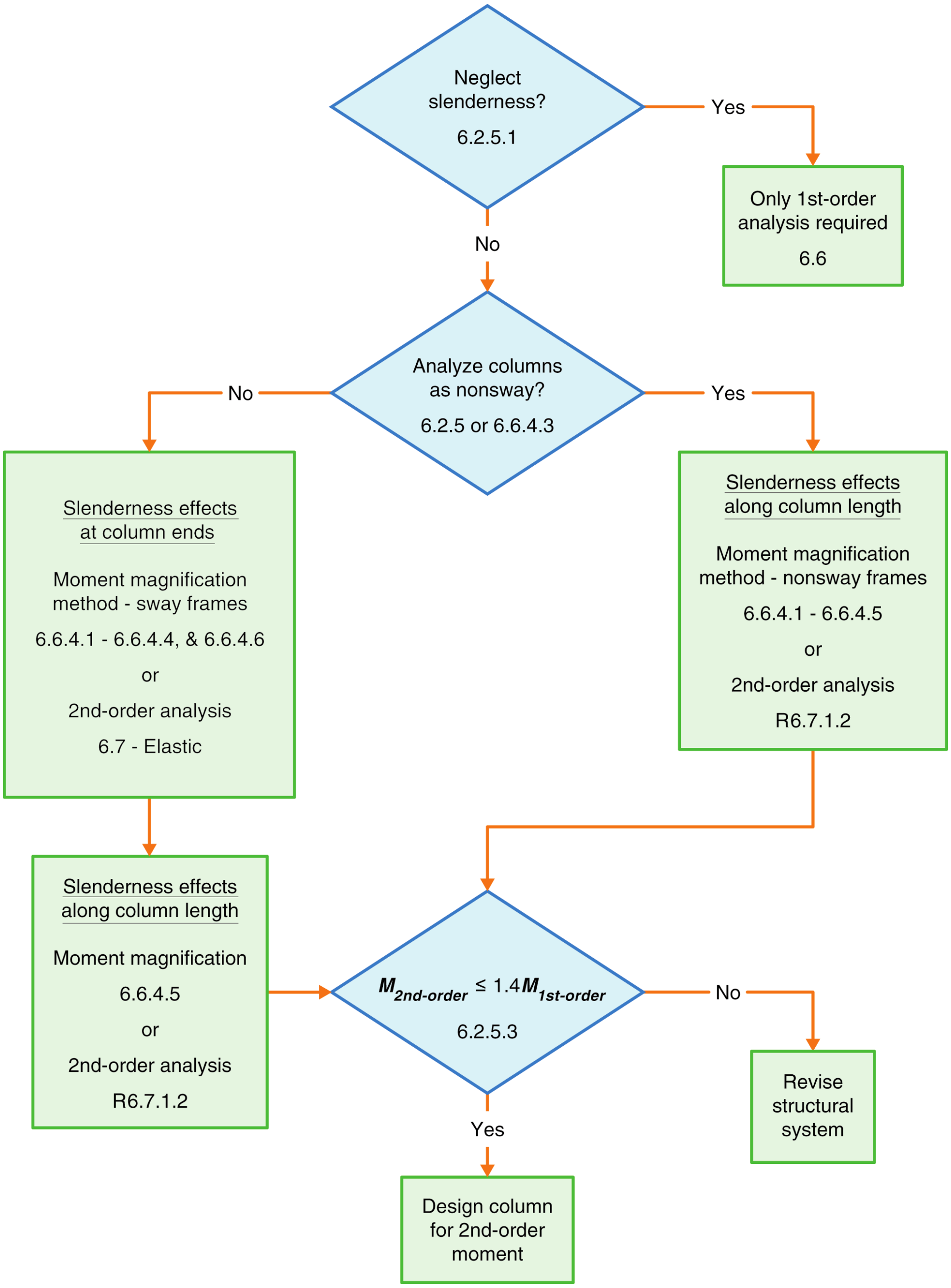

Fig. R6.2.5.3—Flowchart for determining column slenderness effects.

stiffnesses for all members of a frame would make frame analyses inefficient in the design process. Simpler assumptions are required to define flexural and torsional stiffnesses.

For braced frames, relative values of stiffness are important. A common assumption for steel-reinforced concrete is to use $\mathbf{0.5}\boldsymbol{I_g}$ for beams and $\boldsymbol{I_g}$ for columns. For GFRP-reinforced concrete, $\mathbf{0.22}\boldsymbol{I_g}$ can be assumed for beams and $\mathbf{0.6}\boldsymbol{I_g}$ for columns (Bischoff 2017). For sway frames, a realistic estimate of $\boldsymbol{I}$ is desirable and should be used if second-order analyses are performed. Guidance for the choice of $\boldsymbol{I}$ is given in 6.6.3.1.

Two conditions determine whether it is necessary to consider torsional stiffness in the analysis of a given structure: 1) the relative magnitude of the torsional and flexural

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stiffnesses; and 2) whether torsion is required for equilibrium of the structure (equilibrium torsion) or is due to members twisting to maintain deformation compatibility (compatibility torsion). In the case of equilibrium torsion, torsional stiffness should be included in the analysis. It is, for example, necessary to consider the torsional stiffnesses of edge beams. In the case of compatibility torsion, torsional stiffness usually is not included in the analysis. This is because the cracked torsional stiffness of a beam is a small fraction of the flexural stiffness of the members framing into it. Torsion should be considered in design as required in Chapter 9.

6.3.1.2 To calculate moments and shears caused by gravity loads in columns, beams, and slabs, it shall be permitted to use a model limited to the members in the level being considered and the columns above and below that level. It shall be permitted to assume far ends of columns built integrally with the structure to be fixed.

6.3.1.3 The analysis model shall consider the effects of variation of member cross-sectional properties, such as that due to haunches.

6.3.2 *T-beam geometry*

6.3.2.1 For T-beams supporting monolithic or composite slabs, the effective flange width b_f shall include the beam web width b_w plus an effective overhanging flange width in accordance with Table 6.3.2.1, where h is the slab thickness and s_w is the clear distance to the adjacent web.

Table 6.3.2.1—Dimensional limits for effective overhanging flange width for T-beams

Flange location	Effective overhanging flange width, beyond face of web	
Each side of web	Least of:	$8h$
		$s_w/2$
		$\ell_n/8$
One side of web	Least of:	$6h$
		$s_w/2$
		$\ell_n/12$

6.3.2.2 Isolated T-beams in which the flange is used to provide additional compression area shall have a flange thickness greater than or equal to $0.5b_w$ and an effective flange width less than or equal to $4b_w$.

6.3.2.3 Intentionally left blank.

6.4—Arrangement of live load

6.4.1 For the design of floors or roofs to resist gravity loads, it shall be permitted to assume that live load is applied only to the level under consideration.

R6.4—Arrangement of live load

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6.4.2 For one-way slabs and beams, it shall be permitted to assume (a) and (b):

(a) Maximum positive M_u near midspan occurs with factored L on the span and on alternate spans

(b) Maximum negative M_u at a support occurs with factored L on adjacent spans only

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R6.4.2 The most demanding sets of design forces should be established by investigating the effects of live load placed in various critical patterns.

6.4.3 For two-way slab systems, factored moments shall be calculated in accordance with 6.4.3.1, 6.4.3.2, or 6.4.3.3, and shall be at least the moments resulting from factored L applied simultaneously to all panels.

6.4.3.1 If the arrangement of L is known, the slab system shall be analyzed for that arrangement.

6.4.3.2 If L is variable and does not exceed $0.75D$, or the nature of L is such that all panels will be loaded simultaneously, it shall be permitted to assume that maximum M_u at all sections occurs with factored L applied simultaneously to all panels.

6.4.3.3 For loading conditions other than those defined in 6.4.3.1 or 6.4.3.2, it shall be permitted to assume (a) and (b):

(a) Maximum positive M_u near midspan of panel occurs with factored L on the panel and alternate panels

(b) Maximum negative M_u at a support occurs with factored L on adjacent panels only

6.5—Simplified method of analysis for continuous beams and one-way slabs

6.5.1 It shall be permitted to calculate M_u and V_u due to gravity loads in accordance with this section for continuous beams and one-way slabs satisfying (a) through (e):

(a) Members are prismatic

(b) Loads are uniformly distributed

(c) $L \leq 3D$

(d) There are at least two spans

(e) The longer of two adjacent spans does not exceed the shorter by more than 20%

6.5.2 M_u due to gravity loads shall be calculated in accordance with Table 6.5.2.

R6.5—Simplified method of analysis for continuous beams and one-way slabs

R6.5.2 The approximate moments and shears give reasonable values for the stated conditions if the continuous beams and one-way slabs are part of a frame or continuous construction. Because the load patterns that produce critical values for moments in columns of frames differ from those for maximum negative moments in beams, column moments should be evaluated separately.

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Table 6.5.2—Approximate moments for continuous beams and one-way slabs

Moment	Location	Condition	M_u
Positive	End span	Discontinuous end integral with support	$w_u\ell_n^2/14$
		Discontinuous end unrestrained	$w_u\ell_n^2/11$
	Interior spans	All	$w_u\ell_n^2/16$
Negative*	Interior face of exterior support	Member built integrally with supporting spandrel beam	$w_u\ell_n^2/24$
		Member built integrally with supporting column	$w_u\ell_n^2/16$
	Exterior face of first interior support	Two spans	$w_u\ell_n^2/9$
		More than two spans	$w_u\ell_n^2/10$
	Face of other supports	All	$w_u\ell_n^2/11$
	Face of all supports satisfying (a) or (b)	(a) slabs with spans not exceeding 10 ft (b) beams where ratio of sum of column stiffnesses to beam stiffness exceeds 8 at each end of span	$w_u\ell_n^2/12$

*To calculate negative moments, ℓ_n shall be the average of the adjacent clear span lengths.

6.5.3 Moments calculated in accordance with 6.5.2 shall not be redistributed.

6.5.4 V_u due to gravity loads shall be calculated in accordance with Table 6.5.4.

Table 6.5.4—Approximate shears for continuous beams and one-way slabs

Location	V_u
Exterior face of first interior support	$1.15w_u\ell_n/2$
Face of all other supports	$w_u\ell_n/2$

6.5.5 Floor or roof level moments shall be resisted by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffnesses considering conditions of restraint.

R6.5.5 This section is provided to make certain that moments are included in column design. The moment refers to the difference between the end moments of the members framing into the column and exerted at the column centerline.

6.6—First-order analysis

6.6.1 *General*

6.6.1.1 Slenderness effects shall be considered in accordance with 6.6.4 unless they are allowed to be neglected by 6.2.5.1.

6.6.1.2 Intentionally left blank.

6.6.2 *Modeling of members and structural systems*

R6.6—First-order analysis

R6.6.1 *General*

R6.6.1.1 When using linear elastic first-order analysis, slenderness effects are calculated using the moment magnifier approach (MacGregor et al. 1970; MacGregor 1993; Ford et al. 1981).

R6.6.2 *Modeling of members and structural systems*

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=**6.6.2.1** Floor or roof level moments shall be resisted by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffnesses and considering conditions of restraint.

=**6.6.2.2** For frames or continuous construction, consideration shall be given to the effect of floor and roof load patterns on transfer of moment to exterior and interior columns, and of eccentric loading due to other causes.

=**6.6.2.3** It shall be permitted to simplify the analysis model by the assumptions of (a), (b), or both:

(a) Solid slabs or one-way joist systems built integrally with supports, with clear spans not more than 10 ft, shall be permitted to be analyzed as continuous members on knife-edge supports with spans equal to the clear spans of the member and width of support beams otherwise neglected.

(b) For frames or continuous construction, it shall be permitted to assume the intersecting member regions are rigid.

6.6.3 *Section properties*

6.6.3.1 *Factored load analysis*

6.6.3.1.1 Moment of inertia and cross-sectional area of members shall be calculated in accordance with Table 6.6.3.1.1, unless a more rigorous analysis is used. If sustained lateral loads are present, I for columns and walls shall be divided by $(1 + \beta_{ds})$, where β_{ds} is the ratio of maximum factored sustained shear within a story to the maximum factored shear in that story associated with the same load combination.

Table 6.6.3.1.1—Moment of inertia and cross-sectional area permitted for elastic analysis at factored load level

<table>
<tr><th colspan="2">Member and condition</th><th>Moment of inertia</th><th>Cross-sectional area for axial deformations</th><th>Cross-sectional area for shear deformations</th></tr>
<tr><td colspan="2">Columns</td><td>$0.4I_g$</td><td rowspan="5">$1.0A_g$</td><td rowspan="5">b_wh</td></tr>
<tr><td rowspan="2">Walls</td><td>Uncracked</td><td>$0.4I_g$</td></tr>
<tr><td>Cracked</td><td>$0.15I_g$</td></tr>
<tr><td colspan="2">Beams</td><td>$0.15I_g$</td></tr>
<tr><td colspan="2">Flat plates and flat slabs</td><td>$0.15I_g$</td></tr>
</table>

6.6.3.1.2 Intentionally left blank.

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=**R6.6.2.1** This section is provided to make certain that moments are included in column design if members have been proportioned using 6.5.1 and 6.5.2. The moment refers to the difference between the end moments of the members framing into the column and exerted at the column centerline.

=**R6.6.2.3** A common feature of modern frame analysis software is the assumption of rigid connections. Section 6.6.2.3(b) is intended to apply to intersecting elements in frames, such as beam-column joints.

R6.6.3 *Section properties*

R6.6.3.1 *Factored load analysis*

For lateral load analysis, the stiffnesses presented in 6.6.3.1.1 can be used. In general, for effective section properties, E_c may be calculated or specified in accordance with 19.2.2, the shear modulus may be taken as $0.4E_c$, and areas may be taken as given in Table 6.6.3.1.1.

R6.6.3.1.1 The moments of inertia in Table 6.6.3.1.1 are taken from Bischoff (2017) for elastic analysis at factored load levels taking into account the expected range of reinforcing ratios and elastic modulus of GFRP reinforcement. Jawaheri Zadeh and Nanni (2013, 2017) also provide information on flexural stiffness in frame analysis for GFRP-reinforced concrete.

The moment of inertia of T-beams should be based on the effective flange width defined in 6.3.2.1 or 6.3.2.2. It is generally sufficiently accurate to take I_g of a T-beam as $2I_g$ for the web, $2(b_wh^3/12)$.

If the factored moments and shears from an analysis based on the moment of inertia of a wall, taken equal to $0.40I_g$, indicate that the wall will crack in flexure, based on the modulus of rupture, the analysis should be repeated with $I = 0.15I_g$ in those stories where cracking is predicted using factored loads.

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6.6.3.1.3 Intentionally left blank.

6.6.3.2 *Service load analysis*

6.6.3.2.1 Immediate and time-dependent deflections due to gravity loads shall be calculated in accordance with 24.2.

6.6.3.2.2 It shall be permitted to calculate immediate lateral deflections using a moment of inertia of 1.5 times I defined in 6.6.3.1, or using a more detailed analysis, but the value shall not exceed I_g.

6.6.4 *Slenderness effects, moment magnification method*

6.6.4.1 Unless 6.2.5.1 is satisfied, columns and stories in structures shall be designated as being nonsway or sway. Analysis of columns in nonsway frames or stories shall be in accordance with 6.6.4.5. Analysis of columns in sway frames or stories shall be in accordance with 6.6.4.6.

6.6.4.2 The cross-sectional dimensions of each member used in an analysis shall be within 10% of the specified member dimensions in construction documents or the analysis shall be repeated.

6.6.4.3 It shall be permitted to analyze columns and stories in structures as nonsway frames if (a) or (b) is satisfied:

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R6.6.3.2 *Service load analysis*

R6.6.3.2.2 Analyses of deflections, vibrations, and building periods are needed at various service (unfactored) load levels (Grossman 1987, 1990) to determine the performance of the structure in service. The moments of inertia of the structural members in the service load analyses should be representative of the degree of cracking at the various service load levels investigated. Unless a more accurate estimate of the degree of cracking at service load level is available, it is satisfactory to use 1.5 times the moments of inertia provided in 6.6.3.1 (Bischoff 2017), not to exceed I_g, for service load analyses. Serviceability considerations for vibrations are discussed in R24.1.

R6.6.4 *Slenderness effects, moment magnification method*

R6.6.4.1 This section describes an approximate design procedure that uses the moment magnifier concept to account for slenderness effects. Moments calculated using a first-order frame analysis are multiplied by a moment magnifier that is a function of the factored axial load P_u and the critical buckling load P_c for the column. For the sway case, the moment magnifier is a function of the sum of P_u of the story and the sum of P_c of the sway-resisting columns in the story considered. Nonsway and sway frames are treated separately. A first-order frame analysis is an elastic analysis that excludes the internal force effects resulting from deflections.

The moment magnifier design method requires the designer to distinguish between nonsway frames, which are designed according to 6.6.4.5, and sway frames, which are designed according to 6.6.4.6. Frequently this can be done by comparing the total lateral stiffness of the columns in a story to that of the bracing elements. A compression member, such as a column, wall, or brace, may be assumed nonsway if it is located in a story in which the bracing elements (shear walls, shear trusses, or other types of lateral bracing) have such substantial lateral stiffness to resist the lateral deflections of the story that any resulting lateral deflection is not large enough to affect the column strength substantially. If not readily apparent without calculations, 6.6.4.3 provides two possible ways of determining if sway can be neglected.

R6.6.4.3 In 6.6.4.3(a), a story in a frame is classified as nonsway if the increase in the lateral load moments resulting from $P\Delta$ effects does not exceed 5% of the first-order

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(a) The increase in column end moments due to second-order effects does not exceed 5% of the first-order end moments
(b) $\boldsymbol{Q}$ in accordance with 6.6.4.4.1 does not exceed 0.05

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moments (MacGregor and Hage 1977). Section 6.6.4.3(b) provides an alternative method of determining if a frame is classified as nonsway based on the stability index for a story, $\boldsymbol{Q}$. In calculating $\boldsymbol{Q}$, $\sum \boldsymbol{P_u}$ should correspond to the lateral loading case for which $\sum \boldsymbol{P_u}$ is greatest. A frame may contain both nonsway and sway stories.

6.6.4.4 *Stability properties*

=**6.6.4.4.1** The stability index for a story, $\boldsymbol{Q}$, shall be calculated by:

$$Q = \frac{\sum P_u \Delta_o}{V_{us} \ell_c} \tag{6.6.4.4.1}$$

where $\sum \boldsymbol{P_u}$ and $\boldsymbol{V_{us}}$ are the total factored vertical load and horizontal story shear, respectively, in the story being evaluated, and Δ_o is the first-order relative lateral deflection between the top and the bottom of that story due to $\boldsymbol{V_{us}}$.

=**6.6.4.4.2** The critical buckling load $\boldsymbol{P_c}$ shall be calculated by:

$$P_c = \frac{\pi^2 (EI)_{eff}}{(k\ell_u)^2} \tag{6.6.4.4.2}$$

R6.6.4.4.2 In calculating the critical axial buckling load, the primary concern is the choice of a stiffness $\boldsymbol{(EI)_{eff}}$ that reasonably approximates the variations in stiffness due to cracking, creep, and nonlinearity of the concrete stress-strain curve. Section 6.6.4.4.4 may be used to calculate $\boldsymbol{(EI)_{eff}}$.

=**6.6.4.4.3** The effective length factor $\boldsymbol{k}$ shall be calculated using $\boldsymbol{E_c}$ in accordance with 19.2.2 and $\boldsymbol{I}$ in accordance with 6.6.3.1.1. For nonsway members, $\boldsymbol{k}$ shall be permitted to be taken as 1.0, and for sway members, $\boldsymbol{k}$ shall be at least 1.0.

=**R6.6.4.4.3** The effective length factor for a compression member, such as a column, wall, or brace, considering braced behavior, ranges from 0.5 to 1.0. It is recommended that a $\boldsymbol{k}$ value of 1.0 be used. If lower values are used, the calculation of $\boldsymbol{k}$ should be based on analysis of the frame using $\boldsymbol{I}$ values given in 6.6.3.1.1. The Jackson and Moreland Alignment Charts (Fig. R6.2.5.1) can be used to estimate appropriate values of $\boldsymbol{k}$ (ACI SP-17(09); Column Research Council 1966).

6.6.4.4.4 For columns, $\boldsymbol{(EI)_{eff}}$ shall be calculated in accordance with (a) or (b):

(a) $$(EI)_{eff} = \frac{0.24 E_c I_g}{1+\beta_{dns}} \tag{6.6.4.4.4a}$$

(b) $$(EI)_{eff} = \frac{0.2 E_c I_g}{1+\beta_{dns}} + 0.75 E_f I_f \tag{6.6.4.4.4b}$$

where $\boldsymbol{\beta_{dns}}$ shall be the ratio of maximum factored sustained axial load to maximum factored axial load associated with the same load combination and $\boldsymbol{I_f}$ in Eq. (6.6.4.4.4b) is calculated as the moment of inertia of the bars about the centroid of the cross section.

R6.6.4.4.4 Equations (6.6.4.4.4a) and (6.6.4.4.4b) for the effective stiffness of columns are developed in Jawaheri Zadeh and Nanni (2017). Creep due to sustained loads will increase the lateral deflections of a column and, hence, the moment magnification. Creep effects can be approximated by reducing the stiffness $\boldsymbol{(EI)_{eff}}$ used to calculate $\boldsymbol{P_c}$ and, hence, $\boldsymbol{\delta}$, by dividing the $\boldsymbol{E_cI_g}$ term in Eq. (6.6.4.4.4a and 6.6.4.4.4b) by $(\boldsymbol{1 + \beta_{dns}})$ (Jawaheri Zadeh and Nanni 2017). For simplification, it can be assumed that $\boldsymbol{\beta_{dns}} = 0.6$. In this case, Eq. (6.6.4.4.4a) becomes $\boldsymbol{(EI)_{eff} = 0.15E_cI_g}$.

6.6.4.5 *Moment magnification method: Nonsway frames*

R6.6.4.5 *Moment magnification method: Nonsway frames*

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6.6.4.5.1 The factored moment used for design of columns and walls, M_c, shall be the first-order factored moment M_2 amplified for the effects of member curvature.

$$M_c = \delta M_2 \tag{6.6.4.5.1}$$

6.6.4.5.2 Magnification factor **δ** shall be calculated by:

$$\delta = \frac{C_m}{1 - \dfrac{P_u}{0.75P_c}} \geq 1.0 \tag{6.6.4.5.2}$$

6.6.4.5.3 C_m shall be in accordance with (a) or (b):

(a) For columns without transverse loads applied between supports

$$C_m = 0.6 - 0.4\frac{M_1}{M_2} \tag{6.6.4.5.3a}$$

where M_1/M_2 is negative if the column is bent in single curvature, and positive if bent in double curvature. M_1 corresponds to the end moment with the lesser absolute value.

(b) For columns with transverse loads applied between supports.

$$C_m = 1.0 \tag{6.6.4.5.3b}$$

6.6.4.5.4 M_2 in Eq. (6.6.4.5.1) shall be at least $M_{2,min}$ calculated according to Eq. (6.6.4.5.4) about each axis separately.

$$M_{2,min} = P_u(15 + 0.03h) \tag{6.6.4.5.4}$$

If $M_{2,min}$ exceeds M_2, C_m shall be taken equal to 1.0 or calculated based on the ratio of the calculated end moments M_1/M_2, using Eq. (6.6.4.5.3a).

6.6.4.6 *Moment magnification method: Sway frames*

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R6.6.4.5.2 The 0.75 factor in Eq. (6.6.4.5.2) is the stiffness reduction factor ϕ_K, which is based on the probability of understrength of a single isolated slender column. Studies reported in Mirza et al. (1987) indicate that the stiffness reduction factor ϕ_K and the cross-sectional strength reduction ϕ-factors do not have the same values. These studies suggest the stiffness reduction factor ϕ_K for an isolated column should be 0.75 for both tied and spiral columns. In the case of a multistory frame, the column and frame deflections depend on the average concrete strength, which is higher than the strength of the concrete in the critical single understrength column. For this reason, the value of ϕ_K implicit in I values in 6.6.3.1.1 is 0.875.

R6.6.4.5.3 The factor C_m is a correction factor relating the actual moment diagram to an equivalent uniform moment diagram. The derivation of the moment magnifier assumes that the maximum moment is at or near midheight of the column. If the maximum moment occurs at one end of the column, design should be based on an equivalent uniform moment C_mM_2 that leads to the same maximum moment at or near midheight of the column when magnified (MacGregor et al. 1970).

The sign convention for M_1/M_2 follows the right-hand rule convention; hence, M_1/M_2 is negative if bent in single curvature and positive if bent in double curvature.

In the case of columns that are subjected to transverse loading between supports, it is possible that the maximum moment will occur at a section away from the end of the member. If this occurs, the value of the largest calculated moment occurring anywhere along the member should be used for the value of M_2 in Eq. (6.6.4.5.1). C_m is to be taken as 1.0 for this case.

R6.6.4.5.4 In the Code, slenderness is accounted for by magnifying the column end moments. If the factored column moments are small or zero, the design of slender columns should be based on the minimum eccentricity provided in Eq. (6.6.4.5.4). It is not intended that the minimum eccentricity be applied about both axes simultaneously.

The factored column end moments from the structural analysis are used in Eq. (6.6.4.5.3a) in determining the ratio M_1/M_2 for the column when the design is based on the minimum eccentricity. This eliminates what would otherwise be a discontinuity between columns with calculated eccentricities less than the minimum eccentricity and columns with calculated eccentricities equal to or greater than the minimum eccentricity.

R6.6.4.6 *Moment magnification method: Sway frames*

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6.6.4.6.1 Moments M_1 and M_2 at the ends of an individual column shall be calculated by (a) and (b).

(a) $M_1 = M_{1ns} + \delta_s M_{1s}$ (6.6.4.6.1a)

(b) $M_2 = M_{2ns} + \delta_s M_{2s}$ (6.6.4.6.1b)

6.6.4.6.2 The moment magnifier δ_s shall be calculated by (a), (b) or (c). If δ_s exceeds 1.5, only (b) or (c) shall be permitted:

(a) $\delta_s = \dfrac{1}{1-Q} \geq 1$ (6.6.4.6.2a)

(b) $\delta_s = \dfrac{1}{1-\dfrac{\Sigma P_u}{0.75\Sigma P_c}} \geq 1$ (6.6.4.6.2b)

(c) Second-order elastic analysis

where $\sum P_u$ is the summation of all the factored vertical loads in a story and $\sum P_u$ is the summation for all sway-resisting columns in a story. P_c is calculated using Eq. (6.6.4.4.2) with k determined for sway members from 6.6.4.4.3 and $(EI)_{eff}$ from 6.6.4.4.4 with β_{ds} substituted for β_{dns}.

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R6.6.4.6.1 The analysis described in this section deals only with plane frames subjected to loads causing deflections in that plane. If the lateral load deflections involve significant torsional displacement, the moment magnification in the columns farthest from the center of twist may be underestimated by the moment magnifier procedure.

R6.6.4.6.2 Three different methods are allowed for calculating the moment magnifier. These approaches include the Q method, the sum of P concept, and second-order elastic analysis.

(a) Q method:

The iterative $P\Delta$ analysis for second-order moments can be represented by an infinite series. The solution of this series is given by Eq. (6.6.4.6.2a) (MacGregor and Hage 1977). Lai and MacGregor (1983) show that Eq. (6.6.4.6.2a) closely predicts the second-order moments in a sway frame until δ_s exceeds 1.5.

The $P\Delta$ moment diagrams for deflected columns are curved, with Δ related to the deflected shape of the columns. Equation (6.6.4.6.2a) and most commercially available second-order frame analyses have been derived assuming that the $P\Delta$ moments result from equal and opposite forces of $P\Delta/\ell_c$ applied at the bottom and top of the story. These forces give a straight-line $P\Delta$ moment diagram. The curved $P\Delta$ moment diagrams lead to lateral displacements on the order of 15% larger than those from the straight-line $P\Delta$ moment diagrams. This effect can be included in Eq. (6.6.4.6.2a) by writing the denominator as $(1 - 1.15Q)$ rather than $(1 - Q)$. The 1.15 factor has been omitted from Eq. (6.6.4.6.2a) for simplicity.

If deflections have been calculated using service loads, Q in Eq. (6.6.4.6.2a) should be calculated in the manner explained in R6.6.4.3.

The Q factor analysis, which was derived and validated for steel-reinforced concrete, is based on deflections calculated using the I-values from 6.6.3.1.1. No additional ϕ factor is needed. Once the moments are established using Eq. (6.6.4.6.2a), selection of the cross sections of the columns involves the strength reduction factors ϕ from 21.2.2.

(b) Sum of P concept:

To check the effects of story stability, δ_s is calculated as an averaged value for the entire story based on use of $\sum P_u/\sum P_c$. This reflects the interaction of all sway-resisting columns in the story on the $P\Delta$ effects because the lateral deflection of all columns in the story should be equal in the absence of torsional displacements about a vertical axis. In addition, it is possible that a particularly slender individual column in a sway frame could have substantial midheight deflections, even if adequately braced against lateral end deflections by other columns in the story. Such a column is checked using 6.6.4.6.4.

The 0.75 in the denominator of Eq. (6.6.4.6.2b) is a stiffness reduction factor ϕ_K, as explained in R6.6.4.5.2.

In the calculation of $(EI)_{eff}$, β_{ds} will normally be zero for a sway frame because the lateral loads are generally of short

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duration. Sway deflections due to short-term loads, such as wind or earthquake, are a function of the short-term stiffness of the columns following a period of sustained gravity load.

For this case, the definition of β_{ds} in 6.6.3.1.1 gives β_{ds} **= 0**. In the unusual case of a sway frame where the lateral loads are sustained, β_{ds} will not be zero. This might occur if a building on a sloping site is subjected to earth pressure on one side but not on the other.

6.6.4.6.3 Flexural members shall be designed for the total magnified end moments of the columns at the joint.

R6.6.4.6.3 The strength of a sway frame is governed by stability of the columns and the degree of end restraint provided by the beams in the frame.

6.6.4.6.4 Second-order effects shall be considered along the length of columns in sway frames. It shall be permitted to account for these effects using 6.6.4.5, where C_m is calculated using M_1 and M_2 from 6.6.4.6.1.

R6.6.4.6.4 The maximum moment in a compression member, such as a column, wall, or brace, may occur between its ends. While second-order computer analysis programs may be used to evaluate magnification of the end moments, magnification between the ends may not be accounted for unless the member is subdivided along its length. The magnification may be evaluated using the procedure outlined in 6.6.4.5.

6.6.5 *Redistribution of moments in continuous flexural members*—Out of scope

6.7—Linear elastic second-order analysis

6.7.1 *General*

R6.7—Linear elastic second-order analysis

R6.7.1 *General*

In elastic second-order analyses, the deformed geometry of the structure is included in the equations of equilibrium so that $P\Delta$ effects are determined. The structure is assumed to remain elastic, but the effects of cracking and creep are considered by using a reduced stiffness EI. In contrast, elastic first-order analysis satisfies the equations of equilibrium using the original undeformed geometry of the structure and estimates $P\Delta$ effects by magnifying the column-end sway moments using Eq. (6.6.4.6.2a) or (6.6.4.6.2b).

6.7.1.1 A linear elastic second-order analysis shall consider the influence of axial loads, presence of cracked regions along the length of the member, and effects of load duration. These considerations are satisfied using the cross-sectional properties defined in 6.7.2.

R6.7.1.1 The stiffnesses EI used in an analysis for strength design should represent the stiffnesses of the members immediately prior to failure. This is particularly true for a second-order analysis that should predict the lateral deflections at loads approaching ultimate. The EI values should not be based solely on the moment-curvature relationship for the most highly loaded section along the length of each member. Instead, they should correspond to the moment-end rotation relationship for a complete member.

To allow for variability in the actual member properties in the analysis, the member properties used in analysis should be multiplied by a stiffness reduction factor ϕ_K less than 1. The cross-sectional properties defined in 6.7.2 already include this stiffness reduction factor. The stiffness reduction factor ϕ_K may be taken as 0.875. Note that the overall stiffness is further reduced considering that the modulus of elasticity of the concrete, E_c, is based on the specified concrete compressive strength, while the sway deflections

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are a function of the average concrete strength, which is typically higher.

6.7.1.2 Slenderness effects along the length of a column shall be considered. It shall be permitted to calculate these effects using 6.6.4.5.

R6.7.1.2 The maximum moment in a compression member may occur between its ends. In computer analysis programs, columns may be subdivided using nodes along their length to evaluate slenderness effects between the ends. If the column is not subdivided along its length, slenderness effects may be evaluated using the nonsway moment magnifier method specified in 6.6.4.5 with member-end moments from the second-order elastic analysis as input. Second-order analysis already accounts for the relative displacement of member ends.

6.7.1.3 The cross-sectional dimensions of each member used in an analysis to calculate slenderness effects shall be within 10% of the specified member dimensions in construction documents or the analysis shall be repeated.

6.7.1.4 Intentionally left blank.

6.7.2 *Section properties*

R6.7.2 *Section properties*

6.7.2.1 *Factored load analysis*

6.7.2.1.1 It shall be permitted to use section properties calculated in accordance with 6.6.3.1.

6.7.2.2 *Service load analysis*

R6.7.2.2 *Service load analysis*

6.7.2.2.1 Immediate and time-dependent deflections due to gravity loads shall be calculated in accordance with 24.2.

6.7.2.2.2 Alternatively, it shall be permitted to calculate immediate deflections using a moment of inertia of 1.5 times I defined in 6.6.3.1, or calculated using a more detailed analysis, but the value shall not exceed I_g.

R6.7.2.2.2 *Service load analysis*—Refer to R6.6.3.2.2.

6.8—Inelastic analysis—Out of scope

R6.8—Inelastic analysis

Inelastic analysis is not covered by this Code because of a lack of published experimental results on physical tests of GFRP-reinforced concrete components, subassemblages, or structural systems showing good agreement with methods of inelastic analysis.

6.9—Acceptability of finite element analysis

6.9.1 Finite element analysis to determine load effects shall be permitted.

6.9.2 The finite element model shall be appropriate for its intended purpose.

R6.9—Acceptability of finite element analysis

R6.9.1 This section is included to explicitly recognize a widely used analysis method.

R6.9.2 The licensed design professional should ensure that an appropriate analysis model is used for the particular problem of interest. This includes selection of computer software program, element type, model mesh, and other modeling assumptions.

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A great variety of finite element analysis computer software programs are available, including those that perform static, dynamic, elastic, and inelastic analysis.

The element types used should be capable of determining the response required. Finite element models may have beam-column elements that model structural framing members, such as beams and columns, along with plane stress elements; plate elements; and shell elements, brick elements, or both, that are used to model the floor slabs, mat foundations, diaphragms, walls, and connections. The model mesh size selected should be capable of determining the structural response in sufficient detail. The use of any set of reasonable assumptions for member stiffness is allowed.

=**6.9.3** For inelastic analysis, a separate analysis shall be performed for each factored load combination.

=**R6.9.3** For inelastic finite element analysis, the rules of linear superposition do not apply. To determine the ultimate member inelastic response, for example, it is not correct to analyze for service loads and subsequently combine the results linearly using load factors. A separate inelastic analysis should be performed for each factored load combination.

=**6.9.4** The licensed design professional shall confirm that the results are appropriate for the purposes of the analysis.

=**6.9.5** The cross-sectional dimensions of each member used in an analysis shall be within 10% of the specified member dimensions in construction documents or the analysis shall be repeated.

6.9.6 Intentionally left blank.

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CHAPTER 7—ONE-WAY SLABS

7.1—Scope

7.1.1 This chapter shall apply to the design of nonprestressed slabs reinforced for flexure in one direction, including:

(a) Solid slabs

(b) Slabs cast on stay-in-place noncomposite forms

(c) Composite slabs of concrete elements constructed in separate placements but connected so that all elements resist loads as a unit

7.2—General

=**7.2.1** The effects of concentrated loads, slab openings, and voids within the slab shall be considered in design.

7.2.2 *Materials*

=**7.2.2.1** Design properties for concrete shall be selected to be in accordance with Chapter 19.

7.2.2.2 Design properties for GFRP reinforcement shall be selected to be in accordance with Chapter 20.

=**7.2.2.3** Materials, design, and detailing requirements for embedments in concrete shall be in accordance with 20.6.

7.2.3 *Connection to other members*

=**7.2.3.1** For cast-in-place construction, beam-column and slab-column joints shall satisfy Chapter 15.

7.2.3.2 Intentionally left blank.

7.3—Design limits

7.3.1 *Minimum slab thickness*

7.3.1.1 Slab thickness shall be sufficient to satisfy the calculated deflection limits of 7.3.2.

7.3.1.2 Intentionally left blank.

7.3.2 *Calculated deflection limits*

7.3.2.1 Immediate and time-dependent deflections shall be calculated in accordance with 24.2 and shall not exceed the limits in 24.2.2.

7.3.2.2 Intentionally left blank.

7.3.3 *Reinforcement strain limit in nonprestressed slabs*—Not applicable

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CHAPTER R7—ONE-WAY SLABS

R7.1—Scope

R7.1.1 Provisions for one-way joist systems are provided in Chapter 9.

R7.2—General

=**R7.2.1** Concentrated loads and slab openings create local moments and shears and may cause regions of one-way slabs to have two-way behavior. The influence of openings through the slab and voids within the slab (for example, ducts) on flexural and shear strength as well as deflections is to be considered, including evaluating the potential for critical sections created by the openings and voids.

R7.3—Design limits

R7.3.2 *Calculated deflection limits*

The basis for calculated deflections for one-way slabs is the same as that for beams.

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7.3.4 *Stress limits in prestressed slabs*—Out of scope

7.3.5 *Sustained load stress limit*

7.3.5.1 GFRP reinforcement stresses due to the sustained portion of the service load shall satisfy the provisions of 24.6.

7.4—Required strength

7.4.1 *General*

=**7.4.1.1** Required strength shall be calculated in accordance with the factored load combinations in Chapter 5.

=**7.4.1.2** Required strength shall be calculated in accordance with the analysis procedures in Chapter 6.

7.4.1.3 Intentionally left blank.

7.4.2 *Factored moment*

=**7.4.2.1** For slabs built integrally with supports, M_u at the support shall be permitted to be calculated at the face of support.

7.4.3 *Factored shear*

=**7.4.3.1** For slabs built integrally with supports, V_u at the support shall be permitted to be calculated at the face of support.

7.4.3.2 Sections between the face of support and a critical section located d from the face of support shall be permitted to be designed for V_u at that critical section if (a) through (c) are satisfied:

(a) Support reaction, in direction of applied shear, introduces compression into the end region of the slab

(b) Loads are applied at or near the top surface of the slab

(c) No concentrated load occurs between the face of support and critical section

7.5—Design strength

7.5.1 *General*

=**7.5.1.1** For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$ including (a) and (b). Interaction between load effects shall be considered.

(a) $\phi M_n \geq M_u$

(b) $\phi V_n \geq V_u$

=**7.5.1.2** ϕ shall be determined in accordance with 21.2.

7.5.2 *Moment*

=**7.5.2.1** M_n shall be calculated in accordance with 22.3.

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R7.4—Required strength

R7.4.3 *Factored shear*

=**R7.4.3.2** The requirements for the selection of the critical section for shear in one-way slabs are the same as those for beams. Refer to R9.4.3.2 for additional information.

R7.5—Design strength

R7.5.1 *General*

=**R7.5.1.1** Refer to R9.5.1.1.

7.5.2 *Moment*

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7.5.2.2 Intentionally left blank.

7.5.2.3 If primary flexural reinforcement in a slab that is considered to be a T-beam flange is parallel to the longitudinal axis of the beam, reinforcement perpendicular to the longitudinal axis of the beam shall be provided in the top of the slab in accordance with (a) and (b). This provision does not apply to joist construction.

(a) Slab reinforcement perpendicular to the beam shall be designed to resist the factored load on the overhanging slab width assumed to act as a cantilever.

(b) Only the effective overhanging slab width in accordance with 6.3.2 need be considered.

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R7.5.2.3 This provision applies only where a T-beam is parallel to the span of a one-way slab. For example, this beam might be used to support a wall or concentrated load that the slab alone cannot support. In that case, the primary slab reinforcement is parallel to the beam and the perpendicular reinforcement is usually sized for temperature and shrinkage. The reinforcement required by this provision is intended to consider "unintended" negative moments that may develop over the beam that exceed the requirements for temperature and shrinkage reinforcement alone.

7.5.3 *Shear*

7.5.3.1 V_n shall be calculated in accordance with 22.5.

7.5.3.2 For composite concrete slabs, horizontal shear strength V_{nh} shall be calculated in accordance with 16.4

7.6—GFRP reinforcement limits

7.6.1 *Minimum GFRP flexural reinforcement*

7.6.1.1 A minimum area of flexural reinforcement, equal to or greater than the requirement for shrinkage and termpature reinforcement in 24.4.3.2 and $\frac{2.1}{f_{fu}}A_g$ shall be provided.

R7.6—GFRP reinforcement limits

R7.6.1 *Minimum GFRP flexural reinforcement*

R7.6.1.1 The limit of $\frac{2.1}{f_{fu}}A_g$ is based on the same requirements that apply to beams. If the required area of reinforcement to control shrinkage and temperature effects is greater than $\frac{2.1}{f_{fu}}A_g$, reinforcement in excess of $\frac{2.1}{f_{fu}}A_g$ may be distributed between the two faces of the slab as deemed appropriate for specific conditions.

7.6.2 *Minimum flexural reinforcement in prestressed slabs*—Out of scope

7.6.3 *Minimum GFRP shear reinforcement*

R7.6.3 *Minimum GFRP shear reinforcement*

The basis for minimum shear reinforcement for one-way slabs is the same as that for beams. Refer to R9.6.3 for additional information.

7.6.3.1 A minimum area of shear reinforcement, $A_{fv,min}$ shall be provided in all regions where $V_u > \phi V_c$.

R7.6.3.1 Solid slabs and footings have less stringent minimum shear reinforcement requirements than beams because there is a possibility of load sharing between weak and strong areas.

7.6.3.2 If shown by testing that the required M_n and V_n can be developed, 7.6.3.1 need not be satisfied. Such tests shall simulate effects of differential settlement, creep, shrinkage, and temperature change, based on a realistic assessment of these effects occurring in service.

R7.6.3.2 The basis for the testing-based strength evaluation for one-way slabs is the same as that for beams. Refer to R9.6.3.3 for additional information.

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7.6.3.3 If shear reinforcement is required, $A_{fv,min}$ shall be in accordance with 9.6.3.4.

7.6.4 *Minimum GFRP shrinkage and temperature reinforcement*

7.6.4.1 Reinforcement shall be provided to resist shrinkage and temperature stresses in accordance with 24.4.

7.6.4.2 Intentionally left blank.

7.7—GFRP reinforcement detailing

7.7.1 *General*

7.7.1.1 Concrete cover for reinforcement shall be in accordance with 20.5.1.

7.7.1.2 Development lengths of reinforcement shall be in accordance with 25.4.

7.7.1.3 Splices of reinforcement shall be in accordance with 25.5.

7.7.1.4 Intentionally left blank.

7.7.2 *GFRP reinforcement spacing*

7.7.2.1 Minimum spacing s shall be in accordance with 25.2.

7.7.2.2 Spacing of longitudinal reinforcement closest to the tension face shall not exceed s given in 24.3.

7.7.2.3 Maximum spacing s of reinforcement shall be the lesser of $3h$ and 450 mm.

7.7.2.4 Maximum spacing s of reinforcement required by 7.5.2.3 shall not exceed the lesser of $3h$ and 300 mm.

7.7.3 *GFRP flexural reinforcement*

7.7.3.1 Calculated tensile or compressive force in reinforcement at each section of the slab shall be developed on each side of that section.

7.7.3.2 Critical locations for development of reinforcement are points of maximum stress and points along the span where terminated tension reinforcement is no longer required to resist flexure.

7.7.3.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure and provide stiffness to satisfy deflection requirements for a distance at least the greater of d and $12d_b$, except at supports of simply-supported spans and at free ends of cantilevers.

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R7.7—GFRP reinforcement detailing

R7.7.2 *GFRP reinforcement spacing*

R7.7.2.4 The spacing limitations for slab reinforcement are based on flange thickness, which for tapered flanges can be taken as the average thickness.

R7.7.3 *GFRP flexural reinforcement*

Requirements for development of reinforcement in one-way slabs are similar to those for beams. Refer to R9.7.3 for additional information.

R7.7.3.3 GFRP-reinforced concrete slabs are more likely to have the amount of required reinforcement controlled by serviceability requirements than are steel-reinforced concrete slabs. In lieu of detailed deflection calculations, the point at which GFRP bars are no longer required to satisfy

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deflection requirements can be located at sections where the value of I_e, calculated from Table 24.2.3.5 using I_{cr} for the continuing bars and replacing M_a with the service moment at the cut-off location, is not less than the value of I_e calculated from Table 24.2.3.5 at the location of maximum moment.

7.7.3.4 Continuing flexural tension reinforcement shall have an embedment length at least ℓ_d beyond the point where terminated tension reinforcement is no longer required to resist flexure.

7.7.3.5 Flexural tension reinforcement shall not be terminated in a tension zone unless (a), (b), or (c) is satisfied:

(a) $V_u \leq (2/3)\phi V_n$ at the cutoff point.

(b) For No. M32 bars and smaller, continuing reinforcement provides double the area required for flexural strength at the cutoff point and $V_u \leq (3/4)\phi V_n$.

(c) Stirrup area in excess of that required for shear is provided along each terminated bar over a distance $3/4d$ from the cutoff point. Excess stirrup area shall not be less than $60b_w s/f_{ft}$. Spacing s shall not exceed $d/(8\beta_b)$.

=**7.7.3.6** Adequate anchorage shall be provided for tension reinforcement where reinforcement stress is not directly proportional to moment, such as in sloped, stepped, or tapered slabs, or where tension reinforcement is not parallel to the compression face.

7.7.3.7 Intentionally left blank.

7.7.3.8 *Termination of GFRP reinforcement*

=**7.7.3.8.1** At simple supports, at least one-third of the maximum positive moment reinforcement shall extend along the slab bottom into the support, except for precast slabs where such reinforcement shall extend at least to the center of the bearing length.

=**7.7.3.8.2** At other supports, at least one-fourth of the maximum positive moment reinforcement shall extend along the slab bottom into the support at least 150 mm.

7.7.3.8.3 At simple supports and points of inflection, d_b for positive moment tension reinforcement shall be limited such that ℓ_d for that reinforcement satisfies (a) or (b). If reinforcement terminates beyond the centerline of supports by a standard hook or a mechanical anchorage at least equivalent to a standard hook, (a) or (b) need not be satisfied.

(a) $\ell_d \leq (1.3M_n/V_u + \ell_a)$ if end of reinforcement is confined by a compressive reaction

(b) $\ell_d \leq (M_n/V_u + \ell_a)$ if end of reinforcement is not confined by a compressive reaction

M_n and V_u are calculated at the section. At a support, ℓ_a is the embedment length beyond the center of the support. At a point of inflection, ℓ_a is the embedment length beyond the point of inflection, limited to the greater of d and $12d_b$.

R7.7.3.8 *Termination of GFRP reinforcement*

Requirements for termination of reinforcement in one-way slabs are similar to those for beams. Refer to R9.7.3.8 for additional information.

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⁼**7.7.3.8.4** At least one-third of the negative moment reinforcement at a support shall have an embedment length beyond the point of inflection at least the greatest of $\boldsymbol{d}$, $\mathbf{12}\boldsymbol{d_b}$, and $\boldsymbol{\ell_n}/\mathbf{16}$.

7.7.4 *Flexural reinforcement in prestressed slabs*—Out of scope

7.7.5 *GFRP shear reinforcement*

⁼**7.7.5.1** If shear reinforcement is required, transverse reinforcement shall be detailed according to 9.7.6.2.

7.7.6 *GFRP shrinkage and temperature reinforcement*

⁼**7.7.6.1** Shrinkage and temperature reinforcement in accordance with 7.6.4 shall be placed perpendicular to flexural reinforcement.

7.7.6.2 Spacing of shrinkage and temperature reinforcement shall not exceed the lesser of $\mathbf{5}\boldsymbol{h}$ and 450 mm.

7.7.6.3 Intentionally left blank.

7.7.7 *GFRP structural integrity reinforcement in cast-in-place one-way slabs*

⁼**7.7.7.1** Longitudinal structural integrity reinforcement consisting of at least one-fourth of the maximum positive moment reinforcement shall be continuous.

7.7.7.2 Longitudinal structural integrity reinforcement at noncontinuous supports shall be anchored to develop $\boldsymbol{f_{fu}}$ at the face of the support.

7.7.7.3 If splices are necessary in continuous structural integrity reinforcement, the reinforcement shall be spliced near supports. Splices shall be mechanical in accordance with 25.5.7 or Class B tension lap splices in accordance with 25.5.2.

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R7.7.7 *GFRP structural integrity reinforcement in cast-in-place one-way slabs*

⁼Positive moment structural integrity reinforcement for one-way slabs is intended to be similar to that for beams. Refer to R9.7.7 for a discussion of structural integrity reinforcement for beams.

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CHAPTER 8—TWO-WAY SLABS

8.1—Scope

8.1.1 This chapter shall apply to the design of nonprestressed slabs reinforced for flexure in two directions, with or without beams between supports, including (a) through (c):

(a) Solid slabs

(b) Slabs cast on stay-in-place, noncomposite steel deck or FRP forms

(c) Composite slabs of concrete elements constructed in separate placements but connected so that all elements resist loads as a unit

8.2—General

8.2.1 A slab system shall be permitted to be designed by any procedure satisfying equilibrium and geometric compatibility, provided that design strength at every section is at least equal to required strength, and all serviceability requirements are satisfied. The direct design method or the equivalent frame method is permitted.

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CHAPTER R8—TWO-WAY SLABS

R8.1—Scope

The design methods given in this chapter are based on analysis of the results of an extensive series of tests with both steel-reinforced concrete (Burns and Hemakom 1977; Gamble et al. 1969; Gerber and Burns 1971; Guralnick and LaFraugh 1963; Hatcher et al. 1965, 1969; Hawkins 1981; Jirsa et al. 1966; PTI DC20.8; Smith and Burns 1974; Scordelis et al. 1959; Vanderbilt et al. 1969; Xanthakis and Sozen 1963) and GFRP-reinforced concrete (Ospina et al. 2003; El-Ghandour et al. 2003; Lee et al. 2009; Hassan et al. 2013a,b; Hassan et al. 2014, 2015; Gouda and El-Salakawy 2016a,b; El-Gendy and El-Salakawy 2016, 2018; Hassan et al. 2017; Mostafa and El-Salakawy 2018; Hussein and El-Salakawy 2018) two-way slabs and the well-established performance records of various steel-reinforced concrete slab systems. Ahmed et al. (2017) reports on the performance of a GFRP-reinforced concrete two-way slab parking garage three years after construction. The fundamental design principles are applicable to all planar structural systems subjected to transverse loads. Several specific design rules, as well as historical precedents, limit the types of structures to which this chapter applies. General slab systems that may be designed according to this chapter include flat slabs, flat plates, two-way slabs, and waffle slabs. Slabs with paneled ceilings are two-way, wide-band, beam systems.

Slabs-on-ground that do not transmit vertical loads from other parts of the structure to the soil are excluded.

For slabs with beams, the explicit design procedures of this chapter apply only when the beams are located at the edges of the panel and when the beams are supported by columns or other essentially nondeflecting supports at the corners of the panel. Two-way slabs with beams in one direction, with both slab and beams supported by girders in the other direction, may be designed under the general requirements of this chapter. Such designs should be based upon analysis compatible with the deflected position of the supporting beams and girders.

For slabs supported on walls, the explicit design procedures in this chapter treat the wall as a beam of infinite stiffness; therefore, each wall should support the entire length of an edge of the panel (refer to 8.4.1.7). Walls of width less than a full panel length can be treated as columns.

R8.2—General

R8.2.1 This section permits a design to be based directly on fundamental principles of structural mechanics, provided it can be demonstrated explicitly that all strength and serviceability criteria are satisfied. Serviceability limits related to deflections and control of crack widths often govern the design of GFRP-reinforced concrete two-way slabs. Creep rupture of the GFRP reinforcement should also be considered. The effects of creep rupture (static fatigue) are addressed in 24.6. The design of the slab may be achieved through the combined use of classic solutions based on a

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linearly elastic continuum, numerical solutions based on discrete elements, or analyses based on an energy-equivalent moment-curvature response including, in all cases, evaluation of the stress conditions around the supports in relation to shear, torsion, and flexure, as well as the effects of reduced stiffness of elements due to cracking and support geometry. The design of a slab system involves more than its analysis; any deviations in physical dimensions of the slab from common practice should be justified on the basis of knowledge of the expected loads and the reliability of the calculated stresses and deformations of the structure.

Although GFRP reinforcement behaves linearly up to failure without yielding, Gar et al. (2014) suggested the use of an energy-equivalent moment-curvature response in the form of an inelastic moment capacity for GFRP-reinforced concrete slabs that is analogous to the yielding moment capacity from a plastic analysis of steel-reinforced concrete slabs. The equivalent response for GFRP-reinforced concrete sections can be obtained by idealizing the flexural behavior of GFRP-reinforced concrete sections into a trilinear relationship, which is then simplified into an energy-equivalent bi-linear behavior similar to that of steel-reinforced concrete sections. This analogy has been validated against test results of GFRP-reinforced concrete slab-column connections (Gar et al. 2014; El-Gendy and El-Salakawy 2016; Gouda and El-Salakawy 2016a; Hussein and El-Salakawy 2018; Mostafa and El-Salakawy 2018).

The direct design method and the equivalent frame method are limited in application to orthogonal frames subject to gravity loads only.

The concept of moment redistribution, as it applies to the use of the direct design method or the equivalent frame method is well-established for continuous steel-reinforced concrete elements. If steel is used as reinforcement, most of the moment redistribution is usually attributed to the yielding of the reinforcement; however, studies of steel-reinforced concrete beams (do Carmo and Lopes 2008) have reported that some moment redistribution occurs before yielding of steel reinforcement at the critical sections, due to the difference in flexural stiffness from cracking along the member. Moment redistribution in excess of 18% in continuous GFRP-reinforced concrete beams has been reported by El-Mogy et al. (2010), Kara and Ashour (2013), and Rahman et al. (2017a,b). The observed moment redistribution was attributed to the relatively low modulus of elasticity of the GFRP bars making it possible to achieve the required section deformability for moment redistribution to occur, although not to the same extent as in continuous steel-reinforced concrete members. Redistribution of moments in two-way GFRP-reinforced concrete slabs is expected to occur to a greater degree than in GFRP-reinforced concrete beams due to the redundancy of the two-way action. Analysis methods developed for steel-reinforced concrete that rely on moment redistribution can thus be reasonably applied to continuous two-way GFRP-reinforced concrete structural elements, provided that the GFRP reinforcement can attain necessary strain levels to ensure sufficient deformability to allow for the moment redistribution to occur. Moment redistribution beyond that assumed in the direct design method or the equivalent frame method is not appropriate for GFRP-reinforced concrete slabs.

8.2.2 The effects of concentrated loads, slab openings, and voids shall be considered in design.

R8.2.2 Refer to R7.2.1.

8.2.3 Intentionally left blank.

8.2.4 A drop panel, where used to reduce the quantity of negative moment reinforcement at a support in accordance with 8.5.2.2, shall satisfy (a) through (c):

(a) The drop panel shall project below the slab at least one-fourth of the adjacent slab thickness.

(b) The drop panel shall extend in each direction from the centerline of support a distance not less than one-sixth the span length measured from center-to-center of supports in that direction.

(c) The drop panel shall extend in each direction from the centerline of support a distance not less than the development length for GFRP bars in compression in accordance with 25.4.9.

R8.2.4 and R8.2.5 Drop panel dimensions specified in 8.2.4 are necessary when reducing the amount of negative moment reinforcement following 8.5.2.2. If the dimensions are less than specified in 8.2.4, the projection may be used as a shear cap to increase the shear strength of the slab. For slabs with changes in thickness, it is necessary to check the shear strength at several sections (Refer to 22.6.4.1(b)).

8.2.5 A shear cap, where used to increase the critical section for shear at a slab-column joint, shall project below the slab soffit and extend horizontally from the face of the column a distance at least equal to the thickness of the projection below the slab soffit.

8.2.6 *Materials*

8.2.6.1 Design properties for concrete shall be selected to be in accordance with Chapter 19.

8.2.6.2 Design properties for GFRP reinforcement shall be selected to be in accordance with Chapter 20.

8.2.6.3 Materials, design, and detailing requirements for embedments in concrete shall be in accordance with 20.6.

8.2.7 *Connections to other members*

8.2.7.1 Beam-column and slab-column joints shall satisfy Chapter 15.

R8.2.7 *Connections to other members*

Safety of a slab system requires consideration of the transmission of load from the slab to the columns by flexure, torsion, and shear.

8.3—Design limits

R8.3—Design limits

8.3.1 *Minimum slab thickness*

8.3.1.1. Slab thickness shall be sufficient to satisfy the calculated deflection limits of 8.3.2.

8.3.1.2 Intentionally left blank.

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8.3.1.3 Intentionally left blank.

8.3.1.4 Intentionally left blank.

8.3.2 *Calculated deflection limits*

8.3.2.1 Immediate and time-dependent deflections shall be calculated in accordance with 24.2 and shall not exceed the limits in 24.2.2.

8.3.2.2 Intentionally left blank.

8.3.3 *Reinforcement strain limit in nonprestressed slabs*—Not applicable

8.3.4 *Stress limits in prestressed slabs*—Out of scope

8.3.5 *Sustained load stress limit*

8.3.5.1 GFRP reinforcement tensile stresses due to the sustained portion of the service load shall satisfy the provisions of 24.6.

8.3.6 *GFRP reinforcement ratio limit*

8.3.6.1 The GFRP reinforcement ratio ρ_f shall not be greater than $6\rho_{fb}$ nor less than $1.4\rho_{fb}$ if the direct design method, the equivalent frame method, or a finite element analysis based on gross section properties is used.

8.4—Required strength

8.4.1 *General*

8.4.1.1 Required strength shall be calculated in accordance with the factored load combinations in Chapter 5.

8.4.1.2 Required strength shall be calculated in accordance with the analysis procedures given in Chapter 6.

8.4.1.3 Intentionally left blank.

8.4.1.4 For a slab system supported by columns or walls, dimensions c_1, c_2, and ℓ_n shall be based on an effective support area. The effective support area is the intersection of the bottom surface of the slab, or drop panel or shear cap if present, with the largest right circular cone, right pyramid, or tapered wedge whose surfaces are located within the column and the capital or bracket and are oriented no greater than 45 degrees to the axis of the column.

8.4.1.5 A column strip is a design strip with a width on each side of a column centerline equal to the lesser of $0.25\ell_2$ and $0.25\ell_1$. A column strip shall include beams within the strip, if present.

COMMENTARY

R8.3.6 *GFRP reinforcement ratio limit*

R8.3.6.1 An upper and lower limit is imposed on the GFRP reinforcement ratio to ensure sufficient deformability to allow for the necessary moment redistribution upon which the direct design and equivalent frame analysis methods are based.

R8.4—Required strength

R8.4.1 *General*

CODE

8.4.1.6 A middle strip is a design strip bounded by two column strips.

8.4.1.7 A panel is bounded by column, beam, or wall centerlines on all sides.

8.4.1.8 For monolithic or fully composite construction supporting two-way slabs, a beam includes that portion of slab, on each side of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness.

8.4.1.9 Combining the results of a gravity load analysis with the results of a lateral load analysis shall be permitted.

8.4.2 *Factored moment*

8.4.2.1 For slabs built integrally with supports, M_u at the support shall be permitted to be calculated at the face of support.

8.4.2.2 *Factored slab moment resisted by the column*

8.4.2.2.1 If gravity load, wind, earthquake, or other effects cause a transfer of moment between the slab and column, a fraction of M_{sc}, the factored slab moment resisted by the column at a joint, shall be transferred by flexure in accordance with 8.4.2.2.2 and 8.4.2.2.3.

COMMENTARY

R8.4.1.7 A panel includes all flexural elements between column centerlines. Thus, the column strip includes the beam, if any.

R8.4.1.8 For monolithic or fully composite construction, the beams include portions of the slab as flanges. Two examples of the rule are provided in Fig. R8.4.1.8.

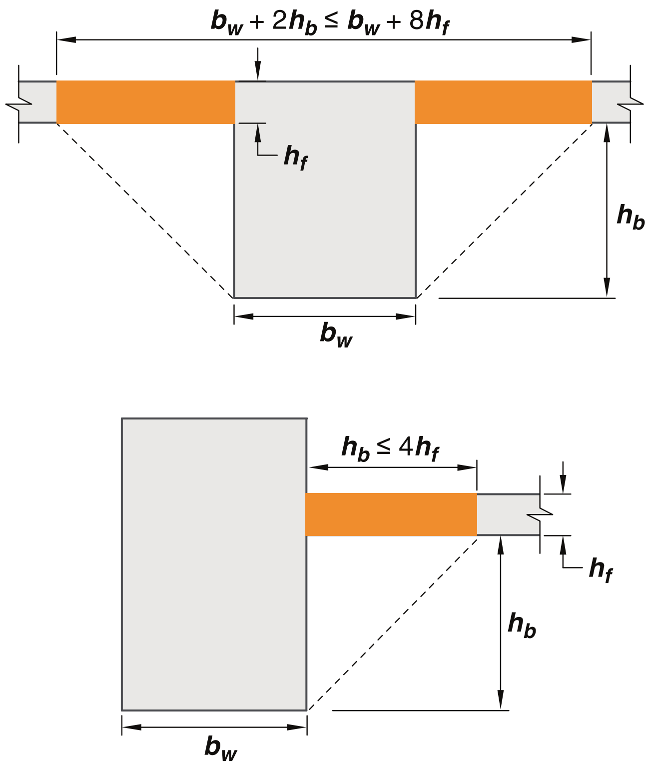

Fig. R8.4.1.8—Examples of the portion of slab to be included with the beam under 8.4.1.8.

R8.4.2 *Factored moment*

R8.4.2.2 *Factored slab moment resisted by the column*

R8.4.2.2.1 This section is concerned primarily with slab systems without beams.

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=**8.4.2.2.2** The fraction of factored slab moment resisted by the column, $\gamma_f M_{sc}$, shall be assumed to be transferred by flexure, where γ_f shall be calculated by:

$$\gamma_f = \frac{1}{1+\left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \qquad (8.4.2.2.2)$$

8.4.2.2.3 The effective slab width b_{slab} for resisting $\gamma_f M_{sc}$ shall be the width of column or capital plus a distance on each side in accordance with Table 8.4.2.2.3.

Table 8.4.2.2.3—Dimensional limits for effective slab width

	Distance on each side of column or capital	
Without drop panel or shear cap	Lesser of	0.5h of slab
		Distance to edge of slab
With drop panel or shear cap	Lesser of	0.5h of drop or cap
		Distance to edge of drop or cap plus 0.5h of slab

8.4.2.2.4 Intentionally left blank.

=**8.4.2.2.5** 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 8.4.2.2.2 and 8.4.2.2.3.

=**8.4.2.2.6** The fraction of M_{sc} not calculated to be resisted by flexure shall be assumed to be resisted by eccentricity of shear in accordance with 8.4.4.2.

8.4.3 *Factored one-way shear*

=**8.4.3.1** For slabs built integrally with supports, V_u at the support shall be permitted to be calculated at the face of support.

8.4.3.2 Sections between the face of support and a critical section located d from the face of support shall be permitted to be designed for V_u at that critical section if (a) through (c) are satisfied:

(a) Support reaction, in direction of applied shear, introduces compression into the end regions of the slab.
(b) Loads are applied at or near the top surface of the slab.
(c) No concentrated load occurs between the face of support and critical section.

8.4.4 *Factored two-way shear*

8.4.4.1 *Critical section*

COMMENTARY

R8.4.2.2.3 Unless measures are taken to resist the torsional and shear stresses, all reinforcement resisting that part of the moment to be transferred to the column by flexure should be placed between lines that are one-half the slab or drop panel thickness, **0.5*h***, on each side of the column or capital (El-Gendy and El-Salakawy 2021).

R8.4.4 *Factored two-way shear*

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8.4.4.1.1 Slabs shall be evaluated for two-way shear in the vicinity of columns, concentrated loads, and reaction areas at critical sections in accordance with 22.6.4.

8.4.4.1.2 Intentionally left blank.

8.4.4.2 *Factored two-way shear stress due to shear and factored slab moment resisted by the column*

8.4.4.2.1 For two-way shear with factored slab moment resisted by the column, factored shear stress $\boldsymbol{v_u}$ shall be calculated at critical sections in accordance with 8.4.4.1. Factored shear stress v_u corresponds to a combination of $\boldsymbol{v_{uv}}$ and the shear stress produced by $\boldsymbol{\gamma_v M_{sc}}$, where $\boldsymbol{\gamma_v}$ is given in 8.4.4.2.2 and $\boldsymbol{M_{sc}}$ is given in 8.4.2.2.1.

8.4.4.2.2 The fraction of $\boldsymbol{M_{sc}}$ transferred by eccentricity of shear, $\boldsymbol{\gamma_v M_{sc}}$, shall be applied at the centroid of the critical section in accordance with 8.4.4.1, where:

$$\gamma_v = 1 - \gamma_f \quad (8.4.4.2.2)$$

8.4.4.2.3 The factored shear stress resulting from $\gamma_v M_{sc}$ shall be assumed to vary linearly about the centroid of the critical section in accordance with 8.4.4.1.

COMMENTARY

The calculated shear stresses in the slab around the column are required to conform to the requirements of 22.6.

R8.4.4.2 *Factored two-way shear stress due to shear and factored slab moment resisted by the column*

R8.4.4.2.2 Hanson and Hanson (1968) found that where moment is transferred between a column and a steel-reinforced concrete slab, 60% of the moment should be considered transferred by flexure across the perimeter of the critical section defined in 22.6.4.1, and 40% by eccentricity of the shear about the centroid of the critical section. For rectangular columns, the portion of the moment transferred by flexure increases as the width of the face of the critical section resisting the moment increases, as given by Eq. (8.4.2.2.2).

Most of the data in Hanson and Hanson (1968) were obtained from tests of steel-reinforced concrete square columns. Limited information is available for round columns; however, these can be approximated as square columns having the same cross-sectional area.

R8.4.4.2.3 The stress distribution is assumed as illustrated in Fig. R8.4.4.2.3 for an interior or exterior column. The perimeter of the critical section, ABCD, is determined in accordance with 22.6.4.1. The factored shear stress $\boldsymbol{v_{uv}}$ and factored slab moment resisted by the column $\boldsymbol{M_{sc}}$ are determined at the centroidal axis c-c of the critical section. The maximum factored shear stress may be calculated from:

$$v_{u,AB} = v_{uv} + \frac{\gamma_v M_{sc} c_{AB}}{J_c}$$

or

$$v_{u,CD} = v_{uv} + \frac{\gamma_v M_{sc} c_{CD}}{J_c}$$

where $\boldsymbol{\gamma_v}$ is given by Eq. (8.4.4.2.2).

For an interior column, $\boldsymbol{J_c}$ may be calculated by:

$\boldsymbol{J_c}$ = property of assumed critical section analogous to polar moment of inertia

$$= \frac{d(c_1+d)^3}{6} + \frac{(c_1+d)d^3}{6} + \frac{d(c_2+d)(c_1+d)^2}{2}$$

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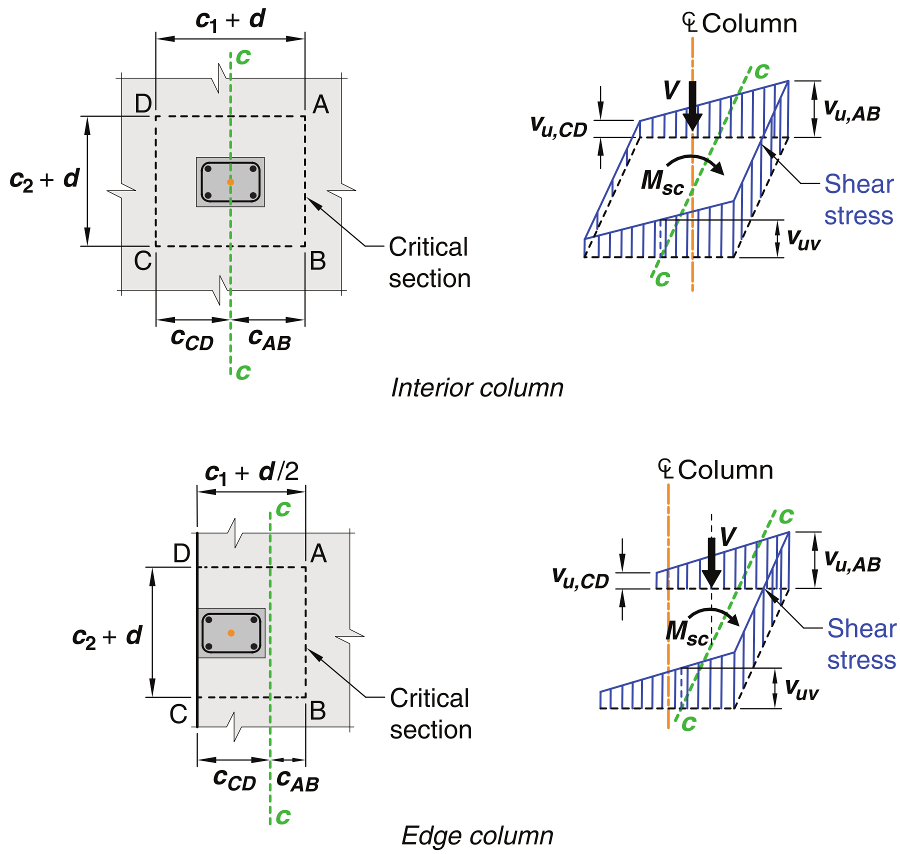

Fig. R8.4.4.2.3—Assumed distribution of shear stress.

Similar equations may be developed for J_c for columns located at the edge or corner of a slab.

The fraction of M_{sc} not transferred by eccentricity of the shear should be transferred by flexure in accordance with 8.4.2.2. A conservative method assigns the fraction transferred by flexure over an effective slab width defined in 8.4.2.2.3. Often, column strip reinforcement is concentrated near the column to accommodate M_{sc}. Available test data on steel-reinforced concrete slabs (Hanson and Hanson 1968) seem to indicate that this practice does not increase shear strength but may be desirable to increase the stiffness of the slab-column junction.

8.5—Design strength

8.5.1 *General*

8.5.1.1 For each applicable factored load combination, design strength shall satisfy $\phi S_n \geq U$, including (a) through (d). Interaction between load effects shall be considered.

(a) $\phi M_n \geq M_u$ at all sections along the span in each direction

(b) $\phi M_n \geq \gamma_f M_{sc}$ within b_{slab} as defined in 8.4.2.2.3

(c) $\phi V_n \geq V_u$ at all sections along the span in each direction for one-way shear

(d) $\phi v_n \geq v_u$ at the critical sections defined in 8.4.4.1 for two-way shear

R8.5—Design strength

R8.5.1 *General*

R8.5.1.1 Refer to R9.5.1.1.

CODE

=**8.5.1.2** ϕ shall be in accordance with 21.2.

8.5.1.3 Intentionally left blank.

8.5.2 *Moment*

=**8.5.2.1** M_n shall be calculated in accordance with 22.3.

=**8.5.2.2** In calculating M_n for slabs with a drop panel, the thickness of the drop panel below the slab shall not be assumed to be greater than one-fourth the distance from the edge of drop panel to the face of column or column capital.

8.5.2.3 Intentionally left blank.

8.5.3 *Shear*

=**8.5.3.1** Design shear strength of slabs in the vicinity of columns, concentrated loads, or reaction areas shall be the more severe of 8.5.3.1.1 and 8.5.3.1.2.

=**8.5.3.1.1** For one-way shear, where each critical section to be investigated extends in a plane across the entire slab width, V_n shall be calculated in accordance with 22.5.

=**8.5.3.1.2** For two-way shear, v_n shall be calculated in accordance with 22.6.

=**8.5.3.2** For composite concrete slabs, horizontal shear strength V_{nh} shall be calculated in accordance with 16.4.

8.5.4 *Openings in slab systems*

=**8.5.4.1** Openings of any size shall be permitted in slab systems if shown by analysis that all strength and serviceability requirements, including the limits on deflections, are satisfied.

8.5.4.2 Intentionally left blank.

8.6—GFRP reinforcement limits

8.6.1 *Minimum GFRP flexural reinforcement*

8.6.1.1 A minimum area of flexural reinforcement, $A_{f,min}$, equal to the greater of the requirement for shrinkage and temperature reinforcement in 24.4.3.2 and $\frac{2.1}{f_{fu}}A_g$ shall be provided near the tension face in the direction of the span under consideration

8.6.1.2 Intentionally left blank.

8.6.2 *Minimum flexural reinforcement in prestressed slabs*—Out of scope

COMMENTARY

R8.5.3 *Shear*

=**R8.5.3.1** Differentiation should be made between a long and narrow slab acting as a beam, and a slab subject to two-way action where failure may occur by punching along a truncated cone or pyramid around a concentrated load or reaction area.

R8.6—GFRP reinforcement limits

R8.6.1 *Minimum GFRP flexural reinforcement*

R8.6.1.1 Refer to R7.6.1.1.

CODE

8.7—GFRP reinforcement detailing

8.7.1 *General*

8.7.1.1 Concrete cover for reinforcement shall be in accordance with 20.6.1.

8.7.1.2 Development lengths of reinforcement shall be in accordance with 25.4.

8.7.1.3 Splice lengths of reinforcement shall be in accordance with 25.5.

8.7.1.4 Intentionally left blank.

8.7.2 *GFRP flexural reinforcement spacing*

8.7.2.1 Minimum spacing s shall be in accordance with 25.2.

8.7.2.2 For solid slabs, maximum spacing s of longitudinal reinforcement shall be in accordance with 24.3.2.

8.7.2.3 Intentionally left blank.

8.7.2.4 Intentionally left blank.

8.7.3 *Corner restraint in slabs*

8.7.3.1 At exterior corners of slabs supported by edge walls or where one or more edge beams have a value of α_f greater than 1.0, reinforcement at top and bottom of slab shall be designed to resist M_u per unit width due to corner effects equal to the maximum positive M_u per unit width in the slab panel.

8.7.3.1.1 Factored moment due to corner effects, M_u, 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.

8.7.3.1.2 Reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth the longer span.

8.7.3.1.3 Reinforcement shall be placed parallel to the diagonal in the top of the slab and perpendicular to the diagonal in the bottom of the slab. Alternatively, reinforcement shall be placed in two layers parallel to the sides of the slab in both the top and bottom of the slab.

COMMENTARY

R8.7—GFRP reinforcement detailing

R8.7.3 *Corner restraint in slabs*

R8.7.3.1 Unrestrained corners of two-way slabs tend to lift when loaded. If this lifting tendency is restrained by edge walls or beams, bending moments result in the slab. This section requires reinforcement to resist these moments and control cracking. Reinforcement provided for flexure in the primary directions may be used to satisfy this requirement. Refer to Fig. R8.7.3.1.

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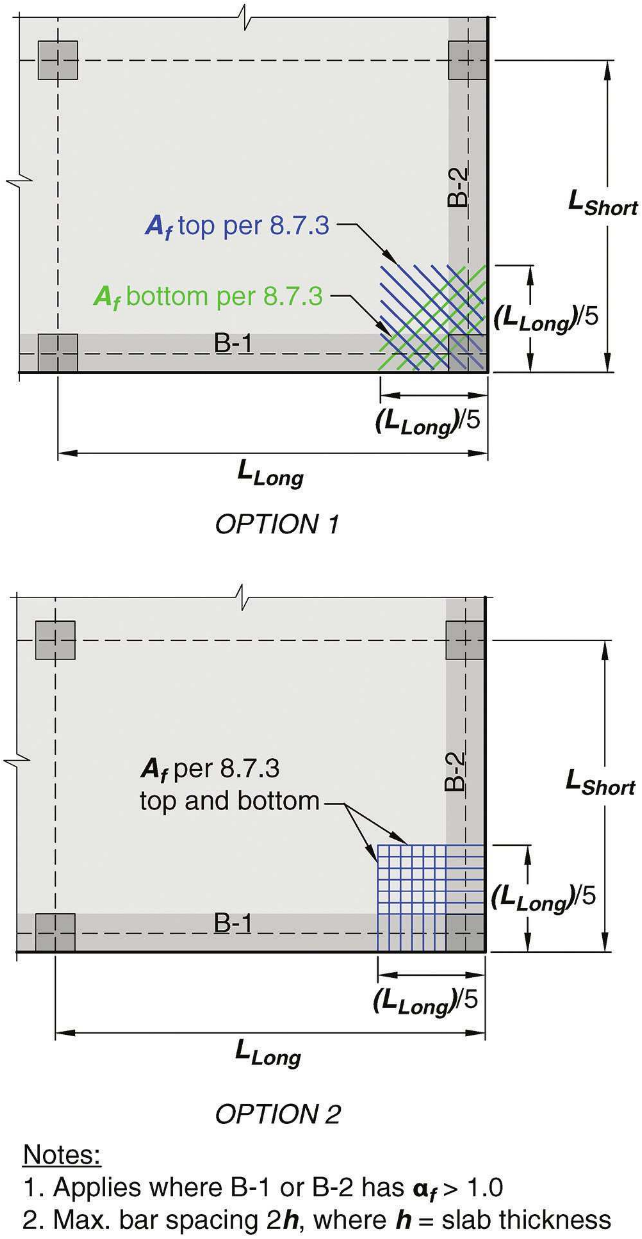

Fig. R8.7.3.1—Slab corner reinforcement.

8.7.4 *GFRP flexural reinforcement*

8.7.4.1 *Termination of GFRP reinforcement*

8.7.4.1.1 Where a slab is supported on spandrel beams, columns, or walls, anchorage of reinforcement perpendicular to a discontinuous edge shall satisfy (a) and (b):

(a) Positive moment reinforcement shall extend to the edge of slab and have embedment, straight or hooked, at least 150 mm into spandrel beams, columns, or walls

R8.7.4 *GFRP flexural reinforcement*

R8.7.4.1 *Termination of GFRP reinforcement*

R8.7.4.1.1 and R8.7.4.1.2 Bending moments in slabs at spandrel beams may vary significantly. If spandrel beams are built solidly into walls, the slab approaches complete fixity. Without an integral wall, the slab could approach being simply supported, depending on the torsional rigidity of the spandrel beam or slab edge. These requirements provide for unknown conditions that might normally occur in a structure.

CODE

(b) Negative moment reinforcement shall be hooked, or otherwise anchored into spandrel beams, columns, or walls, and shall be developed at the face of support

8.7.4.1.2 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.

8.7.4.1.3 For slabs without beams, reinforcement extensions shall be in accordance with (a) and (b):

(a) Reinforcement lengths shall be at least in accordance with Fig. 8.7.4.1.3, and if slabs act as primary members resisting lateral loads, reinforcement lengths shall be at least those required by analysis.

(b) If adjacent spans are unequal, extensions of negative moment reinforcement beyond the face of support in accordance with Fig. 8.7.4.1.3 shall be based on the longer span.

COMMENTARY

R8.7.4.1.3 The minimum lengths and extensions of reinforcement expressed as a fraction of the clear span in Fig. 8.7.4.1.3 were developed for steel-reinforced concrete slabs of ordinary proportions supporting gravity loads. These minimum lengths and extensions may not be sufficient to intercept potential punching shear cracks in thick two-way slabs such as transfer slabs, podium slabs, and mat foundations. Therefore, the Code requires extensions for at least half of the column strip top bars to be at least $5d$. For slabs with drop panels, d is the effective depth within the drop panel. In these thick two-way slabs, continuous reinforcement in each direction near both faces is desirable to improve structural integrity, control cracking, and reduce creep deflections. As illustrated in Fig. R8.7.4.1.3, punching

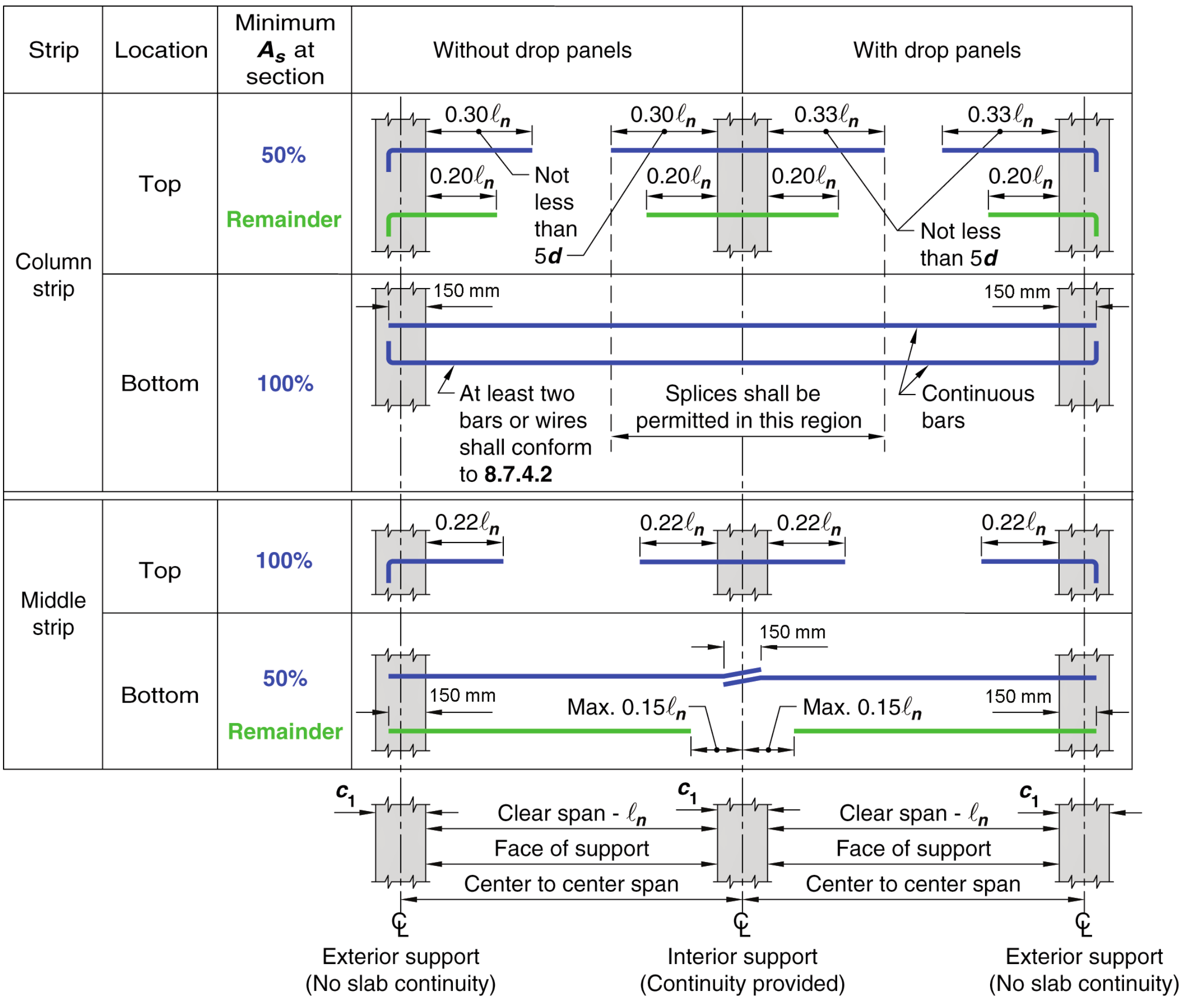

Fig. 8.7.4.1.3—Minimum extensions for reinforcement in two-way slabs without beams.

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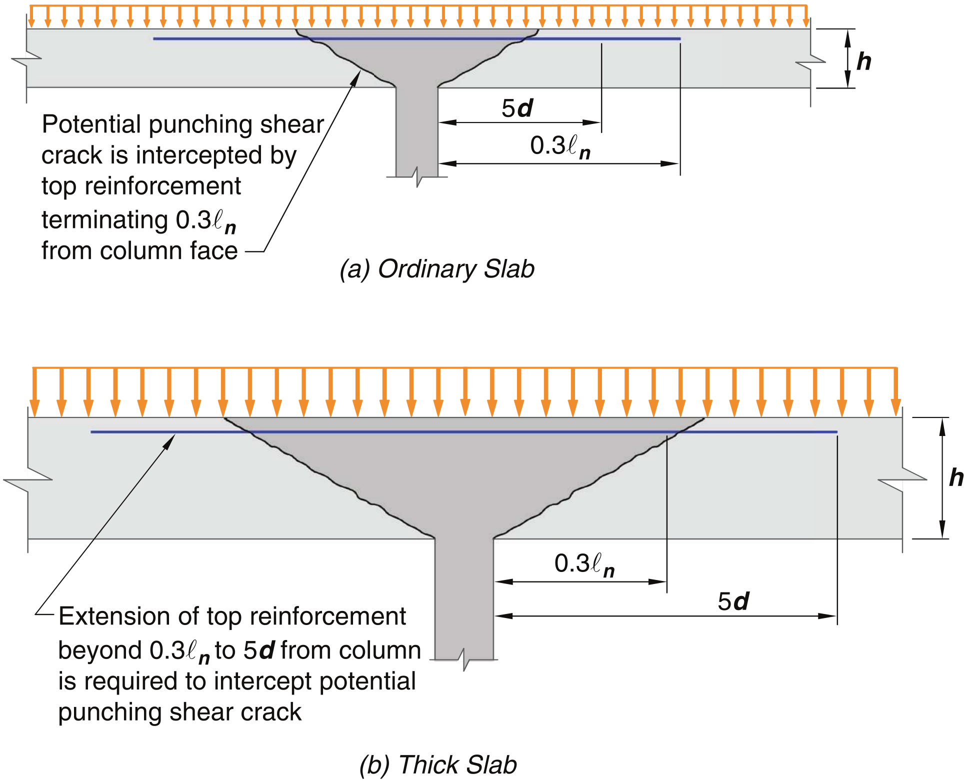

Fig. R8.7.4.1.3—Punching shear in ordinary and thick slabs.

shear cracks, which can develop at angles as low as approximately 20 degrees, may not be intercepted by the tension reinforcement in thick slabs if this reinforcement does not extend to at least **5*d*** beyond the face of the support. The **5*d*** bar extension requirement governs where ℓ_n/h is less than approximately 15. For moments resulting from combined lateral and gravity loadings, these minimum lengths and extensions of bars may not be sufficient.

8.7.4.2 *Structural integrity*

8.7.4.2.1 All bottom bars within the column strip, in each direction, shall be continuous or spliced with full mechanical, or Class B tension splices. Splices shall be located in accordance with Fig. 8.7.4.1.3.

8.7.4.2.2 At least two of the column strip bottom bars in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at exterior supports.

8.7.4.2.3 Intentionally left blank.

8.7.5 *Flexural reinforcement in prestressed slabs*—Out of scope

8.7.6 *Shear reinforcement – stirrups*—Out of scope

8.7.7 *Shear reinforcement – headed studs*—Out of scope

R8.7.4.2 *Structural integrity*

R8.7.4.2.1 and R8.7.4.2.2 The continuous column strip bottom reinforcement provides the slab some residual ability to span to the adjacent supports should a single support be damaged. The two continuous column strip bottom bars through the column may be termed "integrity reinforcement," and are provided to give the slab some residual strength following a single punching shear failure at a single support.

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8.8—Nonprestressed two-way joist systems—Out of scope

8.9—Lift-slab construction—Out of scope

COMMENTARY

R8.8—Nonprestressed two-way joist systems—Out of scope

ACI 318 specifications for the design of nonprestressed two-way joist systems are empirical limits based on past performance of steel-reinforced two-way concrete joist systems. The design of nonprestressed two-way joists systems is not covered in this Code due to a lack of documented performance of GFRP-reinforced two-way concrete joist systems.

R8.9—Lift-slab construction—Out of scope

Guidance on reinforcement through the lifting collar of GFRP-reinforced concrete slabs constructed with lift-slab construction is not covered in this Code due to a lack of published research on and use of lift-slab construction of GFRP-reinforced concrete slabs.

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COMMENTARY

CHAPTER 9—BEAMS

9.1—Scope

9.1.1 This chapter shall apply to the design of nonprestressed beams, including:

(a) Composite beams of concrete elements constructed in separate placements but connected so that all elements resist loads as a unit

(b) One-way joist systems in accordance with 9.8

9.2—General

9.2.1 *Materials*

9.2.1.1 Design properties for concrete shall be selected to be in accordance with Chapter 19.

9.2.1.2 Design properties for GFRP reinforcement shall be selected to be in accordance with Chapter 20.

9.2.1.3 Materials, design, and detailing requirements for embedments in concrete shall be in accordance with 20.6.

9.2.2 *Connection to other members*

9.2.2.1 For cast-in-place construction, beam-column and slab-column joints shall satisfy Chapter 15.

9.2.2.2 Intentionally left blank.

9.2.3 *Stability*

9.2.3.1 If a beam is not continuously laterally braced, (a) and (b) shall be satisfied:

(a) Spacing of lateral bracing shall not exceed 50 times the least width of compression flange or face.

(b) Spacing of lateral bracing shall take into account effects of eccentric loads.

9.2.3.2 Intentionally left blank.

9.2.4 *T-beam construction*

9.2.4.1 In T-beam construction, flange and web concrete shall be placed monolithically or made composite in accordance with 16.4.

9.2.4.2 Effective flange width shall be in accordance with 6.3.2.

9.2.4.3 For T-beam flanges where the primary flexural slab reinforcement is parallel to the longitudinal axis of the

CHAPTER R9—BEAMS

R9.1—Scope

R9.1.1 Composite GFRP-structural profile concrete beams are not covered in this chapter.

R9.2—General

R9.2.3 *Stability*

R9.2.3.1 Tests (Hansell and Winter 1959; Sant and Bletzacker 1961) have shown that laterally unbraced steel-reinforced concrete beams, even when very deep and narrow, will not fail prematurely by lateral buckling, provided the beams are loaded without lateral eccentricity that causes torsion.

Laterally unbraced beams are frequently loaded eccentrically or with slight inclination. Stresses and deformations by such loading become detrimental for narrow, deep beams with long unsupported lengths. Lateral supports spaced closer than **50*b*** may be required for such loading conditions.

R9.2.4 *T-beam construction*

R9.2.4.1 For monolithic or fully composite construction, the beam includes a portion of the slab as flanges.

R9.2.4.3 Refer to R7.5.2.3.

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beam, reinforcement in the flange perpendicular to the longitudinal axis of the beam shall be in accordance with 7.5.2.3.

=**9.2.4.4** For torsional design according to 22.7, the overhanging flange width used to calculate A_{cp}, A_g, and p_{cp} shall be in accordance with (a) and (b):

(a) The overhanging flange width shall include that portion of slab on each side of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness.

(b) The overhanging flanges shall be neglected in cases where the parameter A_{cp}^2/p_{cp} for solid sections or A_g^2/p_{cp} for hollow sections calculated for a beam with flanges is less than that calculated for the same beam ignoring the flanges.

COMMENTARY

=**R9.2.4.4** Two examples of the section to be considered in torsional design are provided in Fig. R9.2.4.4.

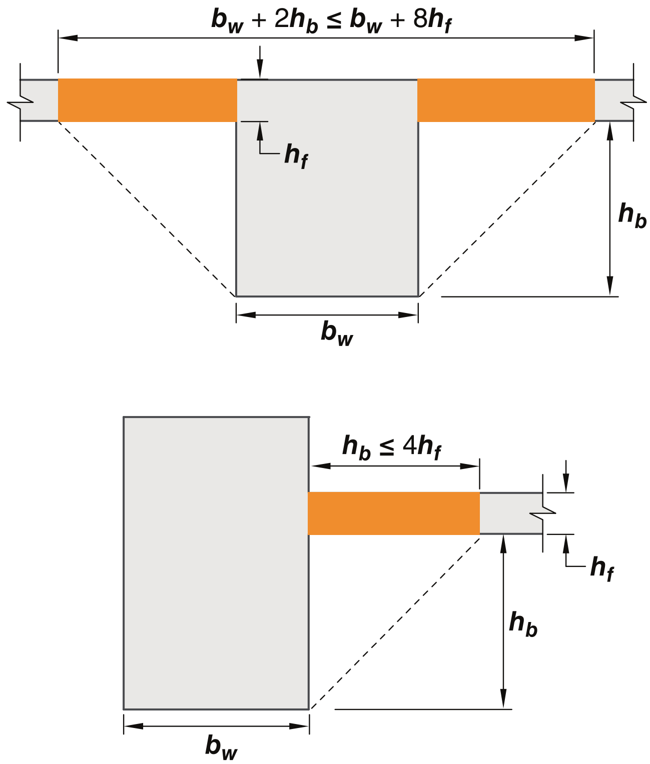

Fig. R9.2.4.4—Examples of the portion of slab to be included with the beam for torsional design.

9.3—Design limits

9.3.1 *Minimum beam depth*

9.3.1.1 The beam depth shall be sufficient to satisfy the calculated deflection limits of 9.3.2.

9.3.1.2 Intentionally left blank.

9.3.2 *Calculated deflection limits*

9.3.2.1 Immediate and time-dependent deflections shall be calculated in accordance with 24.2 and shall not exceed the limits in 24.2.2.

9.3.2.2 Intentionally left blank.

9.3.3 *Reinforcement strain limit in nonprestressed beams*—Not applicable

9.3.4 *Stress limits in prestressed beams*—Out of scope

9.3.5 *Sustained load stress limit*

CODE

9.3.5.1 GFRP reinforcement stresses due to the sustained portion of the service load shall satisfy the provisions of 24.6.

9.4—Required strength

9.4.1 *General*

=9.4.1.1 Required strength shall be calculated in accordance with the factored load combinations in Chapter 5.

=9.4.1.2 Required strength shall be calculated in accordance with the analysis procedures in Chapter 6.

9.4.1.3 Intentionally left blank.

9.4.2 *Factored moment*

=9.4.2.1 For beams built integrally with supports, M_u at the support shall be permitted to be calculated at the face of support.

9.4.3 *Factored shear*

=9.4.3.1 For beams built integrally with supports, V_u at the support shall be permitted to be calculated at the face of support.

9.4.3.2 Sections between the face of support and a critical section located d from the face of support shall be permitted to be designed for V_u at that critical section if (a) through (c) are satisfied:

(a) Support reaction, in direction of applied shear, introduces compression into the end region of the beam

(b) Loads are applied at or near the top surface of the beam

(c) No concentrated load occurs between the face of support and critical section

COMMENTARY

R9.4—Required strength

R9.4.3 *Factored shear*

R9.4.3.2 The closest inclined crack to the support of the beam in Fig. R9.4.3.2a will extend upward from the face of the support reaching the compression zone approximately d from the face of the support. If loads are applied to the top of the beam, the stirrups across this crack need only resist the shear force due to loads acting beyond d (right free body in Fig. R9.4.3.2a). The loads applied to the beam between the face of the support and the point d away from the face are transferred directly to the support by compression in the web above the crack. Accordingly, the Code permits design for a maximum factored shear V_u at a distance d from the support.

In Fig. R9.4.3.2b, loads are shown acting near the bottom of a beam. In this case, the critical section is taken at the face of the support. Loads acting near the support should be transferred across the inclined crack extending upward from the support face. The shear force acting on the critical section should include all loads applied below the potential inclined crack.

Typical support conditions where the shear force at a distance d from the support may be used include:

(a) Beams supported by bearing at the bottom of the beam, such as shown in Fig. R9.4.3.2(c)

(b) Beams framing monolithically into a column, as illustrated in Fig. R9.4.3.2(d)

Typical support conditions where the critical section is taken at the face of support include:

(a) Beams framing into a supporting member in tension, such as shown in Fig. R9.4.3.2(e). Shear within the connection should also be investigated and special GFRP corner reinforcement should be provided.

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COMMENTARY

(b) Beams for which loads are not applied at or near the top, as previously discussed and as shown in Fig. R9.4.3.2b.

(c) Beams loaded such that the shear at sections between the support and a distance ***d*** from the support differs radically from the shear at distance ***d***. This commonly occurs in brackets and in beams where a concentrated load is located close to the support, as shown in Fig. R9.4.3.2(f).

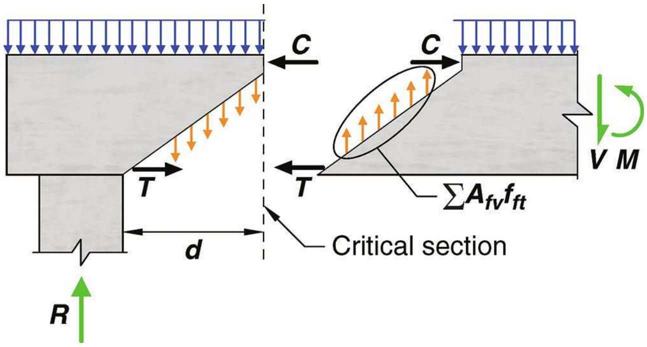

Fig. R9.4.3.2a—Free body diagrams of the end of a beam.

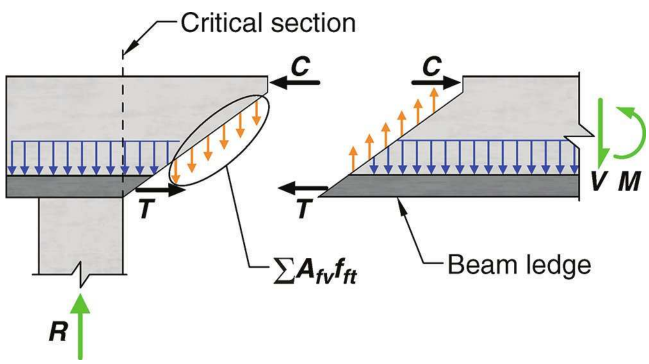

Fig. R9.4.3.2b—Location of critical section for shear in a beam loaded near bottom.

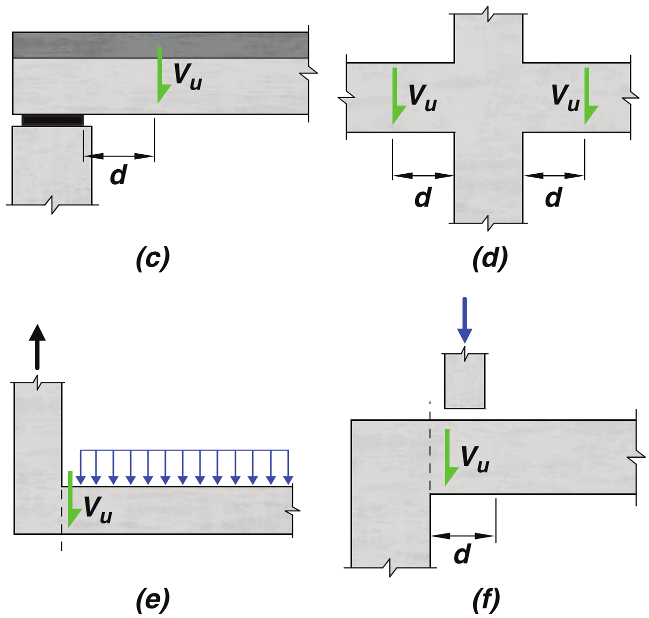

Fig. R9.4.3.2(c), (d), (e), (f)—Typical support conditions for locating factored shear force V_u.

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9.4.4 *Factored torsion*

9.4.4.1 Unless determined by a more detailed analysis, it shall be permitted to take the torsional loading from a slab as uniformly distributed along the beam.

9.4.4.2 For beams built integrally with supports, T_u at the support shall be permitted to be calculated at the face of support.

9.4.4.3 Sections between the face of support and a critical section located d from the face of support shall be permitted to be designed for T_u at that critical section unless a concentrated torsional moment occurs within this distance. In that case, the critical section shall be taken at the face of the support.

9.4.4.4 It shall be permitted to reduce T_u in accordance with 22.7.3.

9.5—Design strength

9.5.1 *General*

9.5.1.1 For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$ including (a) through (d). Interaction between load effects shall be considered.

(a) $\phi M_n \geq M_u$
(b) $\phi V_n \geq V_u$
(c) $\phi T_n \geq T_u$
(d) $\phi P_n \geq P_u$

9.5.1.2 ϕ shall be determined in accordance with 21.2.

9.5.2 *Moment*

9.5.2.1 If $P_u < 0.10 f_c' A_g$, M_n shall be calculated in accordance with 22.3.

9.5.2.2 If $P_u \geq 0.10 f_c' A_g$, M_n shall be calculated in accordance with 22.4.

9.5.2.3 Intentionally left blank.

9.5.3 *Shear*

9.5.3.1 V_n shall be calculated in accordance with 22.5.

9.5.3.2 For composite concrete beams, horizontal shear strength V_{nh} shall be calculated in accordance with 16.4.

COMMENTARY

R9.4.4 *Factored torsion*

R9.4.4.3 It is not uncommon for a beam to frame into one side of a girder near the support of the girder. In such a case, a concentrated shear and torque are applied to the girder.

R9.5—Design strength

R9.5.1 *General*

R9.5.1.1 The design conditions 9.5.1.1(a) through (d) list the typical forces and moments that need to be considered. However, the general condition $\phi S_n \geq U$ indicates that all forces and moments that are relevant for a given structure need to be considered.

R9.5.2 *Moment*

R9.5.2.2 Beams resisting significant axial forces require consideration of the combined effects of axial forces and moments. These beams are not required to satisfy the provisions of Chapter 10, but are required to satisfy the additional requirements for ties or spirals defined in Table 22.4.2.1. For slender beams with significant axial loads, consideration should be given to slenderness effects as required for columns in 6.2.5.

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9.5.4 *Torsion*

9.5.4.1 If $T_u < \phi T_{th}$, where T_{th} is given in 22.7, it shall be permitted to neglect torsional effects. The minimum reinforcement requirements of 9.6.4 and the detailing requirements of 9.7.5 and 9.7.6.3 need not be satisfied.

9.5.4.2 T_n shall be calculated in accordance with 22.7.

9.5.4.3 Longitudinal and transverse reinforcement required for torsion shall be added to that required for the V_u, M_u, and P_u that act in combination with the torsion.

9.5.4.4 Intentionally left blank.

9.5.4.5 It shall be permitted to reduce the area of longitudinal torsional reinforcement in the flexural compression zone by an amount equal to $M_u/(0.9df_f)$ where M_u occurs simultaneously with T_u at that section except that the longitudinal reinforcement area shall not be less than the minimum required in 9.6.4.

9.5.4.6 Intentionally left blank.

9.5.4.7 Intentionally left blank.

COMMENTARY

R9.5.4 *Torsion*

R9.5.4.3 The requirements for torsional reinforcement and shear reinforcement are added and stirrups are provided to supply at least the total amount required. Because the reinforcement area A_{fv} for shear is defined in terms of all the legs of a given stirrup while the reinforcement area A_{ft} for torsion is defined in terms of one leg only, the addition of transverse reinforcement area is calculated as follows:

$$\text{total}\left(\frac{A_{fv+ft}}{s}\right) = \frac{A_{fv}}{s} + 2\frac{A_{ft}}{s} \qquad \text{(R9.5.4.3)}$$

If a stirrup group has more than two legs for shear, only the legs adjacent to the sides of the beam are included in this summation because the inner legs would be ineffective for resisting torsion.

The longitudinal reinforcement required for torsion is added at each section to the longitudinal reinforcement required for bending moment that acts concurrently with the torsion. The longitudinal reinforcement is then chosen for this sum, but should not be less than the amount required for the maximum bending moment at that section if this exceeds the moment acting concurrently with the torsion. If the maximum bending moment occurs at one section, such as midspan, while the maximum torsional moment occurs at another, such as the face of the support, the total longitudinal reinforcement required may be less than that obtained by adding the maximum flexural reinforcement, plus the maximum torsional reinforcement. In such a case, the required longitudinal reinforcement is evaluated at several locations.

R9.5.4.5 The longitudinal tension due to torsion is offset in part by the compression in the flexural compression zone, allowing a reduction in the longitudinal torsional reinforcement required in the compression zone. In lieu of detailed calculations to determine f_f, f_f can be conservatively replaced by f_{fr}.

9.6—GFRP reinforcement limits

9.6.1 *Minimum GFRP flexural reinforcement*

R9.6—GFRP reinforcement limits

R9.6.1 *Minimum GFRP flexural reinforcement*

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9.6.1.1 A minimum area of flexural reinforcement, $A_{f,min}$, shall be provided at every section where tension reinforcement is required by analysis.

9.6.1.2 $A_{f,min}$ shall be at least the greater of (a) and (b), except as provided in 9.6.1.3. For a statically determinate beam with a flange in tension, the value of b_w shall be the lesser of b_f and $2b_w$.

(a) $\dfrac{0.41\sqrt{f_c'}}{f_{fu}}b_w d$

(b) $\dfrac{2.3}{f_{fu}}b_w d$

9.6.1.3 If A_f provided at every section is at least one-third greater than A_f required by analysis, 9.6.1.1 and 9.6.1.2 need not be satisfied.

9.6.2 *Minimum flexural reinforcement in prestressed beams*—Out of scope

9.6.3 *Minimum GFRP shear reinforcement*

9.6.3.1 A minimum area of shear reinforcement, $A_{fv,min}$, shall be provided in all regions where $V_u \geq \phi 2.5 k_{cr}\sqrt{f_c'}\,b_w d$ except for the cases in Table 9.6.3.1. For these cases, at least $A_{fv,min}$ shall be provided where $V_u > \phi V_c$.

COMMENTARY

R9.6.1.1 This provision is intended to result in flexural strength exceeding the cracking strength by a margin. The objective is to produce a beam that will be able to sustain loading after the onset of flexural cracking, with visible cracking and deflection, thereby warning of possible overload. Beams with less reinforcement may sustain sudden failure with the onset of flexural cracking.

In practice, this provision only controls reinforcement design for beams which, for architectural or other reasons, are larger in cross section than required for strength. With a small amount of tension reinforcement required for strength, the calculated moment strength of a reinforced concrete section using cracked section analysis becomes less than that of the corresponding unreinforced concrete section calculated from its modulus of rupture. Failure in such a case could occur at first cracking and without warning. To prevent such a failure, a minimum amount of tension reinforcement is required in both positive and negative moment regions.

R9.6.1.2 The minimum GFRP reinforcement equations are intended to result in the calculated moment strength of sections reinforced with $A_{f,min}$ to exceed the cracking moment of the corresponding unreinforced concrete section by the same margin as is required for sections reinforced with $A_{s,min}$ in ACI 318, after accounting for the difference in flexural strength reduction factors for steel-reinforced and GFRP-reinforced concrete. If a GFRP-reinforced concrete section is not tension controlled, the minimum amount of GFRP reinforcement to prevent failure upon concrete cracking is automatically achieved.

If the flange of a GFRP-reinforced section is in tension, the amount of GFRP tension reinforcement needed to make the strength of the reinforced section equal that of the unreinforced section is greater than that for a rectangular section or that of a flanged section with the flange in compression. A greater amount of minimum GFRP tension reinforcement is particularly necessary in cantilevers.

R9.6.3 *Minimum GFRP shear reinforcement*

R9.6.3.1 Shear reinforcement restrains the growth of inclined cracking so that the deformability of the beam is improved and a warning of failure is provided. In an unreinforced web, the formation of inclined cracking might lead directly to failure without warning. Such reinforcement is of great value if a beam is subjected to an unexpected tensile force or an overload.

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Table 9.6.3.1—Cases where $A_{fv,min}$ is not required if $V_u \leq \phi V_c$

Beam type	Conditions
Shallow depth	$h \leq 250$ mm
Integral with slab	$h \leq$ greater of $2.5t_f$ or $0.5b_w$ and $h \leq 450$ mm
One-way joist system	In accordance with 9.8

9.6.3.2 Intentionally left blank.

9.6.3.3 If shown by testing that the required M_n and V_n can be developed, 9.6.3.1 need not be satisfied. Such tests shall simulate effects of differential settlement, creep, shrinkage, and temperature change, based on a realistic assessment of these effects occurring in service.

9.6.3.4 If shear reinforcement is required and torsional effects can be neglected according to 9.5.4.1, minimum transverse reinforcement $A_{fv,min}$ shall be the greater of (a) and (b).

(a) $0.0062\sqrt{f_c'}\dfrac{b_w}{f_{ft}}s$

(b) $0.35\dfrac{b_w}{f_{ft}}s$

9.6.4 *Minimum GFRP torsional reinforcement*

9.6.4.1 A minimum area of torsional reinforcement shall be provided in all regions where $T_u \geq \phi T_{th}$ in accordance with 22.7.

9.6.4.2 If torsional reinforcement is required, minimum transverse reinforcement $(A_{fv} + 2A_{ft})_{min}/s$ shall be the greater of (a) and (b):

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Joists are excluded from the minimum shear reinforcement requirement as indicated because there is a possibility of load sharing between weak and strong areas.

For repeated loading of beams, the possibility of inclined diagonal tension cracks forming at stresses appreciably smaller than under static loading should be taken into account in design. In these instances, use of at least the minimum shear reinforcement expressed by 9.6.3.4 is recommended even though tests or calculations based on static loads show that shear reinforcement is not required.

R9.6.3.3 When a beam is tested to demonstrate that its shear and flexural strengths are adequate, the actual beam dimensions and material strengths are known. Therefore, the test strengths are considered the nominal strengths V_n and M_n. Considering these strengths as nominal values ensures that if the actual material strengths in the field were less than specified, or the member dimensions were in error such as to result in a reduced member strength, a satisfactory margin of safety will be retained due to the strength reduction factor ϕ.

R9.6.3.4 Tests of steel-reinforced concrete beams (Roller and Russell 1990) have indicated the need to increase the minimum area of shear reinforcement as the concrete strength increases to prevent sudden shear failures when inclined cracking occurs. Therefore, 9.6.3.4(a) provides for a gradual increase in the minimum area of transverse reinforcement with increasing concrete strength. Expression 9.6.3.4(b) provides for a minimum area of transverse reinforcement independent of concrete strength and governs for concrete strengths less than 30 MPa.

The expressions for the minimum amount of shear reinforcement, which were developed for steel-reinforced concrete, are more conservative if used with GFRP-reinforced concrete members because the ratio of the shear strength provided using $A_{fv,min}$ to V_c is greater for GFRP-reinforced concrete than for steel-reinforced concrete. The ratio will decrease as the stiffness of the GFRP longitudinal reinforcement increases or as the strength of the concrete increases.

R9.6.4 *Minimum GFRP torsional reinforcement*

R9.6.4.1 Mohamed and Benmokrane (2016) investigated the performance of concrete beams reinforced with longitudinal GFRP bars and varying amounts of GFRP stirrups under torsion and concluded that satisfactory performance can be attained if the GFRP torsion reinforcement adheres to minimum area requirements.

R9.6.4.2 The differences in the definitions of A_{fv} and A_{ft} should be noted: A_{fv} is the area of two legs of a closed stirrup, whereas A_{ft} is the area of only one leg of a closed stirrup. If a stirrup group has more than two legs, only the legs adja-

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(a) $0.062\sqrt{f_c'}\dfrac{b_w}{f_{ft}}$

(b) $0.35\dfrac{b_w}{f_{ft}}$

9.6.4.3 If torsional reinforcement is required, minimum area of longitudinal reinforcement $A_{ft,min}$ shall be the lesser of (a) and (b):

(a) $\dfrac{0.42\sqrt{f_c'}}{f_{fu}}A_{cp} - \left(\dfrac{A_{fv}}{s}\right)p_h\dfrac{f_{ft}}{f_{fu}}$

(b) $\dfrac{0.42\sqrt{f_c'}}{f_{fu}}A_{cp} - \left(\dfrac{0.175b_w}{f_{fv}}\right)p_h\dfrac{f_{ft}}{f_{fu}}$

9.7—GFRP reinforcement detailing

9.7.1 *General*

9.7.1.1 Concrete cover for reinforcement shall be in accordance with 20.5.1.

9.7.1.2 Development lengths of reinforcement shall be in accordance with 25.4.

9.7.1.3 Splices of reinforcement shall be in accordance with 25.5.

9.7.1.4 Intentionally left blank.

9.7.2 *GFRP reinforcement spacing*

9.7.2.1 Minimum spacing s shall be in accordance with 25.2.

9.7.2.2 Spacing of longitudinal reinforcement closest to the tension face shall not exceed s given in 24.3.

9.7.2.3 For beams with h exceeding 450 mm longitudinal skin reinforcement shall be uniformly distributed on both side faces of the beam for a distance $h/2$ from the tension face. Spacing of skin reinforcement shall not exceed s given in 24.3.2, where c_c is the clear cover from the skin reinforcement to the side face. It shall be permitted to include skin reinforcement in strength calculations if a strain compatibility analysis is made.

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cent to the sides of the beam are considered, as discussed in R9.5.4.3.

Tests (Roller and Russell 1990) of high-strength steel-reinforced concrete beams have indicated the need to increase the minimum area of shear reinforcement to prevent shear failures when inclined cracking occurs. Although there are a limited number of tests of high-strength concrete beams in torsion, the equation for the minimum area of transverse closed stirrups has been made consistent with calculations required for minimum shear reinforcement.

R9.6.4.3 Under combined torsion and shear, the torsional cracking moment decreases with applied shear, which leads to a reduction in torsional reinforcement required to prevent brittle failure immediately after cracking. When subjected to pure torsion, steel-reinforced concrete beam specimens with less than 1% torsional reinforcement by volume have failed at first torsional cracking (MacGregor and Ghoneim 1995).

R9.7—GFRP reinforcement detailing

R9.7.2 *GFRP reinforcement spacing*

R9.7.2.3 For beams with heights greater than 450 mm, some reinforcement should be placed near the vertical faces of the tension zone to control cracking in the web, as shown in Fig. R9.7.2.3. Without such auxiliary reinforcement, the width of the cracks in the web may exceed the crack widths at the level of the flexural tension reinforcement. Analysis of GFRP-reinforced concrete sections, using the physical model (Frosch 2002) that is the basis of the ACI 318 provisions for skin reinforcement, shows that GFRP-reinforced concrete beams with heights greater than 450 mm may require skin

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reinforcement to control mid-depth crack widths. This difference can be attributed in part to the greater depth from the tension face to the neutral axis in beams with GFRP reinforcement as well as larger strains at service loads for beams where the failure mode is by GFRP reinforcement rupture. Tension cracks can merge together and cause larger crack widths approximately mid-height between the neutral axis and the beam tension face, consistent with model predictions.

The size of the skin reinforcement is not specified; research has indicated that the spacing rather than bar size is of primary importance (Frosch 2002). Bar sizes No. M10 to No. M16 are typically provided in steel-reinforced concrete beams.

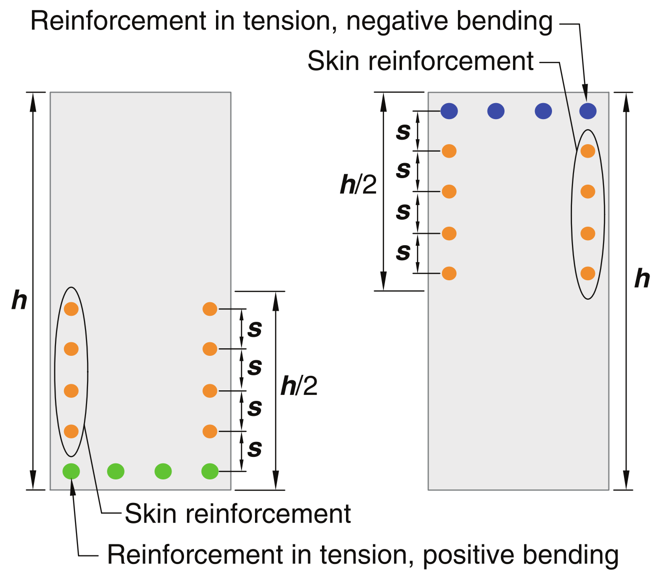

Fig. R9.7.2.3—Skin reinforcement for beams and joists with **h** *>18 in.*

9.7.3 *GFRP flexural reinforcement*

R9.7.3 *GFRP flexural reinforcement*

=**9.7.3.1** Calculated tensile or compressive force in reinforcement at each section of the beam shall be developed on each side of that section.

9.7.3.2 Critical locations for development of reinforcement are points of maximum stress and points along the span where terminated tension reinforcement is no longer required to resist flexure.

R9.7.3.2 Critical sections for a typical continuous beam are indicated with a "c" for points of maximum stress or an "x" for points where terminated tension reinforcement is no longer required to resist flexure (Fig. R9.7.3.2). For uniform loading, the positive reinforcement extending into the support is more likely governed by the requirements of 9.7.3.8.1 or 9.7.3.8.3 than by development length measured from a point of maximum moment or bar cutoff.

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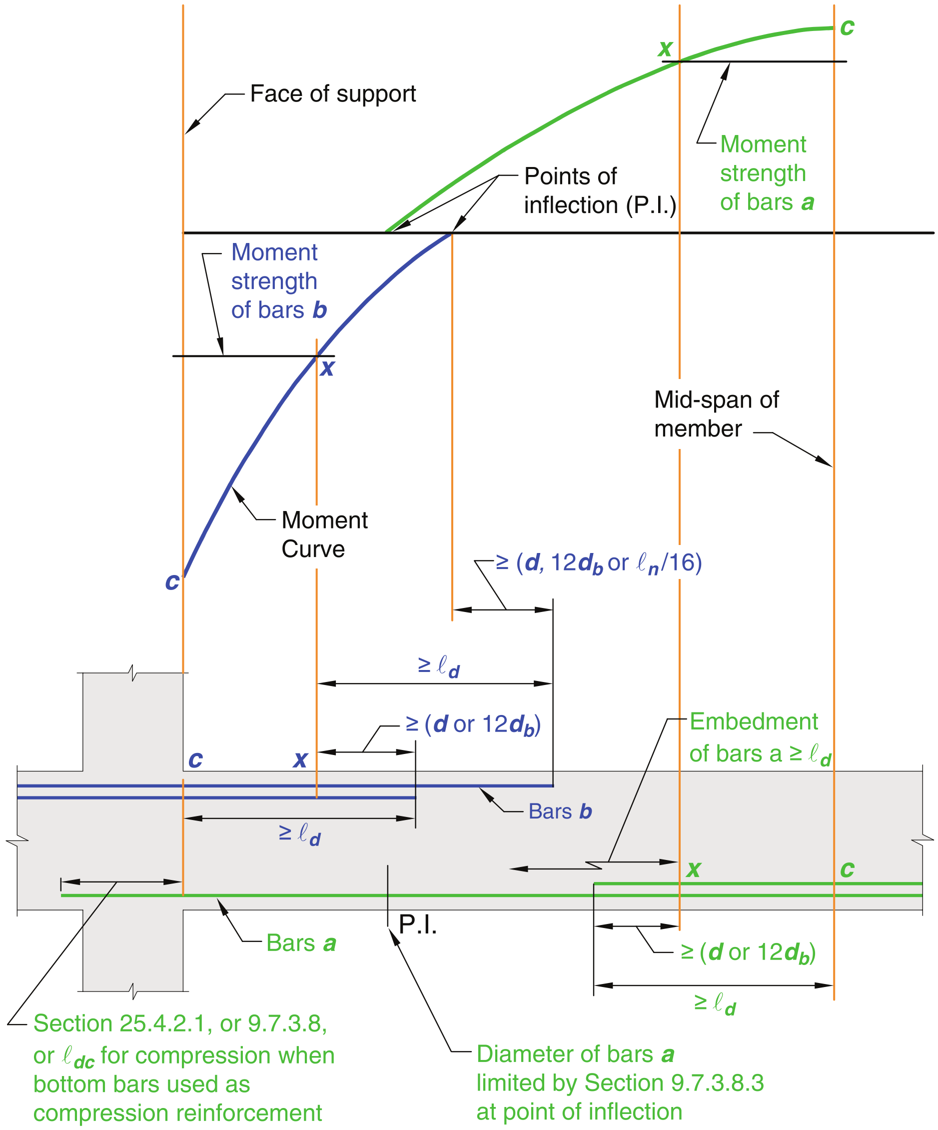

Fig. R9.7.3.2—Development of flexural reinforcement in a typical continuous beam.

9.7.3.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure and provide stiffness to satisfy deflection requirements for a distance equal to the greater of d and $12d_b$, except at supports of simply supported spans and at free ends of cantilevers.

R9.7.3.3 The moment diagrams customarily used in design are approximate; some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, lateral loads, or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location of the calculated tensile stress approximately a distance d toward a point of zero moment. If stirrups are provided, this effect is less severe, although still present to some extent.

To provide for shifts in the location of maximum moments, the Code requires the extension of reinforcement a distance d or $12d_b$ beyond the point at which it is calculated to be no longer required to resist flexure or provide stiffness to satisfy deflection requirements, except as noted.

GFRP-reinforced concrete beams are more likely to have the amount of required reinforcement controlled by serviceability requirements than are steel-reinforced concrete beams. In lieu of detailed deflection calculations, the point

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9.7.3.4 Continuing flexural tension reinforcement shall have an embedment length at least ℓ_d beyond the point where terminated tension reinforcement is no longer required to resist flexure.

9.7.3.5 Flexural tension reinforcement shall not be terminated in a tension zone unless (a), (b), or (c) is satisfied:

(a) $V_u \leq (2/3)\phi V_n$ at the cutoff point

(b) For No. M32 bars and smaller, continuing reinforcement provides double the area required for flexural strength at the cutoff point and $V_u \leq (3/4)\phi V_n$

(c) Stirrup area in excess of that required for shear and torsion is provided along each terminated bar over a distance $3/4d$ from the termination point. Excess stirrup area shall be at least $0.41 b_w s/f_{ft}$. Spacing s shall not exceed $d/(8\beta_b)$.

=**9.7.3.6** Adequate anchorage shall be provided for tension reinforcement where reinforcement stress is not directly proportional to moment, such as in sloped, stepped, or tapered beams, or where tension reinforcement is not parallel to the compression face.

9.7.3.7 Intentionally left blank.

9.7.3.8 *Termination of GFRP reinforcement*

=**9.7.3.8.1** At simple supports, at least one-third of the maximum positive moment reinforcement shall extend along the beam bottom into the support at least 150 mm, except for precast beams where such reinforcement shall extend at least to the center of the bearing length.

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at which bars are no longer required to satisfy deflection requirements can be located at sections where the value of I_e, calculated from Table 24.2.3.5 using I_{cr} for the continuing bars and replacing M_a with the service moment at the cut-off location, is not less than the value of I_e calculated from Table 24.2.3.5 at the location of maximum moment.

Cutoff points of bars to meet this requirement are illustrated in Fig. R9.7.3.2. If different bar sizes are used, the extension should be in accordance with the diameter of the bar being terminated.

R9.7.3.4 Local peak stresses exist in the remaining bars wherever adjacent bars are cut off in tension regions. In Fig. R9.7.3.2, an "x" is used to indicate the point where terminated tension reinforcement is no longer required to resist flexure. If bars were cut off at this location (the required cutoff location is beyond this point in accordance with 9.7.3.3), peak stresses in the continuing bars would reach f_{fr} at "x". Therefore, the continuing reinforcement is required to have a full ℓ_d extension as indicated.

R9.7.3.5 Reduced shear strength and loss of ductility when bars are cut off in a tension zone, as in Fig. R9.7.3.2, have been reported for steel-reinforced concrete beams. The Code does not permit flexural reinforcement to be terminated in a tension zone unless additional conditions are satisfied. Flexural cracks tend to open at low load levels wherever any reinforcement is terminated in a tension zone. If the stress in the continuing reinforcement and the shear strength are each near their limiting values, diagonal tension cracking tends to develop prematurely from these flexural cracks. Diagonal cracks are less likely to form where shear stress is low (9.7.3.5(a)) or flexural reinforcement stress is low (9.7.3.5(b)). Diagonal cracks can be restrained by closely spaced stirrups (9.7.3.5(c)). These requirements are not intended to apply to tension splices that are covered by 25.5.

R9.7.3.8 *Termination of GFRP reinforcement*

=**R9.7.3.8.1** Positive moment reinforcement is extended into the support to provide for some shifting of the moments due to changes in loading, settlement of supports, and lateral loads. It also enhances structural integrity.

For precast beams, tolerances and reinforcement cover should be considered to avoid bearing on plain concrete where reinforcement has been discontinued.

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9.7.3.8.2 At other supports, at least one-fourth of the maximum positive moment reinforcement shall extend along the beam bottom into the support at least 150 mm and, if the beam is part of the primary lateral-load-resisting system, shall be anchored to develop f_{fu} at the face of the support.

9.7.3.8.3 At simple supports and points of inflection, d_b for positive moment tension reinforcement shall be limited such that ℓ_d for that reinforcement satisfies (a) or (b). If reinforcement terminates beyond the centerline of supports by a standard hook or a mechanical anchorage at least equivalent to a standard hook, (a) or (b) need not be satisfied.

(a) $\ell_d \leq (1.3M_n/V_u + \ell_a)$ if end of reinforcement is confined by a compressive reaction

(b) $\ell_d \leq (M_n/V_u + \ell_a)$ if end of reinforcement is not confined by a compressive reaction

M_n and V_u are calculated at the section. At a support, ℓ_a is the embedment length beyond the center of the support. At a point of inflection, ℓ_a is the embedment length beyond the point of inflection limited to the greater of d and $12d_b$.

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R9.7.3.8.2 Development of the positive moment reinforcement at the support is required for beams that are part of the primary lateral-load-resisting system to provide deformability in the event of moment reversal.

R9.7.3.8.3 The diameter of the positive moment tension reinforcement is limited to ensure that the bars are developed in a length short enough such that the moment capacity is greater than the applied moment over the entire length of the beam. As illustrated in the moment diagram of Fig. R9.7.3.8.3(a), the slope of the moment diagram is V_u, while the slope of moment development is M_n/ℓ_d, where M_n is the nominal flexural strength of the cross section. The stress in the GFRP reinforcement f_{fr} is equal to f_{fu} for tension-controlled designs and is less than f_{fu} for any other case. By sizing the reinforcement such that the capacity slope M_n/ℓ_d equals or exceeds the demand slope V_u, proper development is provided. Therefore, M_n/V_u represents the available development length. Under favorable support conditions, a 30% increase for M_n/V_u is permitted when the ends of the reinforcement are confined by a compressive reaction.

The application of this provision is illustrated in Fig. R9.7.3.8.3(b) for simple supports and in Fig. R9.7.3.8.3(c) for points of inflection. For example, the bar size provided at a simple support is satisfactory only if the corresponding bar, ℓ_d, calculated in accordance with 25.4.2, does not exceed $1.3M_n/V_u + \ell_a$.

The ℓ_a to be used at points of inflection is limited to the effective depth of the member d or 12 bar diameters ($12d_b$), whichever is greater. The ℓ_a limitation is provided because test data are not available to show that a long end anchorage length will be fully effective in developing a bar that has only a short length between a point of inflection and a point of maximum stress.

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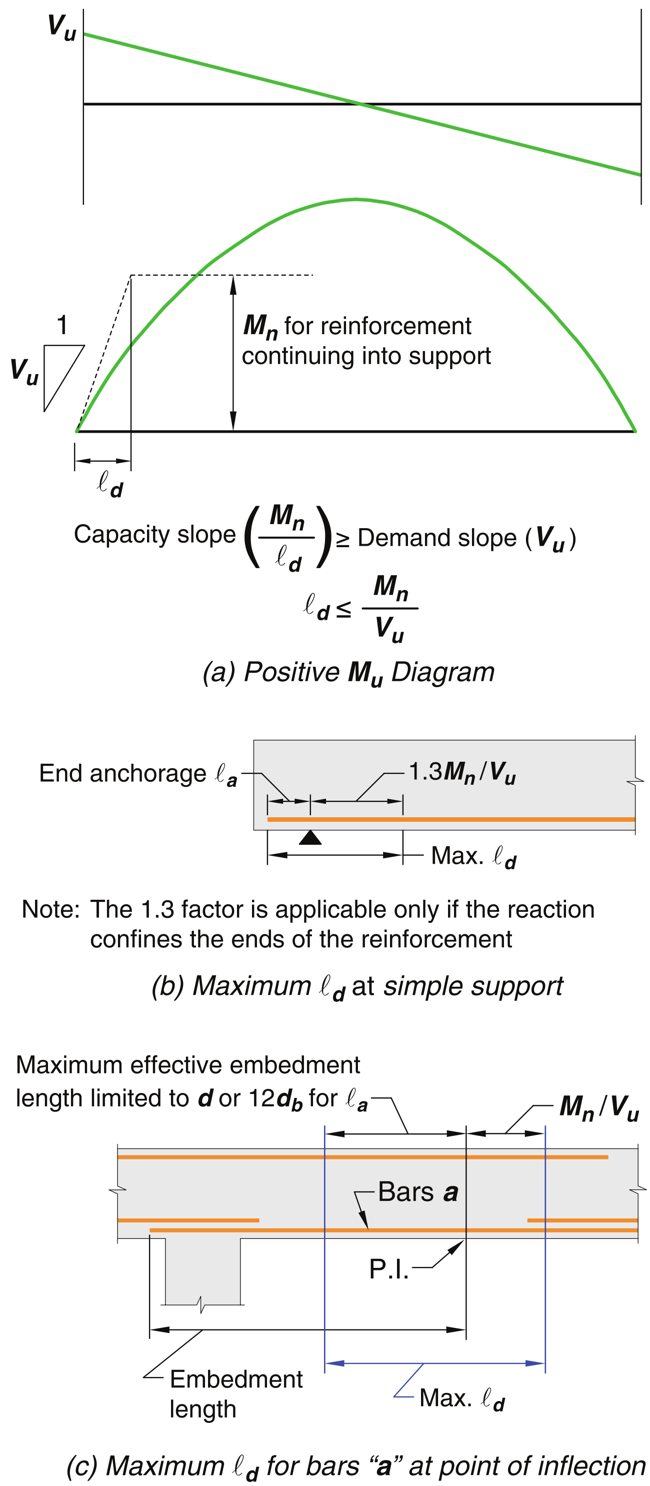

Fig. R9.7.3.8.3—Determination of maximum bar size according to 9.7.3.8.3.

=**9.7.3.8.4** At least one-third of the negative moment reinforcement at a support shall have an embedment length beyond the point of inflection at least the greatest of d, $12d_b$, and $\ell_n/16$.

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9.7.4 *Flexural reinforcement in prestressed beams*—Out of scope

9.7.5 *GFRP longitudinal torsional reinforcement*

9.7.5.1 If torsional reinforcement is required, longitudinal torsional reinforcement shall be distributed around the perimeter of closed stirrups that satisfy 25.7.1.6 with a spacing not greater than 300 mm. The longitudinal reinforcement shall be inside the stirrup, and at least one longitudinal bar shall be placed in each corner.

9.7.5.2 Longitudinal torsional reinforcement shall have a diameter at least 0.084 times the transverse reinforcement spacing, but not less than 10 mm.

⁼**9.7.5.3** Longitudinal torsional reinforcement shall extend for a distance of at least ($\boldsymbol{b_t} + \boldsymbol{d}$) beyond the point required by analysis.

⁼**9.7.5.4** Longitudinal torsional reinforcement shall be developed at the face of the support at both ends of the beam.

9.7.6 *GFRP transverse reinforcement*

9.7.6.1 *General*

⁼**9.7.6.1.1** Transverse reinforcement shall be in accordance with this section. The most restrictive requirements shall apply.

⁼**9.7.6.1.2** Details of transverse reinforcement shall be in accordance with 25.7.

9.7.6.2 *Shear*

9.7.6.2.1 If required, shear reinforcement shall be provided using GFRP stirrups.

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R9.7.5 *GFRP longitudinal torsional reinforcement*

R9.7.5.1 Longitudinal reinforcement is needed to resist the sum of the longitudinal tensile forces due to torsion. Because the force acts along the centroidal axis of the section, the centroid of the additional longitudinal reinforcement for torsion should approximately coincide with the centroid of the section. The Code accomplishes this by requiring the longitudinal torsional reinforcement be distributed around the perimeter of the closed stirrups. Longitudinal bars are required in each corner of the stirrups to provide anchorage for the stirrup legs. Corner bars have also been found to be effective in developing torsional strength and controlling cracks.

R9.7.5.2 Longitudinal reinforcement should be sized to prevent the bars from bending outward between stirrups, weakening the beam. In tests with steel-reinforced concrete, longitudinal corner bars with a diameter 0.032 times the stirrup spacing bent outward at failure (Mitchell and Collins 1976). The 0.084 value specified for longitudinal GFRP bars is twice the amount required by ACI 318 for steel reinforcement to account for the lower stiffness of GFRP compared to steel.

⁼**R9.7.5.3** The distance ($\boldsymbol{b_t} + \boldsymbol{d}$) beyond the point at which longitudinal torsional reinforcement is calculated to be no longer required is greater than that used for shear and flexural reinforcement because torsional diagonal tension cracks develop in a helical form. The same distance is required by 9.7.6.3.2 for transverse torsional reinforcement.

⁼**R9.7.5.4** Longitudinal torsional reinforcement required at a support should be adequately anchored into the support. Sufficient embedment length should be provided outside the inner face of the support to develop the needed tensile force in the bars. This may require hooks or horizontal U-shaped bars lapped with the longitudinal torsional reinforcement.

R9.7.6 *GFRP transverse reinforcement*

R9.7.6.2 *Shear*

R9.7.6.2.1 If a reinforced concrete beam is cast monolithically with a supporting beam and intersects one or both side faces of a supporting beam, the soffit of the supporting

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beam may be subject to premature failure unless additional transverse reinforcement, commonly referred to as hanger reinforcement, is provided (Mattock and Shen 1992). The hanger reinforcement (Fig. R9.7.6.2.1), placed in addition to other transverse reinforcement, is provided to transfer shear from the end of the supported beam.

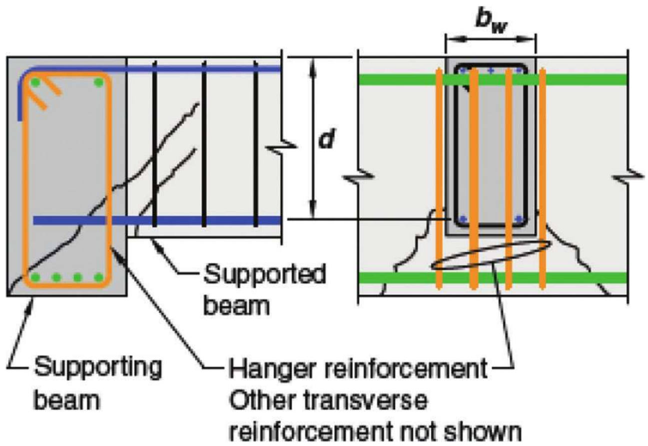

Fig. R9.7.6.2.1—Hanger reinforcement for shear transfer.

9.7.6.2.2 Maximum spacing of legs of shear reinforcement along the length of the member and across the width of the member shall be in accordance with Table 9.7.6.2.2.

Table 9.7.6.2.2—Maximum spacing of legs of shear reinforcement

V_f	Maximum *s*, mm		
		Along width	Across width
$\leq 0.33\sqrt{f'_c}b_w d$	Lesser of	$d/2$	d
		600	
$> 0.33\sqrt{f'_c}b_w d$	Lesser of	$d/4$	$d/2$
		300	

R9.7.6.2.2 Reduced stirrup spacing across the beam width provides a more uniform transfer of diagonal compression across the beam web, enhancing shear capacity. Laboratory tests (Leonhardt and Walther 1964; Anderson and Ramirez 1989; Lubell et al. 2009) of wide steel-reinforced concrete members with large spacing of legs of shear reinforcement across the member width indicate that the nominal shear capacity is not always achieved. The intent of this provision is to provide multiple stirrup legs across wide beams and one-way slabs that require stirrups.

9.7.6.2.3 Intentionally left blank.

9.7.6.2.4 Intentionally left blank.

9.7.6.3 *Torsion*

R9.7.6.3 *Torsion*

9.7.6.3.1 If required, transverse torsional reinforcement shall be closed stirrups satisfying 25.7.1.6.

R9.7.6.3.1 The stirrups are required to be closed because inclined cracking due to torsion may occur on all faces of a member.

In the case of sections subjected primarily to torsion, the concrete side cover over the stirrups spalls off at high torques (Mitchell and Collins 1976). This renders lap-spliced stirrups ineffective, leading to a premature torsional failure (Behera and Rajagopalan 1969). Therefore, closed stirrups should not be made up of pairs of U-stirrups lapping one another. However, pairs of C-shaped stirrups or combination of C-shaped and U-shaped stirrups can be used as closed stirrups, as outlined in 25.7.1.6.1.

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9.7.6.3.2 Transverse torsional reinforcement shall extend a distance of at least ($b_t + d$) beyond the point required by analysis.

9.7.6.3.3 Spacing of transverse torsional reinforcement shall not exceed the lesser of $p_h/8$ and 300 mm.

9.7.6.3.4 Intentionally left blank.

9.7.6.4 *Lateral support of GFRP reinforcement in the compression zone*

9.7.6.4.1 Transverse reinforcement shall be provided throughout the distance where longitudinal reinforcement is present in the compression zone. Lateral support of longitudinal reinforcement in the compression zone shall be provided by closed stirrups in accordance with 9.7.6.4.2 through 9.7.6.4.4.

9.7.6.4.2 No. M10 bars or larger shall be used as transverse reinforcement.

9.7.6.4.3 Spacing of transverse reinforcement shall not exceed the least of (a) through (c):

(a) $12d_b$ of longitudinal reinforcement
(b) $24d_b$ of transverse reinforcement
(c) Least dimension of beam

9.7.6.4.4 Longitudinal reinforcement in the compression zone shall be arranged such that every corner and alternate bar shall be enclosed by the corner of the transverse reinforcement with an included angle of not more than 135 degrees, and no bar shall be farther than 150 mm clear on each side along the transverse reinforcement from such an enclosed bar.

9.7.7 *GFRP structural integrity reinforcement in cast-in-place beams*

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R9.7.6.3.2 The distance ($b_t + d$) beyond the point at which transverse torsional reinforcement is calculated to be no longer required is greater than that used for shear and flexural reinforcement because torsional diagonal tension cracks develop in a helical form. The same distance is required by 9.7.5.3 for longitudinal torsional reinforcement.

R9.7.6.3.3 Spacing of the transverse torsional reinforcement is limited to ensure development of the torsional strength of the beam, prevent excessive loss of torsional stiffness after cracking, and control crack widths. For a square cross section, the $p_h/8$ limitation requires stirrups at approximately $d/2$, which corresponds to 9.7.6.2.

R9.7.6.4 *Lateral support of GFRP reinforcement in the compression zone*

R9.7.6.4.1 Providing lateral support to all bars in the compression zone is good detailing practice.

R9.7.6.4.3 The difference in tie spacing between GFRP-reinforced concrete members with bars in compression and steel-reinforced concrete members with bars in compression is discussed in R25.7.2.1.

R9.7.7 *GFRP structural integrity reinforcement in cast-in-place beams*

Experience has shown that the overall integrity of a steel-reinforced concrete structure can be substantially enhanced by minor changes in detailing of reinforcement and connections. A similar enhancement through detailing is expected to occur in GFRP-reinforced concrete structures. It is the intent of this section of the Code to improve the redundancy and deformability in structures so that in the event of damage to a major supporting element or an abnormal loading event, the resulting damage may be localized and the structure will have a higher probability of maintaining overall stability.

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If the depth of a continuous beam changes at a support, the bottom reinforcement in the deeper member should be terminated into the support with a standard hook or headed bar and the bottom reinforcement in the shallower member should be extended into and fully developed in the deeper member.

9.7.7.1 For beams along the perimeter of the structure, structural integrity reinforcement shall be in accordance with (a) through (c):

(a) At least one-fourth of the maximum positive moment reinforcement, but not less than two bars, shall be continuous.

(b) At least one-sixth of the negative moment reinforcement at the support, but not less than two bars, shall be continuous.

(c) Longitudinal structural integrity reinforcement shall be enclosed by closed stirrups in accordance with 25.7.1.6 along the clear span of the beam.

R9.7.7.1 Requiring continuous top and bottom reinforcement in perimeter or spandrel beams provides a continuous tie around the structure. It is not the intent to require a tension tie of continuous reinforcement of constant size around the entire perimeter of a structure, but rather to require that one-half of the top flexural reinforcement required to extend past the point of inflection by 9.7.3.8.4 be further extended and spliced at or near midspan as required by 9.7.7.5. Similarly, the bottom reinforcement required to extend into the support in 9.7.3.8.2 should be made continuous or spliced with bottom reinforcement from the adjacent span. At noncontinuous supports, the longitudinal reinforcement is anchored as required by 9.7.7.4.

9.7.7.2 For other than perimeter beams, structural integrity reinforcement shall be in accordance with (a) or (b):

(a) At least one-fourth of the maximum positive moment reinforcement, but not less than two bars, shall be continuous.

(b) Longitudinal reinforcement shall be enclosed by closed stirrups in accordance with 25.7.1.6 along the clear span of the beam.

R9.7.7.2 At noncontinuous supports, the longitudinal reinforcement is anchored as required by 9.7.7.4.

=**9.7.7.3** Longitudinal structural integrity reinforcement shall pass through the region bounded by the longitudinal reinforcement of the column.

R9.7.7.3 In the case of walls providing vertical support, the longitudinal reinforcement should pass through or be anchored in the wall.

9.7.7.4 Longitudinal structural integrity reinforcement at noncontinuous supports shall be anchored to develop f_{fu} at the face of supports.

=**9.7.7.5** If splices are necessary in continuous structural integrity reinforcement, the reinforcement shall be spliced in accordance with (a) and (b):

(a) Positive moment reinforcement shall be spliced at or near the support

(b) Negative moment reinforcement shall be spliced at or near midspan

9.7.7.6 Splices shall be full mechanical in accordance with 25.5.7 or Class B tension lap splices in accordance with 25.5.2.

9.8—One-way joist systems

9.8.1 *General*

9.8.1.1 One-way joist construction consists of a monolithic combination of regularly spaced ribs and a top slab designed to span in one direction.

R9.8—One-way joist systems

R9.8.1 *General*

The empirical limits established for reinforced concrete joist floors are based on successful past performance of steel-reinforced concrete joist construction using standard joist forming systems.

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=**9.8.1.2** Width of ribs shall be at least 100 mm at any location along the depth.

=**9.8.1.3** Overall depth of ribs shall not exceed 3.5 times the minimum width.

=**9.8.1.4** Clear spacing between ribs shall not exceed 760 mm.

9.8.1.5 Intentionally left blank.

9.8.1.6 For structural integrity, at least one bottom bar in each joist shall be continuous and shall be anchored to develop f_{fu} at the face of support.

=**9.8.1.7** Reinforcement perpendicular to the ribs shall be provided in the slab as required for flexure, considering load concentrations, and shall be at least that required for shrinkage and temperature in accordance with 24.4.

=**9.8.1.8** One-way joist construction not satisfying the limitations of 9.8.1.1 through 9.8.1.4 shall be designed as slabs and beams.

9.8.2 *Joist systems with structural fillers*—Out of scope

9.8.3 *Joist systems with other fillers*—Out of scope

9.9—Deep beams—Out of scope

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=**R9.8.1.4** A limit on the maximum spacing of ribs is required because of the provisions permitting less concrete cover for the reinforcement for these relatively small, repetitive members.

R9.9—Deep beams—Out of scope

Deep beams are members that are loaded on one face and supported on the opposite face such that strut-like compression elements can develop between the loads and supports and that have either clear spans not exceeding four times the overall member depth h or concentrated loads existing within a distance $2h$ from the face of the support. The design of deep beams reinforced with GFRP reinforcement is not covered in this Code due to a lack of published research on this topic.

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CHAPTER 10—COLUMNS

10.1—Scope

10.1.1 This chapter shall apply to the design of nonprestressed columns reinforced with GFRP bars, including reinforced concrete pedestals.

10.1.2 Intentionally left blank.

10.2—General

10.2.1 *Materials*

=**10.2.1.1** Design properties for concrete shall be selected to be in accordance with Chapter 19.

10.2.1.2 Design properties for GFRP reinforcement shall be selected to be in accordance with Chapter 20.

=**10.2.1.3** Materials, design, and detailing requirements for embedments in concrete shall be in accordance with 20.6.

10.2.2 *Composite columns*—Out of scope

10.2.3 *Connection to other members*

=**10.2.3.1** For cast-in-place construction, beam-column and slab-column joints shall satisfy Chapter 15.

10.2.3.2 Intentionally left blank.

=**10.2.3.3** Connections of columns to foundations shall satisfy 16.3.

10.3—Design limits

10.3.1 *Dimensional limits*

=**10.3.1.1** For columns with a square, octagonal, or other shaped cross section, it shall be permitted to base gross area considered, required reinforcement, and design strength on a circular section with a diameter equal to the least lateral dimension of the actual shape.

10.3.1.2 Intentionally left blank.

=**10.3.1.3** For columns built monolithically with a concrete wall, the outer limits of the effective cross section of the column shall not be taken greater than 38 mm outside the transverse reinforcement.

=**10.3.1.4** For columns with two or more interlocking spirals, outer limits of the effective cross section shall be taken at a distance outside the spirals equal to the minimum required concrete cover.

COMMENTARY

CHAPTER R10—COLUMNS

R10.3—Design limits

R10.3.1 *Dimensional limits*

=Explicit minimum sizes for columns are not specified to permit the use of reinforced concrete columns with small cross sections in lightly loaded structures, such as low-rise residential and light office buildings. If small cross sections are used, there is a greater need for careful workmanship, and shrinkage stresses have increased significance.

CODE

10.3.1.5 If a reduced effective area is considered according to 10.3.1.1, 10.3.1.3, or 10.3.1.4, structural analysis and design of other parts of the structure that interact with the column shall be based on the actual cross section.

10.3.2 *Strain limits*

10.3.2.1 If factored axial compression $P_u > 0.10f_c'A_g$, the tensile design strain of the longitudinal bars shall be limited to 0.01. The corresponding design strength, f_{fd}, shall be the lesser of f_{fu} and $0.01E_f$.

10.4—Required strength

10.4.1 *General*

10.4.1.1 Required strength shall be calculated in accordance with the factored load combinations in Chapter 5.

10.4.1.2 Required strength shall be calculated in accordance with the analysis procedures in Chapter 6.

10.4.2 *Factored axial force and moment*

10.4.2.1 P_u and M_u occurring simultaneously for each applicable factored load combination shall be considered.

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R10.3.2 *Strain limits*

R10.3.2.1 Attaining the full tensile capacity of the GFRP bars requires a degree of curvature which may be either unattainable or unacceptable for columns. The tensile rupture strain of GFRP bars exceeds 2%. Such a large strain leads to unacceptably large deformations (Guérin et al. 2018a). Based on test results and parametric investigation, the failure of GFRP-reinforced concrete columns under large eccentric loading is not triggered by tensile bar rupture, provided that the minimum reinforcement ratio is not less than 1% for normal-strength concrete. The maximum average tensile strain attained by the test specimens at peak load was less than 50% of the ultimate tensile strain of the GFRP bars (Guérin et al. 2018b). Thus, for design purposes if factored axial compression $P_u > 0.10f_c'A_g$, the ultimate tensile design strain may not exceed a fixed limit of 1%. (Jawaheri Zadeh and Nanni 2013; Guérin et al. 2018b; Hadhood et al. 2019).

R10.4—Required strength

R10.4.2 *Factored axial force and moment*

R10.4.2.1 The critical load combinations may be difficult to discern without methodically checking each combination. As illustrated in Fig. R10.4.2.1, considering only the factored load combinations associated with maximum axial force (LC1) and with maximum bending moment (LC2) does not necessarily provide a code-compliant design for other load combinations such as LC3.

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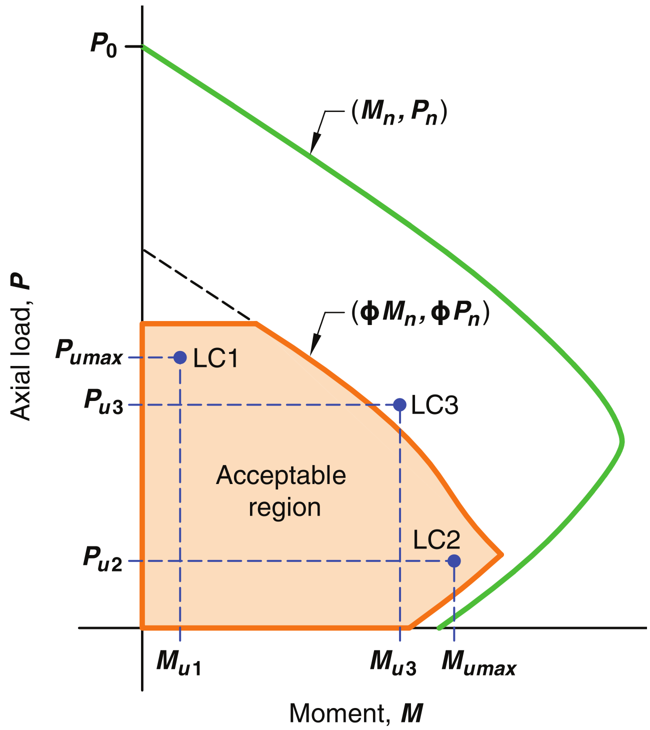

Fig. R10.4.2.1—Critical column load combination.

10.5—Design strength

10.5.1 *General*

⁼**10.5.1.1** For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$, including (a) through (d). Interaction between load effects shall be considered:

(a) $\phi P_n \geq P_u$
(b) $\phi M_n \geq M_u$
(c) $\phi V_n \geq V_u$
(d) $\phi T_n \geq T_u$

⁼**10.5.1.2** ϕ shall be determined in accordance with 21.2.

10.5.2 *Axial force and moment*

⁼**10.5.2.1** P_n and M_n shall be calculated in accordance with 22.4.

10.5.2.2 Intentionally left blank.

10.5.3 *Shear*

⁼**10.5.3.1** V_n shall be calculated in accordance with 22.5.

10.5.4 *Torsion*

⁼**10.5.4.1** If $T_u \geq \phi T_{th}$, where T_{th} is given in 22.7, torsion shall be considered in accordance with Chapter 9.

R10.5—Design strength

R10.5.1 *General*

⁼**R10.5.1.1** Refer to R9.5.1.1.

R10.5.4 *Torsion*

⁼Torsion acting on columns in buildings is typically negligible and is rarely a governing factor in the design of columns.

CODE

10.6—GFRP reinforcement limits

10.6.1 *Minimum and maximum GFRP longitudinal reinforcement*

10.6.1.1 Area of longitudinal reinforcement shall be at least $\mathbf{0.01}\boldsymbol{A_g}$ but shall not exceed $\mathbf{0.08}\boldsymbol{A_g}$.

10.6.1.2 Intentionally left blank.

10.6.2 *Minimum GFRP shear reinforcement*

10.6.2.1 A minimum area of shear reinforcement, $\boldsymbol{A_{fv,min}}$, shall be provided in all regions where $\boldsymbol{V_u \geq \phi 0.21 k_{cr} \sqrt{f_c'} b_w d}$.

10.6.2.2 If shear reinforcement is required, $\boldsymbol{A_{fv,min}}$ shall be the greater of (a) and (b):

(a) $0.062\sqrt{f_c'}\dfrac{b_w s}{f_{ft}}$

(b) $0.35\dfrac{b_w s}{f_{ft}}$

10.7—GFRP reinforcement detailing

10.7.1 *General*

=**10.7.1.1** Concrete cover for reinforcement shall be in accordance with 20.5.1.

COMMENTARY

R10.6—GFRP reinforcement limits

R10.6.1 *Minimum and maximum GFRP longitudinal reinforcement*

R10.6.1.1 Limits are provided for both the minimum and maximum longitudinal reinforcement ratios.

Minimum reinforcement—Reinforcement is necessary to provide resistance to bending, which may exist regardless of analytical results. The proposed 1% limit ensures maintaining the section integrity to achieve the nominal capacity of columns while keeping the GFRP bars on the tension side intact (Hadhood et al. 2019). This limit was validated by structural tests in Hadi and Youssef (2016) and Guérin et al. (2018b) and verified by theoretical analyses (Hadhood et al. 2017c). These studies did not consider the effects of minimum GFRP reinforcement on the control of creep and shrinkage deformations of concrete under sustained load.

Maximum reinforcement—The amount of longitudinal reinforcement is limited to ensure that concrete can be effectively consolidated around the bars and to ensure that columns designed according to the Code are similar to the test specimens. Khorramian and Sadeghian (2017) performed structural tests validating the performance of GFRP-reinforced concrete columns with reinforcement ratios as high as 5.3%. The 0.08 limit applies at all sections, including splice regions, and can also be considered a practical maximum for longitudinal reinforcement in terms of economy and requirements for placing. Longitudinal reinforcement in columns should usually not exceed 4% if the column bars are required to be lap spliced, as the lap splice zone will have twice as much reinforcement if all lap splices occur at the same location.

R10.6.2 *Minimum GFRP shear reinforcement*

=**R10.6.2.1** The basis for the minimum shear reinforcement is the same for columns and beams. Refer to R9.6.3 for more information.

R10.7—GFRP reinforcement detailing

CODE

10.7.1.2 Development lengths of reinforcement shall be in accordance with 25.4.

10.7.2 *GFRP reinforcement spacing*

10.7.2.1 Minimum spacing ***s*** shall be in accordance with 25.2.

10.7.3 *GFRP longitudinal reinforcement*

10.7.3.1 The minimum number of longitudinal bars shall be (a), (b), or (c):

(a) Three within triangular ties
(b) Four within rectangular or circular ties
(c) Six enclosed by spirals

10.7.3.2 Intentionally left blank.

10.7.4 *Offset bent longitudinal reinforcement*—Out of scope

10.7.5 *Splices of GFRP longitudinal reinforcement*

10.7.5.1 *General*

10.7.5.1.1 Lap splices and mechanical splices shall be permitted.

10.7.5.1.2 Splices shall satisfy requirements for all factored load combinations.

10.7.5.1.3 Splices shall be in accordance with 25.5 and, if applicable, shall satisfy the requirements of 10.7.5.2 for lap splices.

10.7.5.2 *Lap splices*

COMMENTARY

R10.7.3 *GFRP longitudinal reinforcement*

R10.7.3.1 At least four longitudinal bars are required when bars are enclosed by rectangular or circular ties. For other tie shapes, one bar should be provided at each apex or corner and proper transverse reinforcement provided. For example, tied triangular columns require at least three longitudinal bars, with one at each apex of the triangular ties. For bars enclosed by spirals, at least six bars are required.

If the number of bars in a circular arrangement is less than eight, the orientation of the bars may significantly affect the moment strength of eccentrically loaded columns and should be considered in design.

R10.7.5 *Splices of GFRP longitudinal reinforcement*

R10.7.5.1 *General*

R10.7.5.1.2 Frequently, the basic gravity load combination will govern the design of the column itself, but a load combination including wind or earthquake effects may induce greater tension in some column bars. Each bar splice should be designed for the maximum calculated bar tensile force.

R10.7.5.2 *Lap splices*

In columns subject to moment and axial force, tensile stresses may occur on one face of the column for moderate and large eccentricities as shown in Fig. R10.7.5.2. If such stresses occur, 10.7.5.2.2 requires tension splices to be used.

The splice requirements have been formulated on the basis that a compression lap splice has a tensile strength of at least $0.25f_{fu}$. Therefore, even if column bars are designed for compression according to 10.7.5.2.1, some tensile strength is inherently provided.

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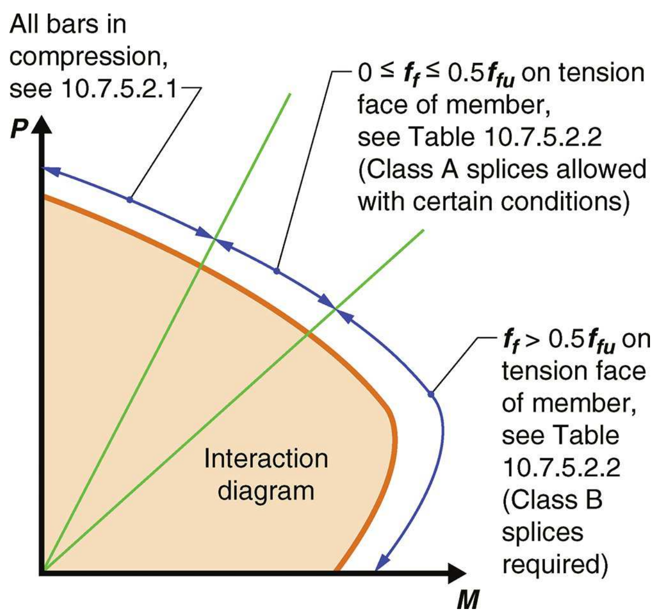

Fig. R10.7.5.2—Lap splice requirements for columns.

10.7.5.2.1 If the bar force due to factored loads is compressive, compression lap splices shall be permitted. Compression splices shall be designed in accordance with 25.5 assuming a maximum compressive stress of $\mathbf{0.25}\boldsymbol{f_{fu}}$.

R10.7.5.2.1 Assuming strain compatibility between the GFRP bars in compression and concrete, the largest expected stress in GFRP bars in compression is 25% of f_{fu}.

=**10.7.5.2.2** If the bar force due to factored loads is tensile, tension lap splices shall be in accordance with Table 10.7.5.2.2.

Table 10.7.5.2.2—Tension lap splice class

Tensile bar stress	Splice details	Splice type
$\leq 0.5f_{fu}$	$\leq$ 50% bars spliced at any section and lap splices on adjacent bars staggered by at least ℓ_d	Class A
	Other	Class B
$> 0.5f_{fu}$	All cases	Class B

10.7.5.3 *End bearing splices*—Out of scope

10.7.6 *GFRP transverse reinforcement*

R10.7.6 *GFRP transverse reinforcement*

10.7.6.1 *General*

R10.7.6.1 *General*

=**10.7.6.1.1** Transverse reinforcement shall satisfy the most restrictive requirements for reinforcement spacing.

10.7.6.1.2 Details of transverse reinforcement shall be in accordance with 25.7.2 for ties, or 25.7.3 for spirals.

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10.7.6.1.3 Intentionally left blank.

10.7.6.1.4 Longitudinal reinforcement shall be laterally supported using ties in accordance with 10.7.6.2 or spirals in accordance with 10.7.6.3 unless tests and structural analyses demonstrate adequate strength and feasibility of construction.

R10.7.6.1.4 All longitudinal bars in compression should be enclosed within transverse reinforcement. Where longitudinal bars are arranged in a circular pattern, only one circular tie per specified spacing is required. This requirement can be satisfied by a continuous circular tie (helix), with the maximum pitch being equal to the required tie spacing.

Precast columns with cover less than 38 mm, columns of concrete with small size coarse aggregate, wall-like columns, and other unusual columns may require special designs for transverse reinforcement.

10.7.6.1.5 Intentionally left blank.

10.7.6.1.6 If mechanical couplers or extended bars for connection to a precast element are placed in the ends of columns or pedestals, the mechanical couplers or extended bars shall be enclosed by transverse reinforcement. The transverse reinforcement shall be distributed within 130 mm of the ends of the column or pedestal and shall consist of at least two No. M13 or three No. M10 ties.

R10.7.6.1.6 Confinement improves load transfer from the mechanical couplers to the column or pedestal where concrete may crack in the vicinity of the mechanical couplers. Such cracking can occur due to unanticipated forces caused by temperature, restrained shrinkage, accidental impact during construction, and similar effects.

10.7.6.2 *Lateral support of GFRP longitudinal bars using GFRP ties*

R10.7.6.2 *Lateral support of GFRP longitudinal bars using GFRP ties*

10.7.6.2.1 In any story, the bottom tie shall be located not more than one-half the tie spacing above the top of footing or slab.

10.7.6.2.2 In any story, the top tie shall be located not more than one-half the tie spacing below the lowest horizontal reinforcement in the slab, drop panel, or shear cap. If beams or brackets frame into all sides of the column, the top tie shall be located not more than 75 mm below the lowest horizontal reinforcement in the shallowest beam or bracket.

R10.7.6.2.2 For rectangular columns, beams or brackets framing into all four sides at the same elevation are considered to provide restraint over a joint depth equal to that of the shallowest beam or bracket. For columns with other shapes, four beams framing into the column from two orthogonal directions are considered to provide equivalent restraint.

10.7.6.3 *Lateral support of GFRP longitudinal bars using GFRP spirals*

R10.7.6.3 *Lateral support of GFRP longitudinal bars using GFRP spirals*

10.7.6.3.1 In any story, the bottom of the spiral shall be located at the top of footing or slab.

10.7.6.3.2 In any story, the top of the spiral shall be located in accordance with Table 10.7.6.3.2.

R10.7.6.3.2 Refer to R10.7.6.2.2.

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COMMENTARY

Table 10.7.6.3.2—Spiral extension requirements at top of column

Framing at column end	Extension requirements
Beams or brackets frame into all sides of the column	Extend to the level of the lowest horizontal reinforcement in members supported above.
Beams or brackets do not frame into all sides of the column	Extend to the level of the lowest horizontal reinforcement in members supported above. Additional column ties shall extend above termination of spiral to bottom of slab, drop panel, or shear cap.
Columns with capitals	Extend to the level at which the diameter or width of capital is twice that of the column.

10.7.6.4 *Lateral support of offset bent longitudinal bars*—Out of scope

10.7.6.5 *Shear*

10.7.6.5.1 If required, shear reinforcement shall be provided using GFRP ties or spirals.

10.7.6.5.2 Maximum spacing of shear reinforcement shall be in accordance with Table 10.7.6.5.2.

Table 10.7.6.5.2—Maximum spacing of shear reinforcement

V_f	Maximum s, mm	
$\leq 0.33\sqrt{f_c'}b_w d$	Lesser of:	$d/2$
		600
$> 0.33\sqrt{f_c'}b_w d$	Lesser of:	$d/4$
		300

CODE

CHAPTER 11—WALLS

11.1—Scope

11.1.1 This chapter shall apply to the design of nonprestressed walls including (a) through (c):

(a) Cast-in-place
(b) Precast in-plant
(c) Precast on-site including tilt-up

11.1.2 Intentionally left blank.

11.1.3 Intentionally left blank.

11.1.4 Design of cantilever retaining walls shall be in accordance with Chapter 13.

=**11.1.5** Design of walls as grade beams shall be in accordance with 13.3.5.

11.1.6 Cast-in-place flat walls with insulating forms using polypropylene crossties shall be permitted by this Code for use in one- or two-story buildings.

11.2—General

11.2.1 *Materials*

=**11.2.1.1** Design properties for concrete shall be selected to be in accordance with Chapter 19.

11.2.1.2 Design properties for GFRP reinforcement shall be selected to be in accordance with Chapter 20.

=**11.2.1.3** Materials, design, and detailing requirements for embedments in concrete shall be in accordance with 20.6.

11.2.2 *Connection to other members*

11.2.2.1 Intentionally left blank.

=**11.2.2.2** Connections of walls to foundations shall satisfy 16.3.

11.2.3 *Load distribution*

=**11.2.3.1** Unless otherwise demonstrated by an analysis, the horizontal length of wall considered as effective for resisting each concentrated load shall not exceed the lesser of the center-to-center distance between loads, and the bearing width plus four times the wall thickness. Effective horizontal length for bearing shall not extend beyond vertical wall joints unless design provides for transfer of forces across the joints.

11.2.4 *Intersecting elements*

COMMENTARY

CHAPTER R11—WALLS

R11.1—Scope

R11.1.1 This chapter applies generally to walls as vertical and lateral force-resisting members. Provisions for in-plane shear in ordinary structural walls are included in this chapter.

R11.1.6 Specific design recommendations for cast-in place walls constructed with insulating concrete forms are not provided in this Code. Guidance can be found in ACI 560R.

R11.2—General

R11.2.4 *Intersecting elements*

CODE

11.2.4.1 Walls shall be anchored to intersecting elements, such as floors and roofs; columns, pilasters, buttresses, or intersecting walls; and to footings.

11.2.4.2 For cast-in-place walls having $P_u > 0.2f_c'A_g$, the portion of the wall within the thickness of the floor system shall have specified compressive strength at least $0.8f_c'$ of the wall.

11.3—Design limits

11.3.1 *Minimum wall thickness*

11.3.1.1 Minimum wall thicknesses shall be in accordance with Table 11.3.1.1. Thinner walls are permitted if adequate strength and stability can be demonstrated by structural analysis.

Table 11.3.1.1—Minimum wall thickness *h*

Wall type	Minimum thickness *h*		
Bearing*	Greater of:	140 mm	(a)
		1/24 the lesser of unsupported length and unsupported height	(b)
Nonbearing	Greater of:	100 mm	(c)
		1/30 the lesser of unsupported length and unsupported height	(d)
Exterior basement and foundation*	190 mm		(e)

*Only applies to walls designed in accordance with the simplified design method of 11.5.3.

11.4—Required strength

11.4.1 *General*

11.4.1.1 Required strength shall be calculated in accordance with the factored load combinations in Chapter 5.

11.4.1.2 Required strength shall be calculated in accordance with the analysis procedures in Chapter 6.

11.4.1.3 Slenderness effects shall be calculated in accordance with 6.6.4 or 6.7.

COMMENTARY

R11.2.4.1 Walls that do not depend on intersecting elements for support, do not have to be connected to those elements. It is not uncommon to separate massive retaining walls from intersecting walls to accommodate differences in deformations.

R11.2.4.2 The 0.8 factor reflects reduced confinement in floor-wall joints compared with floor-column joints under gravity loads.

R11.3—Design limits

R11.3.1 *Minimum wall thickness*

R11.3.1.1 The minimum thickness limits for GFRP- reinforced concrete bearing walls are set to the minimum for unreinforced concrete walls specified in Table 14.3.1.1 of ACI 318. The minimum thickness requirements need not be applied to bearing walls and exterior basement and foundation walls designed by 11.5.2.

R11.4—Required strength

R11.4.1 *General*

R11.4.1.3 The forces typically acting on a wall are illustrated in Fig. R11.4.1.3.

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COMMENTARY

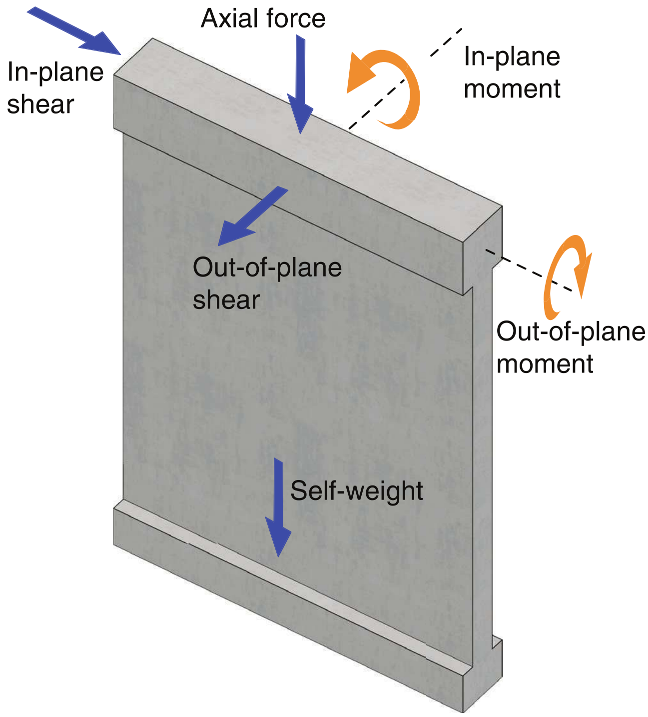

Fig. R11.4.1.3—In-plane and out-of-plane forces.

=**11.4.1.4** Walls shall be designed for eccentric axial loads and any lateral or other loads to which they are subjected.

11.4.2 *Factored axial force and moment*

11.4.2.1 Walls shall be designed for the maximum factored moment M_u that can accompany the factored axial force for each applicable load combination. The factored axial force P_u at given eccentricity shall not exceed $\phi P_{n,max}$, where $P_{n,max}$ shall be as given in 22.4.2.1 and strength reduction factor ϕ shall be that for compression-controlled sections in 21.2.2. The maximum factored moment M_u shall be magnified for slenderness effects in accordance with 6.6.4 or 6.7.

11.4.3 *Factored shear*

=**11.4.3.1** Walls shall be designed for the maximum in-plane V_u and out-of-plane V_u.

11.5—Design strength

11.5.1 *General*

=**11.5.1.1** For each applicable factored load combination, design strength at all sections shall satisfy $\phi S_n \geq U$, including (a) through (c). Interaction between axial load and moment shall be considered.

(a) $\phi P_n \geq P_u$
(b) $\phi M_n \geq M_u$
(c) $\phi V_n \geq V_u$

=**11.5.1.2** ϕ shall be determined in accordance with 21.2.

R11.5—Design strength

CODE

11.5.2 *Axial load and in-plane or out-of-plane flexure*

11.5.2.1 For bearing walls, $\boldsymbol{P_n}$ and $\boldsymbol{M_n}$ (in-plane or out-of-plane) shall be calculated in accordance with 22.4. Alternatively, axial load and out-of-plane flexure shall be permitted to be considered in accordance with 11.5.3.

11.5.2.2 For nonbearing walls, $\boldsymbol{M_n}$ shall be calculated in accordance with 22.3.

11.5.3 *Axial load and out-of-plane flexure – simplified design method*

11.5.3.1 If the resultant of all factored loads is located within the middle third of the thickness of a solid wall with a rectangular cross section, $\boldsymbol{P_n}$ shall be permitted to be calculated by:

$$P_n = 0.45 f_c' A_g \left[1 - \left(\frac{k\ell_c}{32h}\right)^2\right] \quad (11.5.3.1)$$

11.5.3.2 Effective length factor $\boldsymbol{k}$ for use with Eq. (11.5.3.1) shall be in accordance with Table 11.5.3.2.

Table 11.5.3.2—Effective length factor *k* for walls

Boundary conditions	*k*
Walls braced top and bottom against lateral translation and:	
(a) Restrained against rotation at one or both ends (top, bottom, or both)	0.8
(b) Unrestrained against rotation at both ends	1.0
Walls not braced against lateral translation	2.0

11.5.3.3 $\boldsymbol{P_n}$ from Eq. (11.5.3.1) shall be reduced by ϕ for compression-controlled sections in 21.2.2.

11.5.3.4 Wall reinforcement shall be at least that required by 11.6.

11.5.4 *In-plane shear*

COMMENTARY

R11.5.2 *Axial load and in-plane or out-of-plane flexure*

R11.5.2.2 Nonbearing walls, by definition, are not subject to any significant axial force; therefore, flexural strength is not a function of axial force.

R11.5.3.1 The simplified design method applies only to solid rectangular cross sections; all other shapes should be designed in accordance with 11.5.2.

Eccentric axial loads and moments due to out-of-plane forces are used to determine the maximum total eccentricity of the factored axial force $\boldsymbol{P_u}$. When the resultant axial force for all applicable load combinations falls within the middle third of the wall thickness (eccentricity not greater than $\boldsymbol{h/6}$) at all sections along the length of the undeformed wall, no tension is induced in the wall and the simplified design method may be used. The design is then carried out considering $\boldsymbol{P_u}$ as a concentric axial force. The factored axial force $\boldsymbol{P_u}$ should be less than or equal to the design axial strength $\phi\boldsymbol{P_n}$ calculated using Eq. (11.5.3.1).

Equation (11.5.3.1) is based on the resistance of plain concrete walls specified in 14.5.4.2 of ACI 318 and accounts for the lower stiffness of GFRP relative to steel by using a value of $\boldsymbol{0.45 f_c' A_g}$ as the maximum axial capacity of a wall rather than the value of $\boldsymbol{0.55 f_c' A_g}$ used in steel-reinforced concrete.

R11.5.4 *In-plane shear*

CODE

11.5.4.1 V_n shall be calculated in accordance with 11.5.4.2 through 11.5.4.5. Reinforcement shall satisfy the limits of 11.6, 11.7.2, and 11.7.3.

11.5.4.2 V_n at any horizontal section shall not exceed $\mathbf{0.2}f_c'hd$.

11.5.4.3 V_n shall be calculated by:

$$V_n = V_c + V_f \tag{11.5.4.3}$$

11.5.4.4 It shall be permitted to calculate V_c in accordance with 22.5.5 with the term b_w replaced by h and the term d replaced with $0.8\ell_w$. A larger value of d, equal to the distance from extreme compression fiber to center of force of all reinforcement in tension, shall be permitted if the center of tension is calculated by a strain compatibility analysis.

11.5.4.5 V_f shall be provided by transverse shear reinforcement placed in the direction of the applied shear and shall be calculated by:

$$V_f = \frac{A_{fv} f_{ft} d}{s} \tag{11.5.4.5}$$

11.5.5 *Out-of-plane shear*

=**11.5.5.1** V_n shall be calculated in accordance with 22.5.

11.6—GFRP reinforcement limits

11.6.1 If in-plane $V_u \leq \phi 0.21 k_{cr} \sqrt{f_c'} b_w d$, minimum $\rho_{f\ell}$ and minimum ρ_{ft} shall be in accordance with (a) and (b):

(a) $1650/E_f$

(b) 0.0025

These limits need not be satisfied if adequate strength and stability can be demonstrated by structural analysis.

COMMENTARY

R11.5.4.1 Shear in the plane of the wall is primarily of importance for structural walls with a small height-to-length ratio. The design of taller walls, particularly walls with uniformly distributed GFRP reinforcement, will likely be controlled by flexural considerations.

R11.5.4.2 This limit is imposed to guard against diagonal compression failure in shear walls. Refer to R22.5.1.2.

R11.5.4.5 Equation (11.5.4.5) is presented in terms of shear strength V_f provided by the horizontal reinforcement, placed in the direction of applied shear, for direct application in 11.5.4.3.

Vertical shear reinforcement should also be provided in accordance with 11.6 and the spacing limitation of 11.7.2. Arafa et al. (2018a) recommended that the strain in the horizontal bars of walls subject to in-plane shear be limited to 0.005 under factored loads to control the shear crack width in GFRP-reinforced concrete squat walls. The limiting stress on the GFRP shear reinforcement is discussed in more detail in R22.5.3.3.

R11.6—GFRP reinforcement limits

R11.6.1 The minimum reinforcement ratios for GFRP-reinforced concrete walls are the same as provided for shrinkage and temperature in 24.4.3.2. Both horizontal and vertical shear reinforcement are required for all walls. The distributed reinforcement is identified as being oriented parallel to either the longitudinal or transverse axis of the wall. Therefore, for vertical wall segments, the notation used to describe the horizontal distributed reinforcement ratio is ρ_{ft}, and the notation used to describe the vertical distributed reinforcement ratio is $\rho_{f\ell}$.

The minimum area of wall reinforcement for steel-reinforced precast walls has been used for many years and is recommended by the Precast/Prestressed Concrete Institute (PCI MNL-120) and the Canadian Concrete Design Standard (2016). Reduced minimum reinforcement and greater spacings are allowed in steel-reinforced precast wall panels recognizing that precast wall panels have very little restraint at their edges during early stages of curing and develop less shrinkage stress than comparable cast-in-place walls. Currently, there is not enough published research to recommend reducing the minimum reinforcement ratios for precast GFRP-reinforced concrete walls.

11.6.2 If in-plane $V_u \geq \phi 0.21 k_{cr} \sqrt{f_c'} b_w d$, minimum $\rho_{f\ell}$ and minimum ρ_{ft} shall be in accordance with (a) and (b):

(a) $3100/E_f$

(b) 0.0025

R11.6.2 Experimental results on the in-plane shear response of GFRP-reinforced concrete walls with height-to-length ratios ranging from 1.33 to 3.5 are available in the literature (Mohamed et al. 2014b; Arafa et al. 2018a,b; Hassanein et al. 2019). Walls with GFRP web vertical reinforcement ratios between 0.0053 and 0.0062 and GFRP web horizontal reinforcement ratios of 0.0051 had flexural resistances that could be predicted using plane section analysis and shear resistances that could be predicted using calculations accounting for the shear resistance provided by the concrete (V_c) and GFRP horizontal reinforcement (V_f) (Hassanein et al. 2019; Arafa et al. 2018a).

11.7—GFRP reinforcement detailing

R11.7—GFRP reinforcement detailing

11.7.1 *General*

11.7.1.1 Concrete cover for reinforcement shall be in accordance with 20.5.1.

11.7.1.2 Development lengths of reinforcement shall be in accordance with 25.4.

11.7.1.3 Splice lengths of reinforcement shall be in accordance with 25.5.

11.7.2 *Spacing of GFRP longitudinal reinforcement*

11.7.2.1 Spacing s of longitudinal bars in walls shall not exceed the lesser of $3h$ and 300 mm. If shear reinforcement is required for in-plane strength, spacing of longitudinal reinforcement shall not exceed $\ell_w/3$.

11.7.2.2 Intentionally left blank.

11.7.2.3 For walls with h greater than 250 mm, except single-story basement walls and cantilever retaining walls, distributed reinforcement for each direction shall be placed in two layers, one near each face.

11.7.2.4 Flexural tension reinforcement shall be well distributed and placed as close as practicable to the tension face.

11.7.3 *Spacing of GFRP transverse reinforcement*

11.7.3.1 Spacing s of transverse reinforcement in walls shall not exceed the lesser of $3h$ and 300 mm. If shear reinforcement is required for in-plane strength, s shall not exceed $\ell_w/5$.

11.7.3.2 Intentionally left blank.

CODE

11.7.4 *Lateral support of GFRP longitudinal reinforcement*

11.7.4.1 If longitudinal reinforcement is required for axial strength or if A_{fw} exceeds $\mathbf{0.01}A_g$, longitudinal reinforcement shall be laterally supported by transverse ties.

11.7.5 *GFRP reinforcement around openings*

11.7.5.1 In addition to the minimum reinforcement required by 11.6, at least four No. M16 bars in walls having two layers of reinforcement in both directions and two No. M16 bars in walls having a single layer of reinforcement in both directions shall be provided around window, door, and similarly sized openings. In lieu of more detailed analysis that shows lower bar stresses can be considered under factored loads, such bars shall be anchored to develop f_{fu} in tension at the corners of the openings. An additional two No. M16 bars in walls having two layers of reinforcement in both directions and one No. M16 bar in walls having a single layer of reinforcement in both directions shall be placed diagonally at each corner. Diagonal bars shall have a minimum anchorage length of 600 mm from the corner to either end of the bar.

11.8—Alternative method for out-of-plane slender wall analysis—Out of scope

COMMENTARY

R11.7.5 *GFRP reinforcement around openings*

R11.7.5.1 The purpose of the additional reinforcement is to limit crack widths originating at the corners of openings. In steel-reinforced concrete, additional reinforcement consisting of at least two No. M16 bars in walls having two layers of reinforcement in both directions and one No. M16 bar in walls having a single layer of reinforcement around the opening is required. Practical detailing recommendations for these additional bars are given in Fanella and Mota (2019). For GFRP-reinforced concrete, these requirements are doubled to account for the lower modulus of elasticity of GFRP relative to steel. The requirement of an additional diagonal No. M16 bar at each corner accounts for the anticipated diagonal crack angle as well as the lower modulus of elasticity of GFRP relative to steel.

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COMMENTARY

CHAPTER 12—DIAPHRAGMS—NOT ADDRESSED

CHAPTER R12—DIAPHRAGMS—NOT ADDRESSED

Code provisions for diaphragms have not been addressed due to a lack of understanding of shear-friction modeling in GFRP-reinforced concrete. However, the use of this Code for design of GFRP-reinforced cast-in-place concrete diaphragms may be possible, provided such diaphragms have sufficient stiffness to transfer the forces among the lateral load-resisting elements of the structure without the use of collectors, as may occur in low-rise structures or structures assigned to SDC A. As the primary lateral load effect considered in this Code is wind, Fanella and Mota (2018), which provides guidance for steel-reinforced concrete diaphragms, may, with modification, be appropriate to the design of GFRP-reinforced concrete diaphragms. The diaphragm can be treated as a horizontal wall in which the in-plane design shear strength is calculated similarly to that for GFRP-reinforced concrete shear walls using the provisions of 11.5.4.4 from this Code instead of ACI 318-19 Section 12.5.3.3. In the calculation for V_n, b_w can be replaced by h and d replaced by a 1 m unit length. In-plane moment and axial force can be designed in accordance with 22.3 and 22.4 of this Code.

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CHAPTER 13—FOUNDATIONS

13.1—Scope

13.1.1 This chapter shall apply to the design of nonprestressed foundations, including shallow foundations (a) through (e), deep foundations (f) through (g), and retaining walls (j) and (k):

(a) Strip footings
(b) Isolated footings
(c) Combined footings
(d) Mat foundations
(e) Grade beams
(f) Pile caps
(g) Piles
(h) Intentionally left blank
(i) Intentionally left blank
(j) Cantilever retaining walls
(k) Counterfort and buttressed cantilever retaining walls

COMMENTARY

CHAPTER R13—FOUNDATIONS

R13.1—Scope

While requirements applicable to foundations are provided in this chapter, the majority of requirements used for foundation design are found in other chapters of the Code. These other chapters are referenced in Chapter 13. However, the applicability of the specific provisions within these other chapters may not be explicitly defined for foundations.

R13.1.1 Examples of foundation types covered by this chapter are illustrated in Fig. R13.1.1. Stepped and sloped footings are considered to be subsets of other footing types.

CODE | **COMMENTARY**

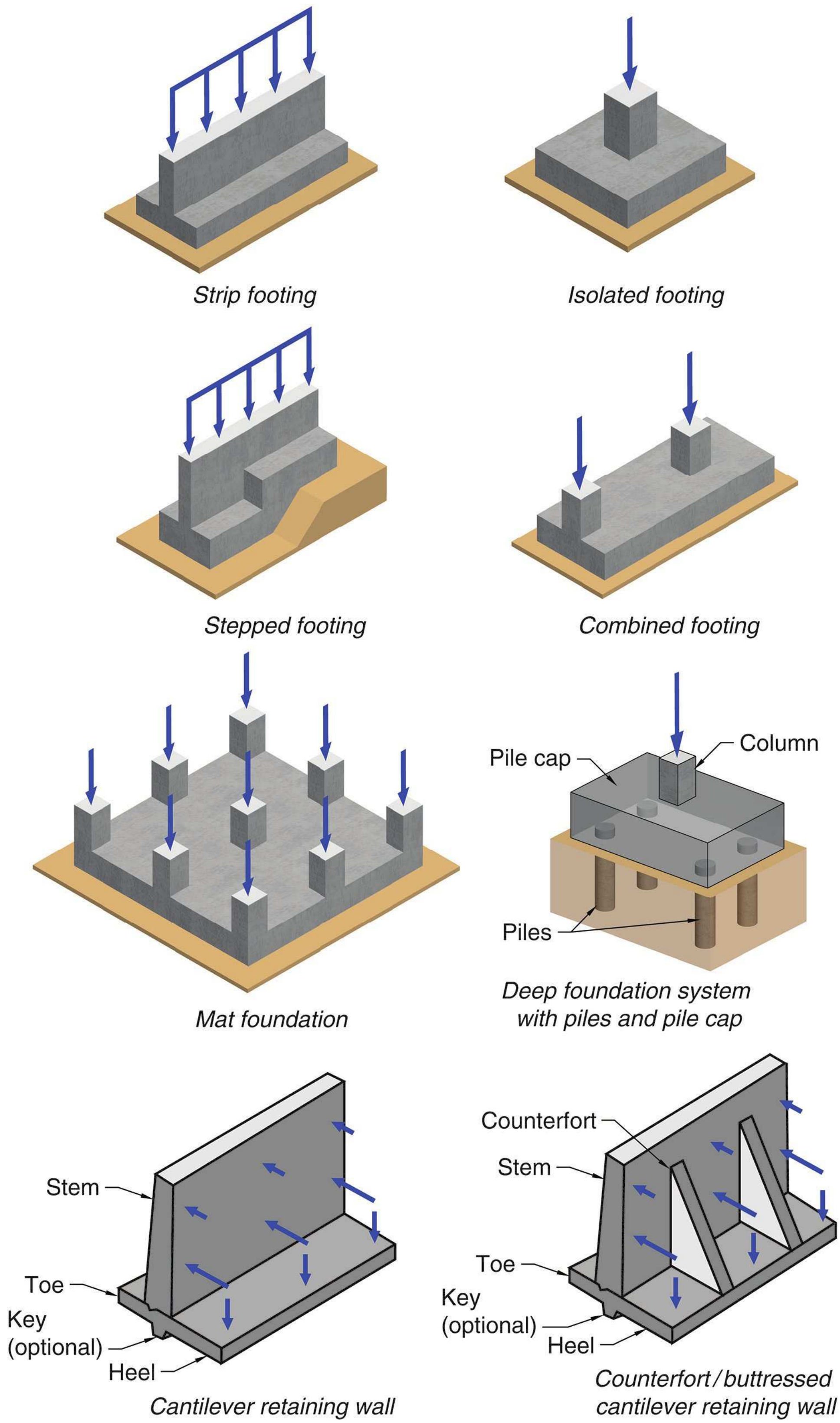

Fig. R13.1.1—Types of foundations.

CODE

13.1.2 The design of concrete piles embedded in the ground are excluded from this chapter, except for portions of piles in soil incapable of providing adequate lateral restraint to prevent buckling throughout their length. Portions of concrete piles in air or water may be designed using the provisions of this Chapter.

COMMENTARY

R13.1.2 GFRP-reinforced precast concrete piles have been successfully driven and tested (Benmokrane et al. 2018; Jimenez Vicariaa et al. 2014).

13.2—General

13.2.1 *Materials*

13.2.1.1 Design properties for concrete shall be selected to be in accordance with Chapter 19.

13.2.1.2 Design properties for GFRP reinforcement shall be selected to be in accordance with Chapter 20.

13.2.1.3 Materials, design, and detailing requirements for embedments in concrete shall be in accordance with 20.6.

13.2.2 *Connection to other members*

13.2.2.1 Design and detailing of cast-in-place and precast column, pedestal, and wall connections to foundations shall be in accordance with 16.3.

13.2.3 *Earthquake effects*—Out of scope

13.2.4 *Slabs-on-ground*

13.2.4.1 Slabs-on-ground that transmit vertical loads or lateral forces from other parts of the structure to the ground shall be designed and detailed in accordance with applicable provisions of this Code.

13.2.4.2 Intentionally left blank.

13.2.5 *Plain concrete*—Not applicable

13.2.6 *Design criteria*

13.2.6.1 Foundations shall be proportioned for bearing effects, stability against overturning, and sliding at the soil-foundation interface in accordance with the general building code.

R13.2—General

R13.2.4 *Slabs-on-ground*

Slabs-on-ground often act as a diaphragm to hold the building together at the ground level and minimize the effects of out-of-phase ground motion that may occur over the footprint of the building. In these cases, the slab-on-ground should be adequately reinforced and detailed. As required in Chapter 26, construction documents should clearly state that these slabs-on-ground are structural members so as to prohibit sawcutting of such slabs.

R13.2.6 *Design criteria*

R13.2.6.1 Permissible soil pressures or permissible pile capacities are determined by principles of soil mechanics and in accordance with the general building code. The size of the base area of a footing on soil or the number and arrangement of deep foundation members are established by using allowable geotechnical strength and service-level load combinations or by using nominal geotechnical strength with resistance factor and factored load combinations.

Only the calculated end moments at the base of a column or pedestal require transfer to the footing. The minimum moment requirement for slenderness considerations given in 6.6.4.5 need not be considered for transfer of forces and moments to footings.

CODE

=13.2.6.2 For one-way shallow foundations, two-way isolated footings, or two-way combined footings and mat foundations, it is permissible to neglect the size effect factor specified in 22.5 for one-way shear strength and 22.6 for two-way shear strength.

=13.2.6.3 Foundation members shall be designed to resist factored loads and corresponding induced reactions.

=13.2.6.4 Foundation systems shall be permitted to be designed by any procedure satisfying equilibrium and geometric compatibility.

13.2.6.5 Intentionally left blank.

=13.2.6.6 External moment on any section of a strip footing, isolated footing, or pile cap shall be calculated by passing a vertical plane through the member and calculating the moment of the forces acting over the entire area of member on one side of that vertical plane.

13.2.7 *Critical sections for shallow foundations and pile caps*

=13.2.7.1 M_u at the supported member shall be permitted to be calculated at the critical section defined in accordance with Table 13.2.7.1.

COMMENTARY

=R13.2.6.3 To design a footing or pile cap for strength, the induced reactions due to factored loads applied to the foundation should be determined. For a single concentrically-loaded spread footing, the soil pressure due to factored loading is calculated as the factored load divided by the base area of the footing. For the case of footings or mats with eccentric loading, applied factored loads may be used to determine soil pressures. For pile caps or mats supported by deep foundations, applied factored loads may be used to determine member reactions. However, the resulting pressures or reactions may be incompatible with the geotechnical design resulting in unacceptable subgrade reactions or instability (Rogowsky and Wight 2010). In such cases, the design should be adjusted in coordination with the geotechnical engineer.

Only the calculated end moments at the base of a column or pedestal require transfer to the footing. The minimum moment requirements for slenderness considerations given in 6.6.4.5 need not be considered for transfer of forces and moments to footings.

=R13.2.6.4 Foundation design is permitted to be based directly on fundamental principles of structural mechanics, provided it can be demonstrated that all strength and serviceability criteria are satisfied. Design of the foundation may be achieved through the use of classic solutions based on a linearly elastic continuum, numerical solutions based on discrete elements, or yield-line analyses. In all cases, analyses and evaluation of the stress conditions at points of load application or pile reactions in relation to shear and torsion, as well as flexure, should be included.

R13.2.7 *Critical sections for shallow foundations and pile caps*

CODE

Table 13.2.7.1—Location of critical section for M_u

Supported member	Location of critical section
Column or pedestal	Face of column or pedestal
Column with steel base plate	Halfway between face of column and edge of steel base plate
Concrete wall	Face of wall
Masonry wall	Halfway between center and face of masonry wall

=**13.2.7.2** The location of critical section for factored shear in accordance with 7.4.3 and 8.4.3 for one-way shear or 8.4.4.1 for two-way shear shall be measured from the location of the critical section for M_u in 13.2.7.1.

=**13.2.7.3** Circular or regular polygon-shaped concrete columns or pedestals shall be permitted to be treated as square members of equivalent area when locating critical sections for moment, shear, and development of reinforcement.

13.2.8 *Development of reinforcement in shallow foundations and pile caps*

=**13.2.8.1** Development of reinforcement shall be in accordance with Chapter 25.

COMMENTARY

=**R13.2.7.2** The shear strength of a footing is determined for the more severe condition of 8.5.3.1.1 and 8.5.3.1.2. The critical section for shear is measured from the face of the supported member (column, pedestal, or wall), except for masonry walls and members supported on steel base plates.

Calculation of shear requires that the soil reaction be obtained from factored loads, and the design strength be in accordance with Chapter 22.

Where necessary, shear around individual piles may be investigated in accordance with 8.5.3.1.2. If shear perimeters overlap, the modified critical perimeter b_o should be taken as that portion of the smallest envelope of individual shear perimeters that will actually resist the critical shear for the group under consideration. One such situation is illustrated in Fig. R13.2.7.2.

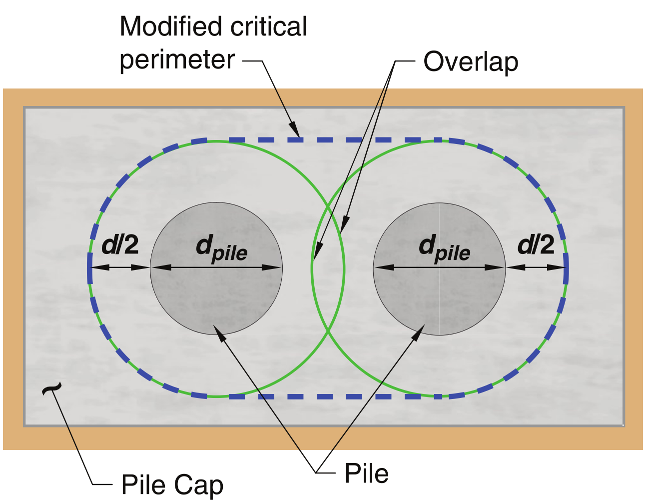

Fig. R13.2.7.2—Modified critical perimeter for shear with overlapping critical perimeters.

CODE

=**13.2.8.2** Calculated tensile or compressive force in reinforcement at each section shall be developed on each side of that section.

=**13.2.8.3** Critical sections for development of reinforcement shall be assumed at the same locations as given in 13.2.7.1 for maximum factored moment and at all other vertical planes where changes of section or reinforcement occur.

=**13.2.8.4** Adequate anchorage shall be provided for tension reinforcement where reinforcement stress is not directly proportional to moment, such as in sloped, stepped, or tapered foundations; or where tension reinforcement is not parallel to the compression face.

13.3—Shallow foundations

13.3.1 *General*

=**13.3.1.1** Minimum base area of foundation shall be proportioned to not exceed the permissible bearing pressure when subjected to forces and moments applied to the foundation. Permissible bearing pressures shall be determined through principles of soil or rock mechanics in accordance with the general building code, or other requirements as determined by the building official.

=**13.3.1.2** Overall depth of foundation shall be selected such that the effective depth of bottom reinforcement is at least 6 in.

=**13.3.1.3** In sloped, stepped, or tapered foundations, depth and location of steps or angle of slope shall be such that design requirements are satisfied at every section.

13.3.2 *One-way shallow foundations*

=**13.3.2.1** The design and detailing of one-way shallow foundations, including strip footings, combined footings, and grade beams, shall be in accordance with this section and the applicable provisions of Chapter 7 and Chapter 9.

=**13.3.2.2** Reinforcement shall be distributed uniformly across entire width of one-way footings.

13.3.3 *Two-way isolated footings*

=**13.3.3.1** The design and detailing of two-way isolated footings shall be in accordance with this section and the applicable provisions of Chapters 7 and 8.

=**13.3.3.2** In square two-way footings, reinforcement shall be distributed uniformly across entire width of footing in both directions.

=**13.3.3.3** In rectangular footings, reinforcement shall be distributed in accordance with (a) and (b):

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R13.3—Shallow foundations

R13.3.1 *General*

=**R13.3.1.1** General discussion on the sizing of shallow foundations is provided in R13.2.6.1.

=**R13.3.1.3** Anchorage of reinforcement in sloped, stepped, or tapered foundations is addressed in 13.2.8.4.

R13.3.3 *Two-way isolated footings*

=**R13.3.3.3** To minimize potential construction errors in placing bars, a common practice is to increase the amount of

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(a) Reinforcement in the long direction shall be distributed uniformly across entire width of footing.
(b) For reinforcement in the short direction, a portion of the total reinforcement, $\gamma_s A_f$, shall be distributed uniformly over a band width equal to the length of short side of footing, centered on centerline of column or pedestal. Remainder of reinforcement required in the short direction, $(1 - \gamma_s)A_f$, shall be distributed uniformly outside the center band width of footing, where γ_s is calculated by:

$$\gamma_s = \frac{2}{(\beta + 1)} \qquad (13.3.3.3)$$

where β is the ratio of long to short side of footing.

13.3.4 *Two-way combined footings and mat foundations*

=**13.3.4.1** The design and detailing of combined footings and mat foundations shall be in accordance with this section and the applicable provisions of Chapter 8.

=**13.3.4.2** The direct design method shall not be used to design combined footings and mat foundations.

=**13.3.4.3** Distribution of bearing pressure under combined footings and mat foundations shall be consistent with properties of the soil or rock and the structure, and with established principles of soil or rock mechanics.

13.3.4.4 Minimum reinforcement in mat foundations shall be in accordance with 8.6.1.1.

13.3.5 *Walls as grade beams*

=**13.3.5.1** The design of walls as grade beams shall be in accordance with the applicable provisions of Chapter 9.

13.3.5.2 Intentionally left blank.

=**13.3.5.3** Grade beam walls shall satisfy the minimum reinforcement requirements of 11.6.

13.3.6 *Wall components of cantilever retaining walls*

=**13.3.6.1** The stem of a cantilever retaining wall shall be designed as a one-way slab in accordance with the applicable provisions of Chapter 7.

=**13.3.6.2** The stem of a counterfort or buttressed cantilever retaining wall shall be designed as a two-way slab in accordance with the applicable provisions of Chapter 8.

COMMENTARY

reinforcement in the short direction by **$2\beta/(\beta + 1)$** and space it uniformly along the long dimension of the footing (CRSI 1984; Fling 1987).

R13.3.4 *Two-way combined footings and mat foundations*

R13.3.4.1 Kakusha et al. (2018) provides an example of the design of a GFRP-reinforced concrete mat foundation.

=**R13.3.4.2** The direct design method is a method used for the design of two-way slabs. Refer to R6.2.4.1.

=**R13.3.4.3** Design methods using factored loads and strength reduction factors ϕ can be applied to combined footings or mat foundations, regardless of the bearing pressure distribution.

=**R13.3.4.4** To improve crack control due to thermal gradients and to intercept potential punching shear cracks with tension reinforcement, the licensed design professional should consider specifying continuous reinforcement in each direction near both faces of mat foundations.

R13.3.6 *Wall components of cantilever retaining walls*

=**R13.3.6.2** Counterfort or buttressed cantilever retaining walls tend to behave more in two-way action than in one-way action; therefore, additional care should be given to crack control in both directions.

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=**13.3.6.3** For walls of uniform thickness, the critical section for shear and flexure shall be at the interface between the stem and the footing. For walls with a tapered or varied thickness, shear and moment shall be investigated throughout the height of the wall.

COMMENTARY

=**R13.3.6.3** In general, the joint between the wall stem and the footing will be opening under lateral loads; therefore, the critical section should be at the face of the joint. If hooks are required to develop the wall flexural reinforcement, hooks should be located near the bottom of the footing with the free end of the bars oriented toward the opposite face of the wall (Nilsson and Losberg 1976).

13.4—Deep foundations

13.4.1 *General*

=**13.4.1.1** Number and arrangement of piles shall be determined such that forces and moments applied to the piles do not exceed the permissible deep foundation strength. Permissible deep foundation strength shall be determined through principles of soil or rock mechanics in accordance with the general building code, or other requirements as determined by the building official.

13.4.1.2 Intentionally left blank.

13.4.2 *Allowable strength design*—Out of scope

13.4.3 *Strength design*—Out of scope

13.4.4 *Cast-in-place deep foundations*

13.4.4.1 Cast-in-place deep foundations that are subject to uplift or where M_u is greater than $0.4M_{cr}$ shall be reinforced.

=**13.4.4.2** Portions of deep foundation members in air, water, or soils not capable of providing adequate restraint throughout the member length to prevent lateral buckling shall be designed as columns in accordance with the applicable provisions of Chapter 10.

13.4.5 *Precast concrete piles*

13.4.5.1 Precast concrete piles supporting buildings assigned to SDC A or B shall satisfy the requirements of Chapter 10.

13.4.5.2 Intentionally left blank.

13.4.5.3 Intentionally left blank.

13.4.5.4 Intentionally left blank.

13.4.5.5 Intentionally left blank.

13.4.6 *Pile caps*

=**13.4.6.1** Overall depth of pile cap shall be selected such that the effective depth of bottom reinforcement is at least 12 in.

R13.4—Deep foundations

R13.4.1 *General*

R13.4.1.1 General discussion on selecting the number and arrangement of piles is provided in R13.2.6.1.

R13.4.6 *Pile caps*

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=**13.4.6.2** Factored moments and shears shall be permitted to be calculated with the reaction from any pile assumed to be concentrated at the centroid of the pile section.

13.4.6.3 The pile cap shall be designed such that (a) is satisfied for one-way foundations and (a) and (b) are satisfied for two-way foundations.

(a) $\phi V_n \geq V_u$, where V_n shall be calculated in accordance with 22.5 for one-way shear, V_u shall be calculated in accordance with 13.4.6.5, and ϕ shall be in accordance with 21.2

(b) $\phi v_n \geq v_u$, where v_n shall be calculated in accordance with 22.6 for two-way shear, v_u shall be calculated in accordance with 13.4.6.5, and ϕ shall be in accordance with 21.2

13.4.6.4 Intentionally left blank.

=**13.4.6.5** Calculation of factored shear on any section through a pile cap shall be in accordance with (a) through (c):

(a) 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.

(b) 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.

(c) For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the section shall be based on a linear interpolation between full value at $d_{pile}/2$ outside the section and zero value at $d_{pile}/2$ inside the section.

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R13.4.6.5 If piles are located inside the critical sections d or $d/2$ from face of column, for one-way or two-way shear, respectively, an upper limit on the shear strength at a section adjacent to the face of the column should be considered.

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CHAPTER 14—PLAIN CONCRETE—NOT APPLICABLE

COMMENTARY

CHAPTER R14—PLAIN CONCRETE—NOT APPLICABLE

Design of plain concrete members is governed by the requirements of ACI 318.

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CHAPTER 15—BEAM-COLUMN AND SLAB-COLUMN JOINTS

15.1—Scope

15.1.1 This chapter shall apply to the design and detailing of cast-in-place beam-column and slab-column joints.

15.2—General

15.2.1 Beam-column and slab-column joints shall satisfy 15.3 for transfer of column axial force through the floor system.

15.2.2 If gravity load, wind, earthquake, or other lateral forces cause transfer of moment at beam-column or slab-column joints, the shear resulting from moment transfer shall be considered in the design of the joint.

15.2.3 Beam-column and slab-column joints that transfer moment to columns shall satisfy the detailing provisions in 15.4.

15.2.4 A beam-column joint shall be considered to be restrained if the joint is laterally supported on four sides by beams of approximately equal depth.

15.2.5 A slab-column joint shall be considered restrained if the joint is laterally supported on four sides by the slab.

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CHAPTER R15—BEAM-COLUMN AND SLAB-COLUMN JOINTS

R15.2—General

Tests (Hanson and Connor 1967) have shown that the joint region of a steel-reinforced concrete beam-to-column connection in the interior of a building does not require shear reinforcement if the joint is laterally supported on four sides by beams of approximately equal depth. However, joints that are not restrained in this manner, such as at the exterior of a building, require shear reinforcement to prevent deterioration due to shear cracking (ACI 352R). These joints may also require transverse reinforcement to prevent buckling of longitudinal column reinforcement. While the development of a plastic hinge in a steel–reinforced concrete beam, due to yielding of the reinforcement, limits the damage penetrating into the joint area, the linear stress-strain behavior of GFRP reinforcement may result in an increase in the joint shear stress. Research on GFRP-reinforced concrete beam-column joints supported by lateral beams has shown that these joints exhibit more concrete damage in the joint area under seismic load tests at large (greater than 5%) drift ratios than steel-reinforced concrete joints with equal joint shear stress. Ghomi and El-Salakawy (2016) tested a beam-column joint with and without lateral beams and found little difference in measured lateral stiffness at the 0.8% drift level. Tests by Sleiman and Polak (2020) on GFRP-reinforced concrete knee joints under monotonic closing loads show that confinement reinforcement in the joint helps to control the formation and widening of shear cracks. Because there is no experimental evidence to show that shear reinforcement is not required in laterally supported beam-column joints, all GFRP-reinforced concrete beam-column joints should contain shear reinforcement to prevent deterioration due to shear cracking.

15.3—Transfer of column axial force through the floor system

15.3.1 If f_c' of a column is greater than 1.4 times that of the floor system, transmission of axial force through the floor system shall be in accordance with (a), (b), or (c):

(a) Concrete of compressive strength specified for the column shall be placed in the floor at the column location. Column concrete shall extend outward at least 600 mm into the floor slab from face of column for the full depth of the slab and be integrated with floor concrete.

(b) Design strength of a column through a floor system shall be calculated using the lower value of concrete strength with vertical dowels and spirals as required to achieve adequate strength.

(c) For beam-column and slab-column joints that are restrained in accordance with 15.2.4 or 15.2.5, respec-

R15.3—Transfer of column axial force through the floor system

The requirements of this section consider the effect of floor concrete strength on column axial strength (Bianchini et al. 1960). Where the column concrete strength does not exceed the floor concrete strength by more than 40%, no special provisions are required. For higher column concrete strengths, methods in 15.3.1(a) or 15.3.1(b) can be used for corner or edge columns. Methods in 15.3.1(a), (b), or (c) can be used for interior columns with adequate restraint on all four sides.

The requirements of 15.3.1(a) locate the interface between column and floor concrete at least 600 mm into the floor. Application of the concrete placement procedure described in 15.3.1(a) requires the placing of two different concrete mixtures in the floor system. The lower-strength mixture

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tively, it shall be permitted to calculate the design 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, where the value of column concrete strength shall not exceed 2.5 times the floor concrete strength.

15.4—Detailing of joints

15.4.1 Beam-column and slab-column joints shall satisfy the provisions for GFRP transverse reinforcement of 15.4.2.

15.4.2 The area of all legs of GFRP transverse reinforcement in each principal direction of beam-column and slab-column joints shall be at least the greater of (a) and (b):

(a) $0.062\sqrt{f_c'}\dfrac{bs}{f_{ft}}$

(b) $0.35\dfrac{bs}{f_{ft}}$

where $\boldsymbol{b_w}$ is the dimension of the column section perpendicular to the direction under consideration.

15.4.2.1 At beam-column and slab-column joints, an area of GFRP transverse reinforcement calculated in accordance with 15.4.2 shall be distributed within the column height not less than the deepest beam or slab element framing into the column.

15.4.2.2 For beam-column joints, the spacing of the GFRP transverse reinforcement $\boldsymbol{s}$ shall not exceed one-half the depth of the shallowest beam.

15.4.3 If GFRP longitudinal beam or column reinforcement is spliced or terminated in a joint, closed GFRP transverse reinforcement in accordance with 10.7.6 shall be provided in the joint.

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should be placed while the higher-strength concrete is still plastic and should be adequately vibrated to ensure the concretes are well integrated. It is important that the higher-strength concrete in the floor region around the column be placed before the lower-strength concrete in the remainder of the floor to prevent accidental placing of the lower-strength concrete in the column area. As required in Chapter 26, it is the responsibility of the licensed design professional to indicate on the construction documents where the higher- and lower-strength concretes are to be placed.

Research (Ospina and Alexander 1998) with steel-reinforced concrete has shown that heavily loaded slabs do not provide as much confinement as lightly loaded slabs when ratios of column concrete strength to slab concrete strength exceed approximately 2.5. Consequently, a limit is placed on the concrete strength ratio assumed in design in 15.3.1(c).

R15.4—Detailing of joints

R15.4.3 GFRP reinforcement is required such that the flexural strength can be developed and maintained under repeated loadings. Tests under low-frequency cyclic loadings with various amplitudes (Ghomi and El-Salakawy 2016, 2018; Hasaballa and El-Salakawy 2016) have shown that under repeated loadings, adequately reinforced GFRP-reinforced concrete exterior joints without lateral beams can support up to $\mathbf{0.83}\sqrt{\boldsymbol{f_c'}}$ joint shear stress; exterior and interior joints with lateral beams can support up to $\mathbf{0.99}\sqrt{\boldsymbol{f_c'}}$ and $\mathbf{1.8}\sqrt{\boldsymbol{f_c'}}$, respectively (Ghomi and El-Salakawy 2016, 2018). Although the elastic nature of GFRP reinforcement

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leads to smaller residual deformations than in steel-reinforced concrete joints, the lower stiffness of GFRP causes larger joint deformations.

15.4.4 Development of GFRP longitudinal reinforcement terminating in the joint shall be in accordance with 25.4.

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CHAPTER 16—CONNECTIONS BETWEEN MEMBERS

16.1—Scope

16.1.1 This chapter shall apply to the design of the following:

(a) Connections at the intersection of cast-in-place GFRP-reinforced concrete members and cast-in-place GFRP-reinforced concrete foundations

(b) Connections between cast-in-place GFRP-reinforced members and steel-reinforced concrete foundations where the interface reinforcement is GFRP

(c) Connections between cast-in-place steel-reinforced concrete members and GFRP-reinforced concrete foundations where the interface reinforcement is GFRP

(d) Horizontal shear transfer in composite flexural members

In cases of connections between steel-reinforced and GFRP-reinforced concrete elements and foundations in which the interface reinforcement is steel, the requirements of ACI 318 for connections to foundations shall apply.

16.2—Connections of precast members—Out of scope

16.3—Connections to foundations

16.3.1 *General*

16.3.1.1 Factored forces and moments at base of columns, walls, or pedestals shall be transferred to supporting foundations by bearing on concrete and by GFRP reinforcement or dowels.

16.3.1.2 GFRP reinforcement or dowels between a supported member and foundation shall be designed to transfer (a) and (b):

(a) Compressive forces that exceed the lesser of the concrete bearing strengths of either the supported member or the foundation, calculated in accordance with 22.8

(b) Any calculated tensile force across the interface

16.3.1.3 Intentionally left blank.

16.3.2 *Required strength*

=**16.3.2.1** Factored forces and moments transferred to foundations shall be calculated in accordance with the

COMMENTARY

CHAPTER R16—CONNECTIONS BETWEEN MEMBERS

R16.2—Connections of precast members—Out of scope

This Code does not cover the design of connections between GFRP-reinforced concrete precast elements due to a lack of published research on the dowel action of GFRP reinforcing bars and GFRP connectors. Where transfer of forces by means of grouted joints, shear keys, bearing, steel anchors, mechanical connectors, steel reinforcement, or a combination of these are permissible, the requirements of ACI 318-19 Section 16.2 may be used to design the connections.

R16.3—Connections to foundations

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factored load combinations in Chapter 5 and analysis procedures in Chapter 6.

16.3.3 *Design strength*

16.3.3.1 Design strengths of connections between columns, walls, or pedestals and foundations shall satisfy Eq. (16.3.3.1) for each applicable load combination.

$$\phi S_n \geq U \tag{16.3.3.1}$$

where S_n is the nominal flexural, shear, axial, torsional, or bearing strength of the connection.

=**16.3.3.2** ϕ shall be determined in accordance with 21.2.

=**16.3.3.3** Combined moment and axial strength of connections shall be calculated in accordance with 22.4.

=**16.3.3.4** At the contact surface between a supported member and foundation, or between a supported member or foundation and an intermediate bearing element, nominal bearing strength B_n shall be calculated in accordance with 22.8 for concrete surfaces. B_n shall be the lesser of the nominal concrete bearing strengths for the supported member or foundation surface, and shall not exceed the strength of intermediate bearing elements, if present.

16.3.3.5 At the contact surface between supported member and foundation, V_n shall be provided by shear keys or other appropriate means. Contribution of GFRP reinforcement to the nominal shear strength V_n at the contact surface between supported member and foundation shall be verified by test.

16.3.3.6 Intentionally left blank.

16.3.3.7 Intentionally left blank.

16.3.4 *Minimum GFRP reinforcement for connections between cast-in-place members and foundation*

16.3.4.1 For connections between a cast-in-place column or pedestal and foundation, A_f crossing the interface shall be at least $\mathbf{0.01}A_g$, where A_g is the gross area of the supported member.

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R16.3.3 *Design strength*

=**R16.3.3.4** In the common case of a column bearing on a footing, where the area of the footing is larger than the area of the column, the bearing strength should be checked at the base of the column and the top of the footing. In the absence of dowels or column reinforcement that continue into the foundation, the strength of the lower part of the column should be checked using the strength of the concrete alone.

R16.3.3.5 The shear-friction provisions in 22.9 of ACI 318 have not been verified for GFRP-reinforced concrete. As an alternative to using shear-friction across a shear plane, shear keys may be used, provided that the GFRP reinforcement crossing the joint satisfies 16.3.4.1.

R16.3.4 *Minimum GFRP reinforcement for connections between cast-in-place members and foundation*

The Code requires a minimum amount of reinforcement between all supported and supporting members. This reinforcement is required to provide a degree of structural integrity during the construction stage and during the life of the structure.

R16.3.4.1 The minimum area of GFRP reinforcement at the base of a column may be provided by extending the longitudinal bars and anchoring them into the footing or by providing properly anchored dowels. The value of $\mathbf{0.01}A_g$ is twice the amount required by ACI 318 for steel reinforcement.

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16.3.4.2 For connections between a cast-in-place wall and foundation, area of GFRP vertical reinforcement crossing the interface shall satisfy 11.6.1.

16.3.5 *Details for connections between cast-in-place members and foundation*

16.3.5.1 At the base of a cast-in-place column, pedestal, or wall, GFRP reinforcement required to satisfy 16.3.3 and 16.3.4 shall be provided either by extending longitudinal bars into supporting foundation or by dowels.

16.3.5.2 Where continuity is required, GFRP reinforcement or dowels shall satisfy 10.7.5 for splices. Lap splice lengths shall not be less than $60d_b$.

=**16.3.5.3** If a pinned or rocker connection is used at the base of a cast-in-place column or pedestal, the connection to foundation shall satisfy 16.3.3.

16.3.5.4 Intentionally left blank.

16.3.6 *Details for connections between precast members and foundation*—Out of scope

16.4—Horizontal shear transfer in composite concrete flexural members

16.4.1 *General*

=**16.4.1.1** In a composite concrete flexural member, full transfer of horizontal shear forces shall be provided at contact surfaces of interconnected elements.

16.4.1.2 Where tension exists across any contact surface between interconnected concrete elements, horizontal shear transfer by contact is not covered by this Code.

=**16.4.1.3** Surface preparation assumed for design shall be specified in the construction documents.

16.4.2 *Required strength*

=**16.4.2.1** Factored forces transferred along the contact surface in composite concrete flexural members shall be calculated in accordance with the factored load combinations in Chapter 5.

=**16.4.2.2** Required strength shall be calculated in accordance with the analysis procedures in Chapter 6.

16.4.3 *Design strength*

COMMENTARY

R16.3.5 *Details for connections between cast-in-place members and foundation*

R16.3.5.2 Tests (Naqvi and El-Salakawy 2017) have shown that a lap splice length of 60 times the diameter of the longitudinal GFRP bars is adequate in transferring the full bond strength along the splice length in GFRP-reinforced concrete columns with dowels and longitudinal bars of the same size under tension-compression reversal loading.

R16.4—Horizontal shear transfer in composite concrete flexural members

R16.4.1 *General*

R16.4.1.1 Full transfer of horizontal shear forces between segments of composite members can be provided by horizontal shear strength at contact surfaces through interface shear.

=**R16.4.1.3** Section 26.5.6 requires the licensed design professional to specify the surface preparation in the construction documents.

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16.4.3.1 Design strength for horizontal shear transfer shall satisfy Eq. (16.4.3.1) at all locations along the contact surface in a composite concrete flexural member:

$$\phi V_{nh} \geq V_u \qquad (16.4.3.1)$$

where nominal horizontal shear strength V_{nh} is calculated in accordance with 16.4.4.

=**16.4.3.2** ϕ shall be determined in accordance with 21.2.

16.4.4 *Nominal horizontal shear strength*

16.4.4.1 Intentionally left blank.

16.4.4.2 V_{nh} shall be $80b_vd$, where b_v is the width of the contact surface and d is in accordance with 16.4.4.3. Concrete shall be placed against hardened concrete intentionally roughened to a full amplitude of approximately 6 mm.

16.4.4.3 In calculations for nominal horizontal shear strength, d shall be the distance from extreme compression fiber for the entire composite section to the centroid of longitudinal tension reinforcement.

16.4.4.4 Intentionally left blank.

16.4.5 *Alternative method for calculating design horizontal shear strength*—Out of scope

16.4.6 *Minimum reinforcement for horizontal shear transfer*—Out of scope

16.4.7 *Reinforcement detailing for horizontal shear transfer*—Out of scope

16.5—Brackets and corbels—Out of scope

COMMENTARY

R16.4.4 *Nominal horizontal shear strength*

R16.4.4.2 The permitted horizontal shear strengths and the requirement of 6 mm amplitude for intentional roughness are based on tests of steel-reinforced concrete composite members discussed in Kaar et al. (1960), Saemann and Washa (1964), and Hanson (1960).

R16.5 Brackets and corbels—Out of scope

Brackets and corbels are short cantilevers that tend to act as simple trusses or deep beams. This Code does not cover the design of brackets and corbels due to a lack of published research on GFRP-reinforced deep beams and a lack of knowledge on using strut-and-tie modeling for GFRP-reinforced concrete.

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CHAPTER 17—ANCHORING TO CONCRETE—NOT ADDRESSED

COMMENTARY

CHAPTER R17—ANCHORING TO CONCRETE—NOT ADDRESSED

Anchoring to concrete is not covered in this Code due to a lack of ANSI-approved material specifications for GFRP headed studs, headed bolts, hooked bolts, and anchors.

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CHAPTER 18—EARTHQUAKE-RESISTANT STRUCTURES—NOT ADDRESSED

COMMENTARY

CHAPTER R18—EARTHQUAKE-RESISTANT STRUCTUES—NOT ADDRESSED

This Code does not cover the design of GFRP-reinforced concrete structures in Seismic Design Categories D through F, nor the design of GFRP-reinforced concrete members in the seismic-force-resisting system in Seismic Design Categories B and C due to a lack of proven methodology to provide adequate ductility and energy absorption during seismic events.

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CHAPTER 19—CONCRETE: DESIGN AND DURABILITY REQUIREMENTS

19.1—Scope

19.1.1 This chapter shall apply to concrete, including:
(a) Properties to be used for design
(b) Durability requirements

19.1.2 Intentionally left blank.

19.2—Concrete design properties

19.2.1 *Specified compressive strength*

19.2.1.1 The value of f_c' shall be specified in construction documents and shall be in accordance with (a) through (c):
(a) Minimum f_c' shall be 21 MPa
(b) Durability requirements in Table 19.3.2.1
(c) Structural strength requirements
(d) Intentionally left blank

19.2.1.2 The specified compressive strength shall be used for proportioning of concrete mixtures in 26.4.3 and for testing and acceptance of concrete in 26.12.3.

19.2.1.3 Unless otherwise specified, f_c' shall be based on 28-day tests. If other than 28 days, test age for f_c' shall be indicated in the construction documents.

19.2.2 *Modulus of elasticity*

19.2.2.1 Modulus of elasticity, E_c, for concrete shall be permitted to be calculated as (a) or (b):
(a) For values of w_c between 1440 and 2560 kg/m^3

$$E_c = w_c^{1.5} 0.043\sqrt{f_c'} \quad \text{(in MPa)} \qquad (19.2.2.1a)$$

(b) For normalweight concrete

$$E_c = 4700\sqrt{f_c'} \quad \text{(in MPa)} \qquad (19.2.2.1b)$$

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CHAPTER R19—CONCRETE: DESIGN AND DURABILITY REQUIREMENTS

R19.2—Concrete design properties

R19.2.1 *Specified compressive strength*

Requirements for concrete mixtures are based on the philosophy that concrete should provide both adequate strength and durability. The Code defines a minimum value of f_c' for structural concrete equal to 21 MPa; this limit is imposed primarily because of a lack of experimental data on the behavior of GFRP-reinforced concrete members made with a concrete strength less than 21 MPa. There is no limit on the maximum value of f_c' except as required by specific Code provisions.

Concrete mixtures proportioned in accordance with 26.4.3 should achieve an average compressive strength that exceeds the value of f_c' used in the structural design calculations. The amount by which the average strength of concrete exceeds f_c' is based on statistical concepts. When concrete is designed to achieve a strength level greater than f_c', it ensures that the concrete strength tests will have a high probability of meeting the strength acceptance criteria in 26.12.3. The durability requirements prescribed in Table 19.3.2.1 are to be satisfied in addition to meeting the minimum f_c' of 19.2.1. Under some circumstances, durability requirements may dictate a higher f_c' than that required for structural purposes.

R19.2.2 *Modulus of elasticity*

R19.2.2.1 Equations in 19.2.2.1 provide an estimate of E_c for general design use. Studies leading to the expression for E_c of concrete are summarized in Pauw (1960), where E_c is defined as the slope of the line drawn from a stress of zero to 45% of the compressive strength using the stress-strain curve of the concrete. This definition is slightly different than the definition in ASTM C469. ASTM C469 defines E_c using 40% of the compressive strength.

The modulus of elasticity is sensitive to a number of variables including aggregate type, concrete constituents, mixture proportions, bond between paste and aggregate, and the age of the concrete. This sensitivity, coupled with the inherent variability in the properties of the constituent materials and quality control exercised during construction, can result in differences between measured and calculated values for deflection, drift, periods of vibration, and other quantities that depend on E_c. Refer to ACI 435R for more information on the use of E_c, especially when used in deflection calculations.

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Modulus of elasticity determined by calculation using the Code equations has been shown to be appropriate for most applications based on many years of use. For some applications, however, these equations may not provide sufficiently accurate estimates of actual values. Larger differences between measured and calculated values of E_c have been observed for high-strength concrete (f_c' > 55 MPa), lightweight concrete, and for mixtures with low coarse aggregate volume, as can occur with self-consolidating concrete. Refer to ACI 363R, ACI 213R, and ACI 237R for more information.

19.2.2.2 It shall be permitted to specify E_c based upon testing of concrete mixtures to be used in the Work in accordance with (a) through (c):

(a) Specified E_c shall be used for proportioning concrete mixtures in accordance with 26.4.3.

(b) Testing to verify that the specified E_c has been achieved shall be conducted, and results shall be provided with the mixture submittal.

(c) Test age of measurement of E_c shall be 28 days or as indicated in the construction documents.

R19.2.2.2 For any project, E_c used for design may be specified and verified by testing. Design conditions that are sensitive to the value of E_c may warrant testing. Examples include applications where deflections are critical, tall buildings or similar structures for which axial deformation or lateral stiffness impact performance, and where estimation of E_c is important to acceptable vibration or seismic performance.

In cases where an unintended change of stiffness may have an adverse effect on the design, such as for some seismic applications, the licensed design professional may choose to specify a range of acceptable values of E_c at a specified test age. If a range of values of E_c is specified, details of a testing program and acceptance criteria should be provided in the construction documents.

The licensed design professional may choose to specify laboratory testing of E_c at multiple ages. It should be recognized that the development of E_c over time cannot be controlled with precision.

19.2.3 *Modulus of rupture*

19.2.3.1 Modulus of rupture, f_r, for concrete shall be calculated by:

$$f_r = 0.62\sqrt{f_c'} \tag{19.2.3.1}$$

19.2.4 *Lightweight concrete*—Out of scope

19.3—Concrete durability requirements

R19.3—Concrete durability requirements

The Code addresses concrete durability on the basis of exposure categories and exposure classes as defined in Table 19.3.1.1. The licensed design professional assigns members in the structure to the appropriate exposure category and class. The assigned exposure classes, which are based on the severity of exposure, are used to establish the appropriate concrete properties from Table 19.3.2.1 to include in the construction documents.

The Code does not include provisions for especially severe exposures, such as acids or high temperatures.

19.3.1 *Exposure categories and classes*

R19.3.1 *Exposure categories and classes*

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19.3.1.1 The licensed design professional shall assign exposure classes in accordance with the severity of the anticipated exposure of members for each exposure category in Table 19.3.1.1.

Table 19.3.1.1—Exposure categories and classes

Category	Class	Condition	
Freezing and thawing (F)	F0	Concrete not exposed to freezing-and-thawing cycles	
	F1	Concrete exposed to freezing-and-thawing cycles with limited exposure to water	
	F2	Concrete exposed to freezing-and-thawing cycles with frequent exposure to water	
	F3	Concrete exposed to freezing-and-thawing cycles with frequent exposure to water and exposure to deicing chemicals	
Sulfate (S)		Water-soluble sulfate (SO_4^{2-}) in soil, percent by mass*	Dissolved sulfate (SO_4^{2-}) in water, ppm†
	S0	$SO_4^{2-} < 0.10$	$SO_4^{2-} < 150$
	S1	$0.10 \leq SO_4^{2-} < 0.20$	$150 \leq SO_4^{2-} < 1500$ or seawater
	S2	$0.20 \leq SO_4^{2-} \leq 2.00$	$1500 \leq SO_4^{2-} \leq 10,000$
	S3	$SO_4^{2-} > 2.00$	$SO_4^{2-} > 10,000$
In contact with water (W)	W0	Concrete dry in service Concrete in contact with water and low permeability is not required	
	W1	Concrete in contact with water where low permeability is not required	
	W2	Concrete in contact with water where low permeability is required	

*Percent sulfate by mass in soil shall be determined by ASTM C1580.

†Concentration of dissolved sulfates in water, in ppm, shall be determined by ASTM D516 or ASTM D4130.

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The Code addresses three exposure categories that affect the requirements for concrete to ensure adequate durability:

Exposure Category F applies to concrete exposed to moisture and cycles of freezing and thawing, with or without deicing chemicals.

Exposure Category S applies to concrete in contact with soil or water containing deleterious amounts of water-soluble sulfate ions.

Exposure Category W applies to concrete in contact with water.

Severity of exposure within each category is defined by classes with increasing numerical values representing increasingly severe exposure conditions. A classification of 0 is assigned if the exposure severity has negligible effect (is benign) or the exposure category does not apply to the member.

The following discussion provides assistance for selecting the appropriate exposure class for each of the exposure categories. Members are required to be assigned to three exposure classes, one for each exposure category, and are also required to meet the most restrictive requirements of all of these exposures. For example, the slabs of a parking garage in a cold climate might be assigned to Exposure Classes F3, S0, and W2, and a potable water tank inside a heated building might be assigned to Exposure Classes F0, S0, and W2.

Exposure Category F: Whether concrete is damaged by cycles of freezing and thawing depends on the amount of water in the pores of the concrete at the time of freezing (Powers 1975). The amount of water present may be described in terms of the degree of saturation of the concrete. If the degree of saturation is high enough, there will be sufficient water in the concrete pores to produce internal tensile stresses large enough to cause cracking when the water freezes and expands. The entire member need not be saturated to be susceptible to damage. For example, if the top 10 mm of a slab or outer 6 mm of a wall is saturated, those portions are vulnerable to damage from freezing and thawing, regardless of how dry the interior may be.

For any portion of a member to be resistant to freezing and thawing, that portion of the concrete needs to have sufficient entrained air and adequate strength. Adequate strength is obtained by requiring a low *w/cm*, which also reduces the pore volume and increases resistance to water penetration. Entrained air makes it more difficult for the concrete to become saturated and allows for expansion of the water when it freezes. Exposure class varies with degree of exposure to water, as this will influence the likelihood that any portion of the concrete will be saturated when exposed to cyclic freezing and thawing. Conditions that increase the potential for saturation include longer-duration or more-frequent contact with water without intervening drainage or drying periods. The likelihood that concrete in a member will be saturated depends on project location, member location and orientation in the structure, and climate. Records of performance of similar members in existing structures

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in the same general location can also provide guidance in assigning exposure classes.

Exposure Category F is subdivided into four exposure classes:

(a) Exposure Class F0 is assigned to concrete that will not be exposed to cycles of freezing and thawing.

(b) Exposure Class F1 is assigned to concrete that will be exposed to cycles of freezing and thawing and that will have limited exposure to water. Limited exposure to water implies some contact with water and water absorption; however, it is not anticipated that the concrete will absorb sufficient water to become saturated. The licensed design professional should review the exposure conditions carefully to support the decision that the concrete is not anticipated to become saturated before freezing. Even though concrete in this exposure class is not expected to become saturated, a minimum entrained air content of 3.5 to 6% is required to reduce the potential for damage in case portions of the concrete member become saturated.

(c) Exposure Class F2 is assigned to concrete that will be exposed to cycles of freezing and thawing and that will have frequent exposure to water. Frequent exposure to water implies that some portions of the concrete will absorb sufficient water such that over time they will have the potential to be saturated before freezing. If there is doubt about whether to assign Exposure Classes F1 or F2 to a member, the more conservative choice, F2, should be selected. Exposure Classes F1 and F2 are conditions where exposure to deicing chemicals is not anticipated.

(d) Exposure Class F3 is assigned to concrete that will be exposed to cycles of freezing and thawing with the same degree of exposure to water as Exposure Class F2. Additionally, concrete in Exposure Class F3 is anticipated to be exposed to deicing chemicals. Deicing chemicals can increase water absorption and retention (Spragg et al. 2011), which would enable the concrete to become saturated more readily.

Table R19.3.1 provides examples of concrete members for each of these exposure classes.

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Table R19.3.1—Examples of structural members in Exposure Category F

Exposure class	Examples
F0	• Members in climates where freezing temperatures will not be encountered • Members that are inside structures and will not be exposed to freezing • Foundations not exposed to freezing • Members that are buried in soil below the frost line
F1	• Members that will not be subject to snow and ice accumulation, such as exterior walls, beams, girders, and slabs not in direct contact with soil • Foundation walls may be in this class depending upon their likelihood of being saturated
F2	• Members that will be subject to snow and ice accumulation, such as exterior elevated slabs • Foundation or basement walls extending above grade that have snow and ice buildup against them • Horizontal and vertical members in contact with soil
F3	• Members exposed to deicing chemicals, such as horizontal members in parking structures • Foundation or basement walls extending above grade that can experience accumulation of snow and ice with deicing chemicals

Exposure Category S is subdivided into four exposure classes:

(a) Exposure Class S0 is assigned for conditions where the water-soluble sulfate concentration in contact with concrete is low and injurious sulfate attack is not a concern.

(b) Exposure Classes S1, S2, and S3 are assigned for structural concrete members in direct contact with soluble sulfates in soil or water. The severity of exposure increases from Exposure Class S1 to S3 based on the more critical value of measured water-soluble sulfate concentration in soil or the concentration of dissolved sulfate in water. Seawater exposure is classified as Exposure Class S1.

Exposure Category W is subdivided into three exposure classes:

(a) Members are assigned to Exposure Class W0 if they are dry in service.

(b) Members are assigned to Exposure Class W1 if they may be exposed to continuous contact with water, to intermittent sources of water, or can absorb water from surrounding soil. Members assigned to W1 do not require concrete with low permeability.

(c) Members are assigned to Exposure Class W2 if they may be exposed to continuous contact with water, to intermittent sources of water, or can absorb water from surrounding soil, and if the penetration of water through the concrete might reduce durability or serviceability. Members assigned to W2 require concrete with low permeability.

19.3.2 *Requirements for concrete mixtures*

19.3.2.1 Based on the exposure classes assigned from Table 19.3.1.1, concrete mixtures shall conform to the most restrictive requirements in Table 19.3.2.1.

R19.3.2 *Requirements for concrete mixtures*

Durability of concrete is impacted by the resistance of the concrete to fluid penetration. This is primarily affected by *w/cm* and the composition of cementitious materials used in concrete. For a given *w/cm*, the use of fly ash, slag cement,

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Table 19.3.2.1—Requirements for concrete by exposure class

Exposure class	Maximum *w/cm*	Minimum f_c', MPa	Additional requirements			Limits on cementitious materials
			Air content			
F0	N/A	21	N/A			N/A
F1	0.55	24	Table 19.3.3.1			N/A
F2	0.45	31	Table 19.3.3.1			N/A
F3	0.45	31	Table 19.3.3.1			26.4.2.2(b)
			Cementitious materials*—Types			Calcium chloride admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	21	No type restriction	No type restriction	No type restriction	No restriction
S1	0.50	28	II†,‡	Types with (MS) designation	MS	No restriction
S2	0.45	31	V‡	Types with (HS) designation	HS	Not permitted
S3 Option 1	0.45	31	V plus pozzolan or slag cement§	Types with (HS) designation plus pozzolan or slag cement§	HS plus pozzolan or slag cement§	Not permitted
S3 Option 2	0.40	34	V#	Types with (HS) designation	HS	Not permitted
W0	N/A	21	None			
W1	N/A	21	26.4.2.2(d)			
W2	0.50	28	26.4.2.2(d)			

*Alternative combinations of cementitious materials to those listed in Table 19.3.2.1 are permitted when tested for sulfate resistance and meeting the criteria in 26.4.2.2(c).

†For seawater exposure, other types of portland cements with tricalcium aluminate (C_3A) contents up to 10% are permitted if the *w/cm* does not exceed 0.40.

‡Other available types of cement such as Type I or Type III are permitted in Exposure Classes S1 or S2 if the C_3A contents are less than 8% for Exposure Class S1 or less than 5% for Exposure Class S2.

§The amount of the specific source of the pozzolan or slag cement to be used shall be at least the amount that has been determined by service record to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slag cement to be used shall be at least the amount tested in accordance with ASTM C1012 and meeting the criteria in 26.4.2.2(c).

#If Type V cement is used as the sole cementitious material, the optional sulfate resistance requirement of 0.040% maximum expansion in ASTM C150 shall be specified.

silica fume, or a combination of these materials will typically increase the resistance of concrete to fluid penetration and thus improve concrete durability. The Code places emphasis on *w/cm* for achieving low permeability to meet durability requirements. ASTM C1202 can be used to provide an indication of concrete's resistance to fluid penetration.

Because *w/cm* of concrete cannot be accurately verified in the field using standard test methods, strength tests are used as a surrogate. Representative values for minimum f_c' have been assigned to each *w/cm* limit in Table 19.3.2.1. The acceptance criteria for strength tests in 26.12 establish a basis to indicate that the maximum *w/cm* has not been exceeded. For this approach to be reliable, the values of f_c' specified in construction documents should be consistent with the maximum *w/cm*. Considering the wide range of materials and concrete mixtures possible, including regional variations, the minimum f_c' limit in Table 19.3.2.1 associated with the maximum *w/cm* should not be considered absolute. The average strength of concrete mixtures for a given *w/cm* can in some cases be considerably higher than the average strength expected for the representative value of f_c'. For a given exposure class, the licensed design professional may

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choose to specify a higher value of f_c' than listed in the table to obtain better consistency between the maximum *w/cm* and f_c'. This improves the confidence that concrete complies with the *w/cm* limit if the strength acceptance criteria are satisfied.

Table 19.3.2.1 provides the requirements for concrete on the basis of the assigned exposure classes. The most restrictive requirements are applicable. For example, a member assigned to Exposure Class W1 and Exposure Class S2 would require concrete to comply with a maximum *w/cm* of 0.45 and a minimum f_c' of 31 MPa because the requirement for Exposure Class S2 is more restrictive than the requirement for Exposure Class W1.

Exposure Classes F1, F2, and F3: In addition to complying with a maximum *w/cm* limit and a minimum f_c', concrete for members subject to freezing-and-thawing exposures is required to be air entrained in accordance with 19.3.3.1. Members assigned to Exposure Class F3 are additionally required to comply with the limitations on the quantity of pozzolans and slag cement in the composition of the cementitious materials as given in 26.4.2.2(b).

The maximum *w/cm* and minimum f_c' requirements for GFRP-reinforced concrete members in Exposure Class F3 are identical to the requirements for plain concrete members in Exposure Class F3 in ACI 318. The requirements for GFRP-reinforced and plain concrete members in Exposure Class F3 are less restrictive than requirements for steel-reinforced concrete members because of the reduced likelihood of problems caused by GFRP reinforcement corrosion. The licensed design professional should consider the details of GFRP-reinforced concrete members to ensure that the less restrictive requirements are appropriate for the specific project.

Exposure Classes S1, S2, and S3: Table 19.3.2.1 lists the appropriate types of cement and the maximum *w/cm* and minimum f_c' for various sulfate exposure conditions. In selecting cement for sulfate resistance, the principal consideration is its tricalcium aluminate (C_3A) content.

The use of fly ash (ASTM C618, Class F), natural pozzolans (ASTM C618, Class N), silica fume (ASTM C1240), or slag cement (ASTM C989) has been shown to improve the sulfate resistance of concrete (Li and Roy 1986; ACI 233R; ACI 234R). Therefore, Footnote [§] to Table 19.3.2.1 provides a performance option to determine the appropriate amounts of these materials to use in combination with the specific cement types listed. ASTM C1012 is permitted to be used to evaluate the sulfate resistance of mixtures using combinations of cementitious materials in accordance with 26.4.2.2(c).

Some ASTM C595 and ASTM C1157 blended cements can meet the testing requirements of 26.4.2.2(c) without addition of pozzolans or slag cement to the blended cement as manufactured.

Note that sulfate-resisting cement will not increase resistance of concrete to some chemically aggressive solutions—for example, sulfuric acid. The construction documents should explicitly cover such cases.

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In addition to the proper selection of cementitious materials, other requirements for durable concrete exposed to water-soluble sulfates are essential, such as *w*/*cm*, strength, consolidation, uniformity, cover of reinforcement, and moist curing to develop the potential properties of the concrete.

Exposure Class S1: ASTM C150 Type II cement is limited to a maximum C_3A content of 8.0% and is acceptable for use in Exposure Class S1. Blended cements under ASTM C595 with the MS designation, which indicates the cements meet requirements for moderate sulfate resistance, are also appropriate for use. Under ASTM C1157, the appropriate designation for moderate sulfate exposure is Type MS.

Seawater is listed under Exposure Class S1 (moderate exposure) in Table 19.3.1.1, even though it generally contains more than 1500 ppm SO_4^{2-}. Less expansion is produced by a given cement in seawater compared with freshwater with the same sulfate content (ACI 201.2R). Therefore, seawater is included in the same exposure class as solutions with lower sulfate concentrations. Portland cement with C_3A up to 10% is allowed in concrete mixtures exposed to seawater if the maximum *w*/*cm* is limited to 0.40 (refer to the footnote to Table 19.3.2.1).

Exposure Class S2: ASTM C150 Type V cement is limited to a maximum C_3A content of 5.0% and is acceptable for use in Exposure Class S2. The appropriate binary and ternary blended cements under ASTM C595 are Types IP, IS, and IT that include the suffix (HS) as part of their designation, which indicates the cement conforms to requirements for high sulfate resistance. Under ASTM C1157, the appropriate designation for severe sulfate exposure is Type HS.

Exposure Class S3 (Option 1): The benefit of the addition of pozzolan or slag cement allows for a greater *w*/*cm* than required for Option 2. The amounts of supplementary cementitious materials are based on records of successful service or testing in accordance with 26.4.2.2(c).

Exposure Class S3 (Option 2): This option allows the use of ASTM C150 Type V portland cement meeting the optional limit of 0.040% maximum expansion, ASTM C595 binary and ternary blended cements with the (HS) suffix in their designation, and ASTM C1157 Type HS cements without the use of additional pozzolan or slag cement, but it instead requires a lower *w*/*cm* than that required for Option 1. This lower *w*/*cm* reduces the permeability of the concrete and thus increases sulfate resistance (Lenz 1992). Use of this lower *w*/*cm* permits a shorter testing period to qualify the sulfate resistance of a cementitious system in accordance with 26.4.2.2(c).

In addition to the proper selection of cementitious materials, other requirements for durable concrete exposed to water-soluble sulfates are essential, such as low *w*/*cm*, strength, adequate consolidation, uniformity, adequate cover of reinforcement, and sufficient moist curing to develop the potential properties of the concrete.

Exposure Class W1: This exposure class does not have specific requirements for low permeability. However, because of the exposure to water, the Code (26.4.2.2(d)) has a requirement to demonstrate that aggregates used in concrete are not alkali reactive according to ASTM C1778. If the aggregates are alkali-silica reactive, the Code (26.4.2.2(d)) also requires submission of proposed mitigation measures. The Code (26.4.2.2(d)) prohibits the use of aggregates that are alkali-carbonate reactive.

Exposure Class W2: This exposure class requires low concrete permeability. The primary means to obtain a concrete with low permeability is to reduce *w/cm*. For a given *w/cm*, permeability can be reduced by optimizing the cementitious materials used in the concrete mixture. In addition, because of the exposure to water, the Code (26.4.2.2(d)) has a requirement to demonstrate that aggregates used in concrete are not alkali reactive according to ASTM C1778. If the aggregates are alkali-silica reactive, the Code (26.4.2.2(d)) also requires submission of proposed mitigation measures. The Code (26.4.2.2(d)) prohibits the use of aggregates that are alkali-carbonate reactive.

19.3.3 *Additional requirements for freezing-and-thawing exposure*

R19.3.3 *Additional requirements for freezing-and-thawing exposure*

⁼**19.3.3.1** Concrete subject to freezing-and-thawing Exposure Classes F1, F2, or F3 shall be air entrained. Except as permitted in 19.3.3.6, air content shall conform to Table 19.3.3.1.

Table 19.3.3.1—Total air content for concrete exposed to cycles of freezing and thawing

Nominal maximum aggregate size, mm	Target air content, percent	
	F1	F2 and F3
10	6	7.5
13	5.5	7
19	5	6
25	4.5	6
38	4.5	5.5
50	4	5
75	3.5	4.5

⁼**R19.3.3.1** A table of required air contents for concrete to resist damage from cycles of freezing and thawing is included in the Code, based on guidance provided for proportioning concrete mixtures in ACI 211.1. Entrained air will not protect concrete containing coarse aggregates that undergo disruptive volume changes when frozen in a saturated condition.

⁼**19.3.3.2** Concrete shall be sampled in accordance with ASTM C172, and air content shall be measured in accordance with ASTM C231 or ASTM C173.

⁼**R19.3.3.2** The sampling of fresh concrete for acceptance based on air content is usually performed as the concrete is discharged from a mixer or a transportation unit (for example, a ready mixed concrete truck) to the conveying equipment used to transfer the concrete to the forms. ASTM C172 primarily covers sampling of concrete as it is discharged from a mixer or a transportation unit but recognizes that specifications may require sampling at other points such as discharge from a pump. Table 19.3.3.1 was developed for testing as-delivered concrete. ASTM C231 is applicable to normalweight concrete and ASTM C173 is applicable to normalweight or lightweight concrete.

If the licensed design professional requires measurement of air content of fresh concrete at additional sampling loca-

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tions, such requirements should be stated in the construction documents, including the sampling protocol, test methods to be used, and the criteria for acceptance.

19.3.3.3 Intentionally left blank.

19.3.3.4 Intentionally left blank.

19.3.3.5 Intentionally left blank.

19.3.3.6 For $f_c' \geq 34$ MPa, reduction of air content indicated in Table 19.3.3.1 by 1.0 percentage point is permitted.

=R19.3.3.6 This section permits a 1.0 percentage point lower air content for concrete with f_c' greater than 34 MPa. Such higher-strength concretes, which have a lower *w/cm* and porosity, have greater resistance to cycles of freezing and thawing.

=19.3.3.7 The maximum percentage of pozzolans, including fly ash and silica fume, and slag cement in concrete assigned to Exposure Class F3, shall be in accordance with 26.4.2.2(b).

=R19.3.3.7 This provision is intended for application during concrete mixture proportioning. The provision has been duplicated in 26.4.2.2(b). Additional commentary information is presented in Chapter 26.

19.3.4 *Additional requirements for chloride ion content*—Not applicable

19.4—Grout durability requirements—Out of scope

R19.4—Ground durability requirements—Out of scope

Grout durability requirements are not provided in this Code because of a lack of published research on the topic, combined with a perceived lack of need due to the Code not covering anchoring to concrete.

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CHAPTER 20—GFRP REINFORCEMENT PROPERTIES, DURABILITY, AND EMBEDMENTS

20.1—Scope

20.1.1 This chapter shall apply to GFRP reinforcement, and shall govern (a) through (c):

(a) Material properties

(b) Properties to be used for design

(c) Durability requirements, including minimum specified cover requirements

=**20.1.2** Provisions of 20.6 shall apply to embedments.

20.2—GFRP bars

20.2.1 *Material properties*

20.2.1.1 GFRP reinforcing bars that depend on the development of bond along their length shall have external surface enhancement.

20.2.1.2 GFRP reinforcing bars used in continuous-closed stirrups, continuous-closed ties or spirals are not required to have external surface enhancement.

20.2.1.3 The tensile behavior of GFRP bars in this Code is considered to be a linear elastic stress-strain relationship until failure.

20.2.1.4 GFRP bars shall conform to ASTM D7957.

20.2.1.4.1 If tensile strength is reported in units of force, the corresponding tensile strength in units of stress shall be calculated by dividing the tensile strength in units of force by the nominal cross-sectional area of the bar as given in ASTM D7957.

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CHAPTER R20—GFRP REINFORCEMENT PROPERTIES, DURABILITY, AND EMBEDMENTS

R20.1—Scope

R20.1.1 Materials permitted for use as reinforcement are specified. Other elements, such as inserts, anchor bolts, or plain bars for dowels at isolation or contraction joints, are not normally considered reinforcement under the provisions of this Code. Fiber-reinforced polymer (FRP) reinforcement other than GFRP, or reinforcement consisting of multiple fiber types, is not addressed in this Code. ACI Committee 440 has developed guidelines for the use of carbon and aramid FRP reinforcement (ACI 440.1R).

R20.2—GFRP bars

R20.2.1 *Material properties*

R20.2.1.1 Bond surface enhancement of GFRP bars may take the form of protrusions, lugs, sand coatings, helical wrapping with fibers, deformations or any surface treatment that provides means of mechanically transmitting force between the bar and the concrete surrounding the bar.

R20.2.1.2 Continuous stirrups and ties and spirals that are formed as closed sections without overlapping ends do not rely on surface enhancement to transmit force between the bar and the concrete, as they develop their stresses from the continuity of the fiber roving and from corner bends if present.

R20.2.1.3 If loaded in tension, GFRP bars do not exhibit plastic behavior (yielding) before rupture. The tensile behavior of GFRP bars permitted by this Code is characterized by a linear elastic stress-strain relationship until failure.

R20.2.1.4 The tensile strength and stiffness of GFRP bars are primarily governed by the fiber volume fraction. ASTM D7957 defines requirements for geometrical, material, mechanical, and physical characteristics of GFRP bars. Additionally, ASTM D7957 prescribes sampling protocols for bar qualification, quality control, and certification. Unlike steel reinforcing bars, the unit tensile strength of a GFRP bar decreases with increasing diameter due to the mechanics of shear transfer between the glass fibers. Test methods for determining the tensile strength and stiffness of GFRP bars can be found in ASTM D7205 and ASTM D7914.

R20.2.1.4.1 The nominal cross-sectional area defined by ASTM D7957 is based on the nominal bar diameter. ASTM D7957 specifies permissible ranges of the measured cross-sectional area associated with the corresponding nominal area.

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20.2.1.4.2 GFRP bars used as continuous-closed stirrups, continuous-closed ties or spirals shall not be required to meet the bond provisions in ASTM D7957.

20.2.1.5 Intentionally left blank.

20.2.1.6 Intentionally left blank.

20.2.1.7 Intentionally left blank.

20.2.2 *Design properties*

20.2.2.1 For straight GFRP bars and the straight portions of bent GFRP bars, the stress below the design tensile strength f_{fu} shall be E_f times GFRP strain.

20.2.2.2 The modulus of elasticity for straight GFRP bars shall be the value reported by the manufacturer as the mean elastic modulus in accordance with the requirements of ASTM D7957, or it shall be permitted to take E_f as the minimum value for the modulus of elasticity as specified in ASTM D7957.

20.2.2.3 For straight bars, the design tensile strength f_{fu} shall be determined according to:

$$f_{fu} = C_E f_{fu}^*$$

where

f_{fu}^* = guaranteed ultimate tensile strength, psi, which shall be the value reported by the manufacturer as the guaranteed ultimate tension force, computed as no larger than the mean strength minus three standard deviations, divided by the nominal cross-sectional area of the bar, in accordance with the requirements of ASTM D7957, or it shall be permitted to take f_{fu}^* as the minimum guaranteed ultimate tensile force specified in ASTM D7957 divided by the nominal cross-sectional area of the bar; and

C_E = environmental reduction factor, which shall be 0.85 for concrete both exposed and not exposed to earth or weather.

20.2.2.4 For a bent portion of bar, the design tensile strength f_{fb} shall be determined according to:

$$f_{fb} = C_E f_{fb}^*$$

and shall not exceed f_{fu}, where

f_{fb}^* = guaranteed ultimate tensile strength of bent portion of bar, psi, which shall be the value reported by the manufacturer as the guaranteed ultimate tensile force, computed as no larger than the mean strength minus three standard deviations, of bent portion of bar divided by the nominal cross-sectional area of the bar, in accordance with the requirements of ASTM D7957, or it shall be permitted to take f_{fb}^* as the minimum value specified in ASTM D7957 for the guar-

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R20.2.2 *Design properties*

R20.2.2.1 The tensile behavior of GFRP bars permitted by this Code is characterized by a linear elastic stress-strain relationship until failure.

R20.2.2.2 Instead of a manufacturer reported value, the minimum value for the modulus of elasticity as specified in ASTM D7957 may be used as the value on which design can be based without preselecting a bar manufacturer.

R20.2.2.3 Instead of a manufacturer reported value for f_{fu}^*, the minimum values as specified in ASTM D7957 for the guaranteed ultimate tension force divided by the nominal cross-sectional area of the bar may be used as the values on which design can be based without preselecting a bar manufacturer.

Benmokrane et al. (2020) determined that a C_E value of 0.85 was appropriate for GFRP bars based on a compiled database of 361 accelerated aging tests of unstressed bars.

R20.2.2.4 Instead of a manufacturer reported value for f_{fb}^*, the minimum value as specified in ASTM D7957 for the guaranteed ultimate tensile force of bent portion of bar divided by the nominal cross-sectional area of the bar may be used as the value on which design can be based without preselecting a bar manufacturer.

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anteed ultimate tensile force of bent portion of bar divided by the nominal cross-sectional area of the bar; and
C_E = environmental reduction factor, as specified in 20.2.2.3.

20.2.2.5 For GFRP straight bars, the design rupture strain ε_{fu} shall be determined according to $\varepsilon_{fu} = f_{fu}/E_f$.

20.2.2.6 For GFRP transverse reinforcement, the design tensile strength f_{ft} shall not exceed the smaller of f_{fb} and $0.005E_f$.

20.2.2.7 Nominal dimensions of reinforcing bars shall be used to calculate the area of GFRP reinforcing bars.

20.3—Prestressing strands, wires, and bars—Out of scope

20.4—Headed shear stud reinforcement—Out of scope

20.5—Provisions for durability of GFRP reinforcement

20.5.1 *Specified concrete cover*

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R20.2.2.6 The design tensile strength of GFRP transverse reinforcement is controlled by the strength of the bent portion of the bar and by a strain limit of 0.005 to avoid loss of aggregate interlock. Refer to R22.5.3.3.

R20.3—Prestressing strands, wires, and bars—Out of scope

Specification of prestressing strands, wires, and bars are not covered in this Code because this Code does not cover GFRP-prestressed concrete members.

R20.4—Headed shear stud reinforcement—Out of scope

Requirements for headed shear stud reinforcement are not covered in this Code due to a lack of ANSI-approved specifications for GFRP headed shear studs.

R20.5—Provisions for durability of GFRP reinforcement

Durability requirements for structures utilizing GFRP bars are inherently different from those of steel-reinforced concrete due to the corrosion-resistant nature of GFRP. Design criteria intended to mitigate corrosion of the internal reinforcement such as increased concrete cover, the use of corrosion inhibiting admixtures, use of epoxy coatings, and limitations on crack widths to delay the initiation of corrosion are not necessary in GFRP-reinforced concrete structures. However, the Code does not include provisions for especially severe exposures such as acids, although low-pH environments are less severe for GFRP reinforcement than are high-pH environments (Al-Zahrani et al. 2002; Bazli et al. 2017). In addition, the effects of creep rupture (static fatigue) and/or time dependent properties of the GFRP bar must be taken into account to ensure long-term safe use. The durability provisions of this section pertain to fire protection and the long-term bond properties of GFRP bars. Effects of creep rupture (static fatigue) are addressed in 24.6.

R20.5.1 *Specified concrete cover*

Unlike steel reinforcing bars where concrete cover and durability are related, the provisions of concrete cover for GFRP bars are related only to constructability, bond, and fire-related performance issues. This section addresses concrete cover over GFRP reinforcement and does not include requirements for concrete cover over embedments such as pipes, conduits, and fittings, which are addressed in 20.6.5.

20.5.1.1 The minimum specified concrete cover shall be in accordance with 20.5.1.2 through 20.5.1.3.

R20.5.1.1 Concrete cover as protection of GFRP reinforcement from environmental and other effects is measured from the concrete surface to the outermost surface of the reinforcement to which the cover requirement applies. Where concrete cover is prescribed for a class of structural members, it is measured to the outer edge of stirrups, ties, or spirals if transverse reinforcement encloses main bars; or to the outermost layer of bars if more than one layer is used without stirrups or ties. The condition "exposed to weather" refers to temperature changes typically involving solar radiative heating. Moisture does not adversely affect the cover requirement for GFRP-reinforced concrete members. Alternative methods of protecting the GFRP reinforcement from weather may be provided if they are equivalent to the additional concrete cover required by the Code. When approved by the building official under the provisions of 1.10, reinforcement with alternative protection from weather may not have concrete cover less than the cover required for reinforcement not exposed to weather. Development length provisions given in Chapter 25 and maximum bar spacing requirements given in Chapter 24 are a function of cover over the GFRP reinforcement. To meet requirements for development length, it may be necessary to use cover greater than the minimums specified in 20.5.1.3.

⁼**20.5.1.2** It shall be permitted to consider concrete floor finishes as part of required cover for nonstructural purposes.

⁼**R20.5.1.2** Concrete floor finishes may be considered for nonstructural purposes such as cover for GFRP reinforcement and fire protection. Provisions should be made, however, to ensure that the concrete finish will not spall off, thus resulting in decreased cover. Furthermore, considerations for development of reinforcement require minimum monolithic concrete cover in accordance with 20.5.1.3.

20.5.1.3 *Specified concrete cover requirements*

R20.5.1.3 *Specified concrete cover requirements*

20.5.1.3.1 Cast-in-place and precast concrete members shall have specified concrete cover for reinforcement at least that given in Table 20.5.1.3.1.

R20.5.1.3.1 Table 20.5.1.3.1 reports minimum concrete cover requirements for GFRP reinforcement. Larger cover or other fire mitigation strategies may be required for fire protection.

The required concrete cover provided in Table 20.5.1.3.1 is sufficient to assure that the bond and development length equations provided in Chapter 25 are valid. Concrete exposed to weather will experience thermal cycling. Because of the difference in coefficients of thermal expansion between concrete and GFRP reinforcement in the transverse direction, thermal cycling can cause longitudinal cracking that affects the bond between the bar and the concrete if sufficient cover is not provided. A cover of at least $2d_b$ has been found sufficient to control cracking due to thermal cycling alone in smaller diameter bars (Aiello et al. 2001). Additional cover beyond the $2d_b$ is required to address the additional splitting stresses due to flexural bond. Unlike steel bars, if GFRP bars are not adequately anchored, high temperatures

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Table 20.5.1.3.1—Specified concrete cover for cast-in-place and precast concrete members

Concrete exposure	Member	GFRP reinforcement	Specified cover, mm
Cast against and permanently in contact with ground	All	All	75
Exposed to weather	All	No. M19 through No. M32 bars	50
		No. M16 bar and smaller	38
Not exposed to weather or cast against the ground	Slabs, joists, and walls	All	19
	Beams, columns, pedestals, and tension ties	All	38

20.5.1.4 *Specified concrete cover requirements for corrosive environments*—Not applicable

20.5.2 *Nonprestressed coated reinforcement*—Not applicable

20.5.3 *Corrosion protection for unbonded prestressing reinforcement*—Not applicable

20.5.4 *Corrosion protection for grouted tendons*—Not applicable

20.5.5 *Corrosion protection for post-tensioning anchorages, couplers, and end fittings*—Not applicable

20.5.6 *Corrosion protection for external post-tensioning*—Not applicable

20.6—Embedments

=**20.6.1** Embedments shall not significantly impair the strength of the structure and shall not reduce fire protection.

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during fire can cause a loss of bond. In addition to sufficient cover, specific GFRP reinforcement detailing for anchorage is needed to ensure bond of the GFRP bars during fire (Nigro et al. 2013; McIntyre et al. 2014). The concrete covers listed in Table 20.5.1.3.1 provide the minimum fire resistances shown in Table R20.5.1.3.1 for applications that are not bond critical, such as anchoring of the GFRP bars at the end of a member in a location not directly exposed to fire (Nigro et al. 2013). These fire resistances are based on a semi-infinite concrete mass cast from siliceous aggregate concrete, and conservatively assume a GFRP polymer resin decomposition temperature of 350°C applied in a non-bond-critical application. The minimum overall member dimensions given in ACI 216.1M also need to be observed to achieve these fire resistances.

Table R20.5.1.3.1—Fire resistance rating provided by minimum cover for non-bond-critical GFRP reinforcement

Specified cover, mm	Fire resistance, *h*		
	Slabs and non-load-bearing walls	Beams	Columns and load-bearing walls
50	1.5	1	0.5
38	1	0.5	0.5
19	0.5	NA	Less than 0.5

R20.6—Embedments

=**R20.6.1** Any embedments not harmful to concrete or reinforcement can be placed in the concrete, but the work should be done in such a manner that the structure will not be endangered. Many general building codes have adopted ASME Piping Code B31.1 for power piping and B31.3 for chemical and petroleum piping. The licensed design professional should be sure that the appropriate piping codes are used in the design and testing of the system. The contractor

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=**20.6.2** Embedment materials shall not be harmful to concrete or reinforcement.

20.6.3 Aluminum embedments shall be coated or covered to prevent aluminum-concrete reaction.

20.6.4 Reinforcement with an area at least $\mathbf{1650/}\boldsymbol{E_f}$ times the area of the concrete section shall be provided perpendicular to pipe embedments.

=**20.6.5** Specified concrete cover for pipe embedments with their fittings shall be at least 38 mm for concrete exposed to earth or weather, and at least 19 mm for concrete not exposed to weather, or not in contact with ground.

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should not be permitted to install conduits, pipes, ducts, or sleeves that are not shown in the construction documents or not approved by the licensed design professional.

R20.6.3 The Code prohibits the use of aluminum in structural concrete unless it is effectively coated or covered. Aluminum reacts with concrete causing cracking, spalling, or both. Aluminum electrical conduits present a special problem because stray electric current accelerates the adverse reaction.

R20.6.4 The value of $\mathbf{1650/}\boldsymbol{E_f}$ is the same as the requirement for shrinkage and temperature reinforcement.

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CHAPTER 21—STRENGTH REDUCTION FACTORS

21.1—Scope

21.1.1 This chapter shall apply to the selection of strength reduction factors used in design.

21.2—Strength reduction factors for structural concrete members and connections

21.2.1 Strength reduction factors ϕ shall be in accordance with Table 21.2.1, except as modified by 21.2.2.

Table 21.2.1—Strength reduction factors ϕ

	Action or structural element	ϕ
(a)	Moment, axial force, or combined moment and axial force	0.55 to 0.65 in accordance with 21.2.2
(b)	Shear	0.75
(c)	Torsion	0.75
(d)	Bearing	0.65

21.2.2 Strength reduction factor for moment, axial force, or combined moment and axial force shall be in accordance with Table 21.2.2.

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CHAPTER R21—STRENGTH REDUCTION FACTORS

R21.1—Scope

R21.1.1 The purposes of strength reduction factors ϕ are: (1) to account for the probability of under-strength members due to variations in material strengths and dimensions; (2) to account for inaccuracies in the design equations; (3) to reflect the required reliability of the member under the load effects being considered; and (4) to reflect the importance of the member in the structure (MacGregor 1976; Winter 1979).

R21.2—Strength reduction factors for structural concrete members and connections

R21.2.1 The strength reduction factors in this Code are compatible with the ASCE/SEI 7 load combinations which are the basis for the required factored load combinations in Chapter 5.

Table 21.2.1 Row (a): Because GFRP-reinforced concrete sections have greater variability than tension-controlled steel-reinforced concrete sections and do not exhibit ductile behavior, a lower resistance factor has been adopted to ensure the same reliability. Based on ACI 318, the ϕ factor for design of a compression-controlled section is 0.65, with a target reliability index between 3.5 and 4.0 (Szerszen and Nowak 2003). A reliability analysis on GFRP-reinforced concrete beams in flexure using Load Combination 2 from ACI 318 for live to dead load ratios between 1 and 3 indicated reliability indexes between 3.5 and 4.0 when the ϕ factor was set to 0.65 for a compression-controlled section, and 0.55 for a tension-controlled section (Shield et al. 2011). A nonlinear sectional analysis of curvatures at failure showed that the curvatures of representative GFRP-reinforced concrete beams at failure varied between $\mathbf{0.016}/d$ and $\mathbf{0.018}/d$ for tension-controlled failures, and between $\mathbf{0.011}/d$ and $\mathbf{0.02}/d$ for compression-controlled failures (Shield et al. 2011). ACI 318 considers the section tension-controlled whenever the curvature is greater than $\mathbf{0.008}/d$ in steel-reinforced concrete sections (corresponding to a strain in the steel of 0.005). Due to the low modulus of elasticity of the reinforcement, GFRP-reinforced concrete beams will have large deflections at ultimate, and GFRP-reinforced concrete beams with controlling limit states of reinforcing bar rupture will typically have larger deflections at ultimate than those that are controlled by concrete crushing. Even though the curvature values of GFRP-reinforced concrete beams are larger than those of equivalent steel-reinforced concrete beams, a ϕ factor of 0.55 is used for tension-controlled section design to maintain a minimum reliability index of 3.5.

R21.2.2 ACI 440.1R establishes values of ϕ for flexure by comparing the GFRP reinforcement ratio ρ_f to the GFRP balanced reinforcement ratio ρ_{fb}. The expressions from ACI 440.1R can be rewritten in terms of ε_{ft}, which is the net tensile strain at failure in the outermost layer of GFRP reinforcement.

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Table 21.2.2—Strength reduction factor ϕ for moment, axial force, or combined moment and axial force

Net tensile strain at failure in the outermost layer of reinforcement, ε_{ft}	Classification	ϕ
$\varepsilon_{ft} = \varepsilon_{fu}$	Tension-controlled	0.55
$\varepsilon_{fu} > \varepsilon_{ft} > 0.8\varepsilon_{fu}$	Transition*	$1.05 - 0.5\varepsilon_{ft}/\varepsilon_{fu}$
$\varepsilon_{ft} \leq 0.8\varepsilon_{fu}$	Compression-controlled	0.65

*For sections classified as transition, it shall be permitted to use ϕ corresponding to tension-controlled sections.

21.2.3 Intentionally left blank.

21.2.4 Intentionally left blank.

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The $\mathbf{0.8\varepsilon_{fu}}$ boundary between the transition- and compression-controlled classifications in Table 21.2.2 is to ensure that failure will occur by concrete crushing in the compression-controlled classifications if the placed concrete is overstrength and is in keeping with the philosophy from ACI 440.1R. For sections within the transition region, the value of ϕ may be determined by linear interpolation, as shown in Fig. R21.2.2.

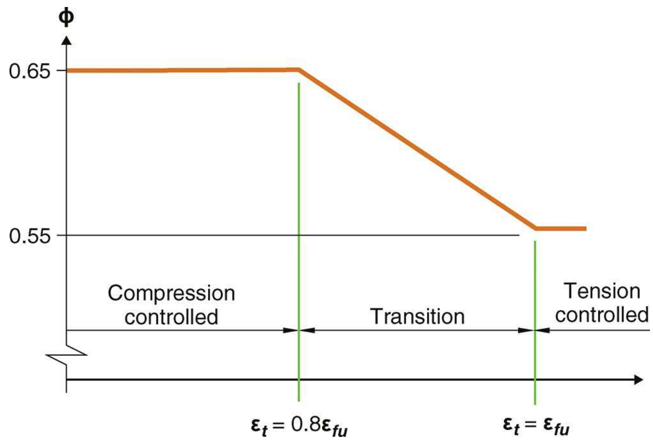

Fig. R21.2.2—Variation of ϕ with net tensile strain in extreme tension reinforcement ε_t.

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CHAPTER 22—SECTIONAL STRENGTH

22.1—Scope

22.1.1 This chapter shall apply to calculating nominal strength at sections of members, including (a) through (f):

(a) Flexural strength
(b) Axial strength or combined flexural and axial strength
(c) One-way shear strength
(d) Two-way shear strength
(e) Torsional strength
(f) Bearing

22.1.2 Intentionally left blank.

⁼**22.1.3** Design strength at a section shall be taken as the nominal strength multiplied by the applicable strength reduction factor, ϕ, given in Chapter 21.

22.2—Design assumptions for moment and axial strength

22.2.1 *Equilibrium and strain compatibility*

⁼**22.2.1.1** Equilibrium shall be satisfied at each section.

22.2.1.2 Strain in concrete and reinforcement shall be assumed proportional to the distance from neutral axis.

22.2.1.3 Intentionally left blank.

22.2.1.4 Intentionally left blank.

22.2.2 *Design assumptions for concrete*

⁼**22.2.2.1** Maximum strain at the extreme concrete compression fiber shall be assumed equal to 0.003.

CHAPTER R22—SECTIONAL STRENGTH

R22.1—Scope

R22.1.1 The provisions in this chapter apply where the strength of the member is evaluated at critical sections. Existing methods for designing discontinuity regions in steel-reinforced concrete members cannot be applied to GFRP-reinforced concrete members due to a lack of published research on this topic. Strut-and-tie models are most appropriate when considering elastic-perfectly plastic behavior and are therefore generally not suitable for use with GFRP reinforcement.

R22.2—Design assumptions for moment and axial strength

R22.2.1 *Equilibrium and strain compatibility*

The flexural and axial strength of a member calculated by the strength design method of the Code requires that two basic conditions be satisfied: (1) equilibrium; and (2) compatibility of strains. Equilibrium refers to the balancing of forces acting on the cross section at nominal strength. The relationship between the stress and strain for the concrete and the reinforcement at nominal strength is established within the design assumptions allowed by 22.2.

R22.2.1.2 Many tests have confirmed that it is reasonable to assume a linear distribution of strain across a reinforced concrete cross section (plane sections remain plane), even near nominal strength except in cases as described in ACI 318 Chapter 23.

The strain in reinforcement and in concrete is assumed to be directly proportional to the distance from the neutral axis. This assumption is of primary importance in design for determining the strain and corresponding stress in the reinforcement.

R22.2.2 *Design assumptions for concrete*

R22.2.2.1 The maximum concrete compressive strain at crushing of the concrete has been observed in tests of various kind to vary from 0.003 to higher than 0.008 under special conditions for concrete reinforced with steel. However, the strain at which strength of a steel-reinforced concrete member is developed is usually 0.003 to 0.004 for members of normal proportions, materials and strength. Similar

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behavior has been observed for compression-controlled flexural failures in concrete reinforced with GFRP (Kassem et al. 2011; Mousa et al. 2018; Nanni 1993).

22.2.2.2 Tensile strength of concrete shall be neglected in flexural and axial strength calculations.

R22.2.2.2 The tensile strength of concrete in flexure (modulus of rupture) is a more variable property than the compressive strength and is about 10 to 15% of the compressive strength. Tensile strength of concrete in flexure is conservatively neglected in calculating the nominal flexural strength. The strength of concrete in tension, however, is important in evaluating cracking and deflections at service loads.

22.2.2.3 The relationship between concrete compressive stress and strain shall be represented by a rectangular, trapezoidal, parabolic, or other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

R22.2.2.3 At high strain levels, the stress-strain relationship for concrete is nonlinear (stress is not proportional to strain). As stated in 22.2.2.1, the maximum usable strain is set at 0.003 for design. The actual distribution of concrete compressive stress within a cross section is complex and usually not known explicitly. Research has shown that the important properties of the concrete stress distribution can be approximated closely using any one of several different assumptions for the shape of the stress distribution.

22.2.2.4 The equivalent rectangular concrete stress distribution in accordance with 22.2.2.4.1 through 22.2.2.4.3 satisfies 22.2.2.3.

R22.2.2.4 For design, the Code allows the use of an equivalent rectangular compressive stress distribution (stress block) to replace the more detailed approximation of the concrete stress distribution.

22.2.2.4.1 Concrete stress of $\mathbf{0.85}\boldsymbol{f_c'}$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a line parallel to the neutral axis located a distance $\boldsymbol{a}$ from the fiber of maximum compressive strain, as calculated by:

$$a = \beta_1 c \qquad (22.2.2.4.1)$$

R22.2.2.4.1 The equivalent rectangular stress distribution does not represent the actual stress distribution in the compression zone at nominal strength, but does provide essentially the same nominal combined flexural and axial compressive strength as obtained in tests for concrete reinforced with steel (Mattock et al. 1961). Similar behavior has been observed for compression-controlled flexural failures in concrete reinforced with GFRP (GangaRao and Vijay 1997; Kassem et al. 2011). In cases where the failure mode is by GFRP rupture, nominal strength can be conservatively calculated from the equivalent rectangular stress distribution corresponding to a balanced failure. Refer to R22.3.1.1.

22.2.2.4.2 Distance from the fiber of maximum compressive strain to the neutral axis, $\boldsymbol{c}$, shall be measured perpendicular to the neutral axis.

22.2.2.4.3 Values of $\boldsymbol{\beta_1}$ shall be in accordance with Table 22.2.2.4.3.

Table 22.2.2.4.3—Values of β_1 for equivalent rectangular concrete stress distribution

f_c', MPa	β_1	
$21 \le f_c' \le 28$	0.85	(a)
$28 < f_c' \le 55$	$0.85 - \dfrac{0.05\,(f_c' - 28)}{7}$	(b)
$f_c' \ge 55$	0.65	(c)

R22.2.2.4.3 The values for $\boldsymbol{\beta_1}$ were determined experimentally. The lower limit of $\boldsymbol{\beta_1}$ is based on experimental data from steel-reinforced concrete beams constructed with concrete strengths greater than 55 MPa (Leslie et al. 1976; Karr et al. 1978). Similar behavior has been observed for concrete reinforced with GFRP (GangaRao and Vijay 1997; Hadhood et al. 2018a; Kassem et al. 2011).

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22.2.3 *Design assumptions for GFRP reinforcement*

22.2.3.1 GFRP reinforcement shall conform to 20.2.1.

22.2.3.2 Stress-strain relationship and modulus of elasticity for reinforcement in tension shall be idealized in accordance with 20.2.2.1 and 20.2.2.2.

22.2.3.3 GFRP reinforcement in compression is permitted. If present, the area of GFRP reinforcement in compression shall be treated as having the same strength and stiffness as the concrete in the surrounding compression zone.

22.2.4 *Design assumptions for prestressing reinforcement*—Out of scope

22.3—Flexural strength

22.3.1 *General*

=**22.3.1.1** Nominal flexural strength M_n shall be calculated in accordance with the assumptions of 22.2.

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R22.2.3.3 Testing of GFRP reinforcement in compression is complicated by GFRP's anisotropic and non-homogeneous nature. Deitz et al. (2003) reported a reduction in compressive strength of 50% and no compressive elastic modulus reduction when compared to the values in tension. The axial stiffness of GFRP moderately exceeds that of concrete in compression. Therefore, the modulus of elasticity of GFRP compression reinforcement can be treated as equal to the modulus of elasticity of the concrete it replaces, and the assumption of a modular ratio of 1 for GFRP reinforcement under compression when performing analysis and design is justifiable (Hadhood et al. 2017c).

R22.3—Flexural strength

R22.3.1 *General*

R22.3.1.1 The nominal flexural strength of a GFRP-reinforced concrete member can be determined based on (1) strain compatibility, in which the strain in each layer of GFRP bars must be considered separately; (2) internal force equilibrium; and (3) the controlling strength limit state (concrete crushing or GFRP rupture). The controlling limit state can be determined by comparing the GFRP reinforcement ratio ρ_f to the GFRP balanced reinforcement ratio ρ_{fb}, with the GFRP balanced reinforcement ratio ρ_{fb} calculated assuming that the concrete attains a 0.003 crushing strain simultaneously with the GFRP attaining the design rupture strain ε_{fu}. GFRP-reinforced concrete flexural members are typically designed first for serviceability, which often results in compression-controlled failure, where the GFRP reinforcement ratio is greater than the balanced ratio ($\rho_f > \rho_{fb}$), and the controlling limit state is crushing of the concrete. The corresponding tensile stress in the GFRP in the extreme tension layer at failure f_{fr} will be less than the design tensile strength f_{fu}. The stress distribution in the concrete can be approximated with the ACI rectangular stress block because the maximum concrete strain ε_{cu} is attained (ACI 440.1R).

If the GFRP reinforcement ratio is less than the balanced ratio ($\rho_f < \rho_{fb}$), the GFRP rupture limit state controls, and the nominal flexural strength of the section can be computed assuming the tensile stress in the GFRP f_{fr} is equal to the design tensile strength f_{fu}. Although the stress in the GFRP reinforcement is known, the analysis incorporates two unknowns: the concrete compressive strain at ultimate when the GFRP ruptures in tension (ε_c) and the depth to the neutral

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axis c. The analysis involving these unknowns becomes complex and is not easily solved by a closed-form solution. The ACI equivalent rectangular stress block parameters are not applicable because the maximum concrete strain may not be attained ($\varepsilon_c < \varepsilon_{cu}$). In this case, equivalent rectangular stress block parameters (the ratio of the average concrete stress to the concrete strength α_1 and the ratio of the depth of the equivalent rectangular stress block to the depth of the neutral axis β_1) that approximate the equivalent stress and centroid of the stress distribution in the concrete at the particular strain level reached would be required. For a given section, the product of $\beta_1 c$ varies depending on material properties and GFRP reinforcement ratio. For a section controlled by the limit state of GFRP rupture, the maximum value for this product is equal to $\beta_1 c_{bal}$ and is achieved if the maximum concrete strain (ε_{cu} = 0.003) is attained. Although more exact calculations for the neutral axis depth are permitted, a simplified and conservative lower bound for the nominal flexural strength of a rectangular section controlled by the GFRP rupture limit state can be based on the equilibrium of forces and strain compatibility shown in Fig. R22.3.1.1(c) as follows (ACI 440.1R)

$$M_n = A_f f_{fu}\left(d - \frac{\beta_1 c_{bal}}{2}\right) \qquad \text{(R22.3.1.1a)}$$

with

$$c_{bal} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} d \qquad \text{(R22.3.1.1b)}$$

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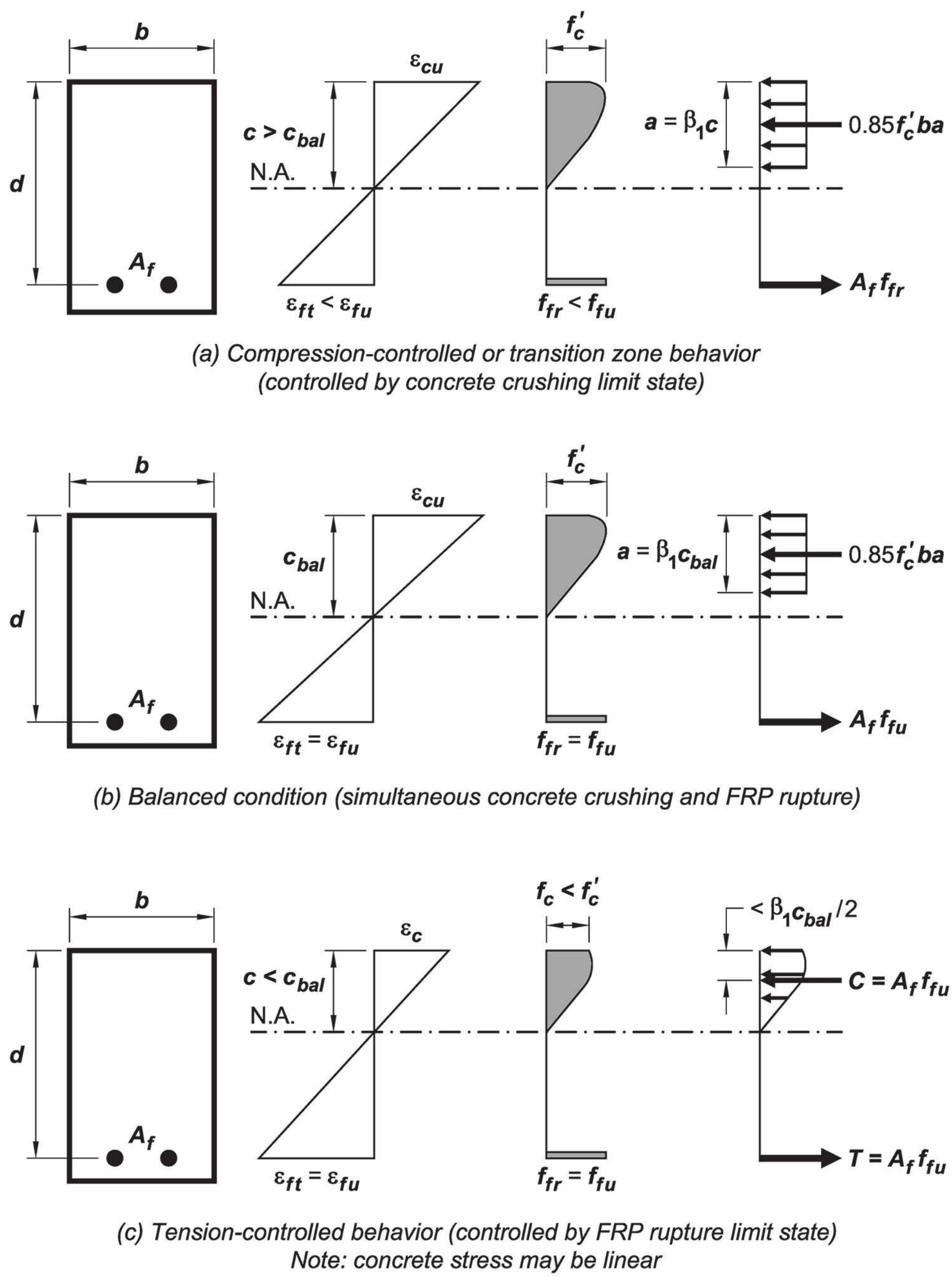

Fig. R22.3.1.1—Strain and stress distribution at ultimate conditions.

22.3.2 *Prestressed concrete members*—Out of scope

22.3.3 *Composite concrete members*

22.3.3.1 Provisions of 22.3.3 apply to members constructed in separate placements but connected so that all elements resist loads as a unit provided that the composite action does not rely on dowel action. Composite action which relies on GFRP dowels or GFRP bars continuous across an interface is not permitted.

R22.3.3 *Composite concrete members*

R22.3.3.1 The scope of Chapter 22 is intended to include composite concrete flexural members such as GFRP-reinforced precast concrete members composite with a concrete topping. The topping may be considered to contribute to the member strength provided that the shear transfer between the topping and the precast concrete occurs by friction at the interface. Shear transfer between the topping and the precast concrete which relies on GFRP reinforcement across the interface is not covered in this Code. In some cases with cast-in-place concrete, separate placements of concrete may be designed to act as a unit. In these cases, the interface is designed for the loads that will be transferred across the interface; shear transfer of such loads which relies on GFRP reinforcement across the interface is not covered in this Code.

22.3.3.2 For calculation of M_n for composite slabs and beams, use of the entire composite section shall be permitted.

22.3.3.3 For calculation of M_n for composite slabs and beams, no distinction shall be made between shored and unshored members.

22.3.3.4 For calculation of M_n for composite members where the specified concrete compressive strength of different elements vary, properties of the individual elements shall be used in design. Alternatively, it shall be permitted to use the value of f_c' for the element that results in the lowest value of ϕM_n.

22.4—Axial strength or combined flexural and axial strength

22.4.1 *General*

22.4.1.1 Nominal flexural and axial strength shall be calculated in accordance with the assumptions of 22.2.

R22.4—Axial strength or combined flexural and axial strength

R22.4.1 *General*

R22.4.1.1 The nominal flexural strength M_n and axial strength P_n of a column are based on design assumptions for concrete from 22.2.2, design assumptions for reinforcement from 22.2.3 and the tensile strain limit of reinforcement from 10.3.2. For bars in tension, if $P_u > 0.10f_c'A_g$, the stress in the reinforcement is limited by both f_{fd}, the tensile stress corresponding to a tensile strain of 0.01, and the design tensile strength f_{fu}. For bars in compression, the GFRP reinforcement is treated as having the same strength and stiffness as the concrete in the surrounding compression zone. The balanced failure point corresponds to the GFRP reinforcement reaching the maximum tensile strain (usually 0.010) at the same time the concrete crushes ($\varepsilon_{cu} = 0.003$). For axial loads less than the axial load at the balanced point, the compressive strain in the concrete will be less than 0.003 at failure, except if $P_u \leq 0.10f_c'A_g$. In some situations, the balanced point for GFRP-reinforced concrete columns may occur with a tensile axial load.

22.4.2 *Maximum axial strength*

22.4.2.1 Nominal axial compressive strength, P_n, shall not exceed $P_{n,max}$, in accordance with Table 22.4.2.1, where P_o is calculated by Eq. (22.4.2.2).

Table 22.4.2.1—Maximum axial strength

Transverse reinforcement	$P_{n,max}$	
Ties conforming to 22.4.2.4	$0.80P_o$	(a)
Spirals conforming to 22.4.2.5	$0.85P_o$	(b)

R22.4.2 *Maximum axial strength*

R22.4.2.1 To account for accidental eccentricity, the design axial strength of a section in pure compression is limited to 80 to 85% of the nominal axial strength. These percentage values approximate the axial strengths at eccentricity-to-depth ratios of 0.10 and 0.08 for tied and spirally GFRP-reinforced members conforming to 22.4.2.4 and 22.4.2.5, respectively (Hadhood et al. 2018b). The same axial load limitation applies to both cast-in-place and precast compression members.

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22.4.2.2 P_o shall be calculated by

$$P_o = 0.85f_c'A_g \qquad (22.4.2.2)$$

22.4.2.3 Intentionally left blank.

=**22.4.2.4** Tie reinforcement for lateral support of longitudinal reinforcement in compression members shall satisfy 10.7.6.2 and 25.7.2.

=**22.4.2.5** Spiral reinforcement for lateral support of longitudinal reinforcement in compression members shall satisfy 10.7.6.3 and 25.7.3.

22.4.3 *Maximum axial tensile strength*

22.4.3.1 Nominal axial tensile strength, P_{nt}, shall not be taken greater than $P_{nt,max}$, calculated by:

$$P_{nt,max} = f_{fu}A_f \qquad (22.4.3.1)$$

22.5—One-way shear strength

22.5.1 *General*

22.5.1.1 Nominal one-way shear strength at a section, V_n, shall be calculated by:

$$V_n = V_c + V_f \qquad (22.5.1.1)$$

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R22.4.2.2 GFRP compression reinforcement, while permitted, will not contribute significantly to the axial capacity of the cross section. The calculation of nominal axial strength may be simplified by assuming that GFRP reinforcement in compression has the same stiffness and strength as the surrounding concrete, and that P_o may be calculated using the gross area of concrete and f_c'. Several studies have shown that effectively neglecting the contribution of GFRP reinforcement in compression in this manner is conservative (Choo et al. 2006; De Luca et al. 2010; Tobbi et al. 2012; Jawaheri Zadeh and Nanni 2013; Afifi et al. 2014; Hadhood et al. 2016).

R22.5—One-way shear strength

R22.5.1 *General*

R22.5.1.1 In a member without shear reinforcement, shear is assumed to be resisted by the concrete. In a member with shear reinforcement, a portion of the shear strength is assumed to be provided by the concrete and the remainder by the shear reinforcement. Compared with a steel-reinforced concrete section with equal areas of longitudinal reinforcement, a cross section using GFRP flexural reinforcement has a smaller depth to the neutral axis after cracking, because of the lower axial stiffness E_fA_f. The compression region of the cross section is reduced, and the crack widths are larger. As a result, the shear resistance provided by both aggregate interlock and the uncracked flexural compression zone is smaller. Research on the shear capacity of steel-reinforced and GFRP-reinforced concrete flexural members without shear reinforcement has indicated that the concrete shear strength is influenced by the stiffness of the flexural tensile reinforcement (Zhao et al. 1995, Sonobe et al. 1997; Michaluk et al. 1998; Tureyen and Frosch 2002, 2003; El-Sayed et al. 2005a,b, 2006a,b). The contribution of longitudinal GFRP reinforcement in terms of dowel action has not been determined. Because of the lower strength and stiffness of GFRP bars in the transverse direction, the dowel action contribution is assumed to be less than that of an equivalent steel area.

22.5.1.2 Cross-sectional dimensions shall be selected to satisfy Eq. (22.5.1.2). For solid, circular sections b_w shall be permitted to be taken as the diameter and d shall be permitted to be taken as 0.8 times the diameter.

$$V_u \leq \phi 0.2 f_c' b_w d \quad (22.5.1.2)$$

R22.5.1.2 The limit on cross-sectional dimensions in 22.5.1.2 is intended to minimize the likelihood of diagonal compression failure in the concrete. The maximum shear a cross section can resist is limited by compression failure of the concrete diagonals in the web; the addition of shear reinforcement beyond this limit will not increase the shear capacity of the section. Therefore, irrespective of the amount of shear reinforcement, the maximum contribution to shear resistance from the shear reinforcement is limited by the crushing strength of the diagonal struts which is a function of both the diagonal crack angle and the strain in the shear reinforcement. Equation (22.5.1.2) minimizes the possibility of failure from crushing of the concrete in the web of the beam (Razaqpur and Spadea 2015). The limit in 22.5.1.2 of ACI 318, intended to control both diagonal compression failure and the width of inclined cracks (Joint ACI-ASCE Committee 426 1973), has been replaced by separate limits in this Code. Limiting the strain in the shear reinforcement to control diagonal cracking and maintain aggregate interlock is addressed in 22.5.3.3.

Shear tests of members with circular sections indicate that the effective area can be taken as the gross area of the section or as an equivalent rectangular area (Joint ACI-ASCE Committee 426 1973; Faradji and Diaz de Cossio 1965; Khalifa and Collins 1981).

22.5.1.3 V_c shall be calculated in accordance with 22.5.5.

22.5.1.4 Intentionally left blank.

22.5.1.5 Intentionally left blank.

22.5.1.6 V_f shall be calculated in accordance with 22.5.8.

22.5.1.7 Effect of any openings in members shall be considered in calculating V_n.

R22.5.1.7 Openings in the web of a member can reduce its shear strength. The effects of openings in steel-reinforced concrete are discussed in Section 4.7 of Joint ACI-ASCE Committee 426 (1973).

22.5.1.8 Effect of axial tension due to creep and shrinkage in restrained members shall be considered in calculating V_c.

R22.5.1.8 Consideration of axial tension requires engineering judgment. Axial tension often occurs due to volume changes, but it may be low enough not to be detrimental to the performance of a structure with adequate expansion joints and satisfying minimum longitudinal reinforcement requirements. It may be desirable to design shear reinforcement to resist the total shear if there is uncertainty about the magnitude of axial tension.

22.5.1.9 Effect of inclined flexural compression in variable depth members shall be permitted to be considered in calculating V_c.

R22.5.1.9 In a member of variable depth, the internal shear at any section is increased or decreased by the vertical component of the inclined flexural stresses.

22.5.1.10 Intentionally left blank.

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22.5.1.11 Intentionally left blank.

22.5.2 *Geometric assumptions*

22.5.2.1 Intentionally left blank.

22.5.2.2 For solid circular sections, (a) through (c) shall apply

(a) For calculations of V_c in Table 22.5.5.1 expression (a) and (c), $b_w k_{cr} d$ shall be replaced by the compression area of the elastic cracked transformed section

(b) For calculations of V_c using Table 22.5.5.1 expression (b), b_w shall be permitted to be taken as the diameter and d shall be permitted to be taken as 0.8 times the diameter.

(c) For calculations of V_f, d shall be permitted to be taken as 0.8 times the diameter.

22.5.3 *Limiting material strengths*

22.5.3.1 The value of $\sqrt{f_c'}$ used to calculate V_c for one-way shear shall not exceed 0.69 MPa.

22.5.3.2 Intentionally left blank.

22.5.3.3 The value of f_{ft} used to calculate V_f shall not exceed the limits in 20.2.2.6.

COMMENTARY

R22.5.3 *Limiting material strengths*

R22.5.3.1 Because of a lack of test data and practical experience with concretes having compressive strengths greater than 69 MPa, the Code imposes a maximum value of 0.69 MPa on $\sqrt{f_c'}$ for use in the calculation of shear strength of concrete members.

R22.5.3.3 The permissible stress level in GFRP shear reinforcement as specified in 20.2.2.6 is based on three criteria: (1) the maximum stress that a GFRP stirrup bar can carry due to the reduction in its strength caused by the bend at its corners, f_{fb}; (2) the maximum size of the diagonal cracks at ultimate state that would not seriously diminish shear transfer by aggregate interlock; and (3) the allowable size of the diagonal cracks under service load. In the case of steel-reinforced concrete, shear failure does not coincide with the initiation of yielding of transverse reinforcement; strains three to four times higher than the yield strain in steel-reinforced concrete have been observed prior to failure (Razaqpur and Spadea 2015). The 0.005 limit on level of strain for GFRP-reinforced concrete members can thus be attained without prematurely jeopardizing the shear capacity from loss of aggregate interlock. Joint ACI-ASCE Committee 426 (1973) concluded that it is possible to control crack widths at service loads by limiting the strain in the stirrups at ultimate. Their report noted that there is good correlation between inclined crack width and stirrup strain, and that the $\mathbf{0.66}\sqrt{f_c'}\, b_w d$ limit imposed by ACI 318 on $V_{s,max}$ corresponded to a maximum crack width of approximately 0.013 in. at service loads. Carpenter and Hanson (1969) used crack width relationships developed from flexural cracks to predict diagonal shear crack widths; they noted that ignoring the skew crack orientation led to reasonably conservative results. A similar approach based on flexural crack width data for GFRP bars (Shield et al. 2019) indicates that a stirrup strain of 0.005 would correspond to a 1.1 mm inclined crack width at ultimate; for the extreme case

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of dead to live load ratio of 5, the resulting maximum crack width at service load would be approximately 0.28 mm.

22.5.4 *Composite concrete members*

R22.5.4 *Composite concrete members*

22.5.4.1 This section shall apply to members constructed in separate placements but connected so that all elements resist loads as a unit provided that the composite action does not rely on GFRP dowel action. Composite action which relies on GFRP dowels or GFRP bars continuous across an interface is not permitted.

R22.5.4.1 The scope of Chapter 22 includes composite concrete members such as GFRP-reinforced precast concrete members composite with a concrete topping. The topping may be considered to contribute to the member strength provided that the shear transfer between the topping and the precast concrete occurs by friction at the interface. Shear transfer between the topping and the precast concrete which relies on GFRP reinforcement across the interface is not covered by this Code. In some cases with cast-in-place concrete, separate placements of concrete may be designed to act as a unit. In these cases, the interface is designed for the loads that will be transferred across the interface; shear transfer of such loads which relies on GFRP reinforcement across the interface is not covered by this Code.

=**22.5.4.2** For calculation of V_n for composite members, no distinction shall be made between shored and unshored members.

=**22.5.4.3** For calculation of V_n for composite members where the specified concrete compressive strength, unit weight, or other properties of different elements vary, properties of the individual elements shall be used in design. Alternatively, it shall be permitted to use the properties of the element that results in the most critical value of V_n.

=**22.5.4.4** If an entire composite member is assumed to resist vertical shear, it shall be permitted to calculate V_c assuming a monolithically cast member of the same cross-sectional shape.

22.5.4.5 If an entire composite member is assumed to resist vertical shear, it shall be permitted to calculate V_f assuming a monolithically cast member of the same cross-sectional shape if shear reinforcement is fully anchored into the interconnected elements in accordance with 25.7.

22.5.5 V_c *for nonprestressed members*

R22.5.5 V_c *for nonprestressed members*

22.5.5.1 V_c shall be calculated in accordance with Table 22.5.5.1 and 22.5.5.1.1 through 22.5.5.1.3. The ratio of the elastic cracked transformed section neutral axis depth to the effective depth, k_{cr}, shall be calculated taking into account the presence of axial load. The value of k_{cr} shall not be taken greater than 1, nor less than 0.

R22.5.5.1 The shear strength provided by concrete, V_c, is taken as the shear causing inclined cracking.

Whereas ACI 318 accounts for axial load, N_u, explicitly in the equations for one-way shear strength of the concrete, this Code implicitly incorporates axial load through the depth to the elastic cracked transformed section neutral axis, $k_{cr}d$. Directly applied axial load is included by considering the simultaneous action of service-level axial force in combination with service-level bending moment, at the location where V_c is to be computed.

Compared with a steel-reinforced concrete section with equal areas of longitudinal reinforcement, a cross section

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Table 22.5.5.1—V_c for members with and without axial load

Net axial load on section	V_c		
Compressive or no axial load	Greater of	$0.42\lambda_s k_{cr}\sqrt{f'_c}b_w d$	(a)
		$0.066\lambda_s\sqrt{f'_c}\,b_w d$	(b)
Tensile axial load	$0.42\lambda_s k_{cr}\sqrt{f'_c}\,b_w d$		(c)

22.5.5.1.1 It shall be permitted to neglect direct axial compression in the calculation of k_{cr}.

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using GFRP flexural reinforcement has a smaller depth to the neutral axis after cracking, because of the lower axial stiffness $E_f A_f$ of the reinforcement. In Table 22.5.5.1, expression (a) accounts for the axial stiffness of the GFRP reinforcement through the ratio of the elastic cracked transformed neutral axis depth to the effective depth of the section, k_{cr}, which is a function of the GFRP reinforcement ratio ρ_f and the modular ratio $n_f = E_f/E_c$. This equation has been shown to provide a reasonable estimate of shear strength for GFRP-reinforced concrete specimens across the range of reinforcement ratios and concrete strengths tested (Tureyen and Frosch 2003).

For lightly reinforced concrete members without shear reinforcement, such as slabs and foundations, Table 22.5.5.1 expression (a) may lead to unreasonably low estimates of shear capacity and thus expression (b) provides a lower limit on the shear capacity of the concrete, effectively providing a lower bound of 0.16 on k_{cr} for members in which axial tension is not present. The 0.16 lower limit for k_{cr} is based on a reliability analysis of slabs, and not by analogy with plain concrete (Nanni et al. 2014).

Direct tension in combination with flexure has the effect of reducing k_{cr} and, thus, V_c; therefore, the lower limit in expression (b) of Table 22.4.4.1 does not apply. Neglecting the effects of either sustained or short-term axial tension on the concrete contribution to shear strength is unconservative. Axial tension often occurs due to volume changes, but the levels may not be detrimental to the performance of a structure having adequate expansion joints and minimum reinforcement. If there is uncertainty about the magnitude of axial tension present, it may be appropriate to design GFRP shear reinforcement assuming $V_c = 0$.

Equations may be developed to calculate the ratio of the cracked transformed section neutral axis depth to the effective depth, k_{cr}. For singly reinforced, rectangular cross sections without axial tension or compression, k_{cr} may be determined from Eq. (R22.5.5.1a) and (R22.5.5.1b).

$$k_{cr,\,rect} = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \quad \text{(R22.5.5.1a)}$$

$$\rho_f = \frac{A_f}{b_w d} \quad \text{(R22.5.5.1b)}$$

For non-rectangular sections (such as T-beams in which the compression block extends into the web), k_{cr} can be computed based on strain compatibility and force equilibrium. An example of calculation for non-rectangular sections can be found in Higgins et al. (2022).

R22.5.5.1.1 Calculating the value for k_{cr} based on service-level moment alone (that is, not considering effects of direct axial compression on the location of the elastic neutral axis), results in a method that simply and conservatively estimates V_c for beams with axial compression. A more accurate value for the location of the neutral axis of the elastic cracked transformed section, $k_{cr}d$, may be calculated using

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strain compatibility and force equilibrium considering the effects of service-level axial load and service-level bending moment together. Such a calculation should consider only the sustained portion of the axial load that may be reasonably assumed to be present on the cross section in combination with the bending moment.

22.5.5.1.2 Intentionally left blank.

22.5.5.1.3 The size effect modification factor in Table 22.5.5.1, λ_s, shall be calculated in accordance with Table 22.5.5.1.3.

Table 22.5.5.1.3—Size effect modification factor λ_s

Criteria	λ_s
$A_{fv} < A_{fv,min}$	$\sqrt{\frac{2}{1+0.004d}} \leq 1.0$
$A_{fv} \geq Af_{v,min}$	1.0

R22.5.5.1.3 Test results (Frosch et al. 2017) for steel- and GFRP-reinforced nonprestressed concrete members without shear reinforcement indicate that the measured shear strength attributed to concrete does not increase in direct proportion with member depth. This phenomenon is often referred to as the "size effect." For example, if the member depth doubles, the shear at failure for the deeper beam may be less than twice the shear at failure of the shallower beam. $A_{fv,min}$ for beams and one-way slabs is defined in 9.6.3.4.

Research (Anderson 1978; Bažant and Kim 1984; Becker and Buettner 1985; Angelakos et al. 2001; Lubell et al. 2004; Brown et al. 2006; Bažant et al. 2007) has shown that shear stress at failure is lower for beams with increased depth and a reduced area of longitudinal reinforcement. The parameters within the size effect modification factor, λ_s, are consistent with the fracture mechanics theory for reinforced concrete and are appropriate for sections reinforced with either steel (Bažant et al. 2007 and Frosch et al. 2017) or GFRP (Frosch et al. 2017) reinforcement.

22.5.6 V$_c$ *for prestressed members*—Out of scope

22.5.7 V$_c$ *for pretensioned members in regions of reduced prestress force*—Out of scope

22.5.8 *One-way GFRP shear reinforcement*

R22.5.8 *One-way GFRP shear reinforcement*

22.5.8.1 At each section where $V_u > \phi V_c$, transverse reinforcement shall be provided such that Eq. (22.5.8.1) is satisfied.

$$V_f \geq \frac{V_u}{\phi} - V_c \qquad (22.5.8.1)$$

22.5.8.2 For one-way members reinforced with transverse reinforcement, V_f shall be calculated in accordance with 22.5.8.5.

R22.5.8.2 Provisions of 22.5.8.5 apply to all types of transverse reinforcement, including stirrups, ties, crossties, and spirals.

22.5.8.3 Intentionally left blank.

22.5.8.4 Intentionally left blank.

22.5.8.5 *One-way shear strength provided by GFRP transverse reinforcement*

R22.5.8.5 *One-way shear strength provided by GFRP transverse reinforcement*

Design of shear reinforcement is based on a modified truss analogy. In the truss analogy, the force in vertical ties is resisted by shear reinforcement. However, considerable research on both nonprestressed and prestressed steel-reinforced concrete members has indicated that shear reinforcement needs to be designed to resist only the shear exceeding that which causes inclined cracking, provided the diagonal members in the truss are assumed to be inclined at 45 degrees. Ahmed et al. (2010a,b) stated that the inclination angle of the shear crack in concrete beams reinforced with GFRP stirrups was in good agreement with the traditional 45-degree truss model. Shear failure modes of members with GFRP as shear reinforcement can be classified into two types: shear-tension failure mode (controlled by the rupture of GFRP shear reinforcement) and shear-compression failure mode (controlled by the crushing of the concrete web). The first mode is more brittle, and the latter results in larger deflections. Experimental results (Nagasaka et al. 1993; Shehata et al. 2000; Ahmed et al. 2010a,b,c) have shown that the modes of failure depend on the GFRP shear reinforcement index $\rho_{fv}E_f$, where ρ_{fv} is the ratio of GFRP shear reinforcement $A_{fv}/b_w s$. As the value of $\rho_{fv}E_f$ increases, the shear capacity in shear tension increases, and the mode of failure changes from shear tension to shear compression. In addition, the GFRP shear reinforcement index and the bond characteristics of the GFRP stirrups have a combined effect on the shear crack width (Ahmed et al. 2010c), with increased reinforcement index and higher bond strengths leading to better control of shear crack widths.

Equation (22.5.8.5.3) is presented in terms of nominal shear strength provided by shear reinforcement V_f. Where shear reinforcement perpendicular to the axis of the member is used, the required area of shear reinforcement, A_{fv}, and its spacing, s, are calculated by

$$\frac{A_{fv}}{s} = \frac{(V_u - \phi V_c)}{\phi f_{ft} d} \qquad \text{(R22.5.8.5)}$$

22.5.8.5.1 Shear reinforcement satisfying (a) or (b) shall be permitted:

(a) Stirrups or ties perpendicular to longitudinal axis of member

(b) Spiral reinforcement

22.5.8.5.2 Intentionally left blank.

22.5.8.5.3 V_f for shear reinforcement in 22.5.8.5.1 shall be calculated by:

$$V_f = A_{fv} f_{ft} \frac{d}{s} \qquad \text{(22.5.8.5.3)}$$

where s is the spiral pitch or the longitudinal spacing of the shear reinforcement, and A_{fv} is given in 22.5.8.5.5 or 22.5.8.5.6. For solid, circular sections d shall be permitted to be taken as 0.8 times the diameter.

R22.5.8.5.3 Although the transverse reinforcement in a circular section may not consist of straight legs, tests indicate that Eq. (22.5.8.5.3) is conservative if d is taken as 0.8 times the diameter (Ali et al. 2016; Mohamed et al. 2017).

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22.5.8.5.4 Intentionally left blank.

22.5.8.5.5 For each rectangular tie, stirrup, or crosstie, A_{fv} shall be the effective area of all bar legs within spacing s.

22.5.8.5.6 For each circular tie or spiral, A_{fv} shall be two times the area of the bar within spacing s.

25.5.8.6 *One-way shear strength provided by bent-up longitudinal bars*—Out of scope

22.6—Two-way shear strength

22.6.1 *General*

22.6.1.1 Provisions 22.6.1 through 22.6.5 apply to the nominal shear strength of two-way members without shear reinforcement.

22.6.1.2 Nominal shear strength for two-way members shall be calculated by

$$v_n = v_c \tag{22.6.1.2}$$

22.6.1.3 Intentionally left blank.

=**22.6.1.4** Two-way shear shall be resisted by a section with a depth d and an assumed critical perimeter b_o as defined in 22.6.4.

22.6.1.5 v_c for two-way shear shall be calculated in accordance with 22.6.5.

22.6.1.6 Intentionally left blank.

22.6.1.7 Intentionally left blank.

22.6.1.8 Intentionally left blank.

22.6.2 *Effective depth*

22.6.2.1 For calculation of v_c for two-way shear, d shall be the average of the effective depths in the two orthogonal directions and k_{cr} shall be based on the average reinforcement ratio ρ_f across all sides of the critical punching shear perimeter defined in 22.6.4.

22.6.2.2 Intentionally left blank.

COMMENTARY

R22.6—Two-way shear strength

Factored shear stress in two-way members due to shear and moment transfer is calculated in accordance with the requirements of 8.4.4. Section 22.6 provides requirements for determining nominal shear strength without shear reinforcement. Factored shear demand and strength are calculated in terms of stress, permitting superposition of effects from direct shear and moment transfer.

R22.6.1.1 Two-way members with shear reinforcement are not covered by this Code. Ignoring the effects of shear reinforcement on the shear strength of two-way members is conservative.

=**R22.6.1.4** The critical section perimeter b_o is defined in 22.6.4.

CODE

22.6.3 *Limiting material strengths*

⁼**22.6.3.1** The value of $\sqrt{f_c'}$ used to calculate v_c for two-way shear shall not exceed 0.69 MPa.

22.6.3.2 Intentionally left blank.

22.6.4 *Critical sections for two-way members*

⁼**22.6.4.1** For two-way shear, critical sections shall be located so that the perimeter b_o is a minimum but need not be closer than $d/2$ to (a) and (b):

(a) Edges or corners of columns, concentrated loads, or reaction areas

(b) Changes in slab or footing thickness, such as edges of capitals, drop panels, or shear caps

⁼**22.6.4.1.1** For square or rectangular columns, concentrated loads, or reaction areas, critical sections for two-way shear in accordance with 22.6.4.1(a) and (b) shall be permitted to be defined assuming straight sides.

⁼**22.6.4.1.2** For a circular or regular polygon-shaped column, critical sections for two-way shear in accordance with 22.6.4.1(a) and (b) shall be permitted to be defined assuming a square column of equivalent area.

22.6.4.2 Intentionally left blank.

22.6.4.3 If an opening is located closer than $10h$ from the periphery of a column, a concentrated load, or reaction area, a portion of b_o enclosed by straight lines projecting from the centroid of the column, concentrated load or reaction area and tangent to the boundaries of the opening shall be considered ineffective.

COMMENTARY

R22.6.3 *Limiting material strengths*

⁼**R22.6.3.1** There are limited test data on the two-way shear strength of high-strength concrete slabs. Until more experience is obtained for two-way slabs constructed with concretes that have compressive strengths greater than 69 MPa, it is prudent to limit $\sqrt{f_c'}$ to 0.69 MPa for the calculation of shear strength.

R22.6.4 *Critical sections for two-way members*

The critical section defined in 22.6.4.1(a) for shear in slabs and footings subjected to bending in two directions follows the perimeter at the edge of the loaded area (Joint ACI-ASCE Committee 326 1962). Loaded area for shear in two-way slabs and footings includes columns, concentrated loads, and reaction areas. An idealized critical section located a distance $d/2$ from the periphery of the loaded area is considered.

For members of uniform thickness without shear reinforcement, it is sufficient to check shear using one section. For slabs with changes in thickness, it is necessary to check shear at multiple sections as defined in 22.6.4.1(a) and (b).

For columns near an edge or corner, the critical perimeter may extend to the edge of the slab.

R22.6.4.3 Provisions for design of openings in slabs (and footings) were developed in Joint ACI-ASCE Committee 326 (1962). The locations of the effective portions of the critical section near typical openings and free edges are shown by the dashed lines in Fig. R22.6.4.3. Research (Joint ACI-ASCE Committee 426 1974) has confirmed that these provisions are conservative.

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22.6.5 *Two-way shear strength provided by concrete*

22.6.5.1 For two-way members, v_c shall be calculated in accordance with 22.6.5.2.

22.6.5.2 v_c shall be calculated in accordance with Eq. (22.6.5.2a) and (22.6.5.2b).

$$v_c = 0.83\lambda_s k_{cr}\sqrt{f_c'} \quad (22.6.5.2a)$$

but v_c need not be less than

$$v_c = 0.13\lambda_s\sqrt{f_c'} \quad (22.6.5.2b)$$

where k_{cr} is the ratio of the elastic cracked transformed section neutral axis depth to the effective depth and λ_s is the size effect factor as given in Table 22.5.5.1.3.

COMMENTARY

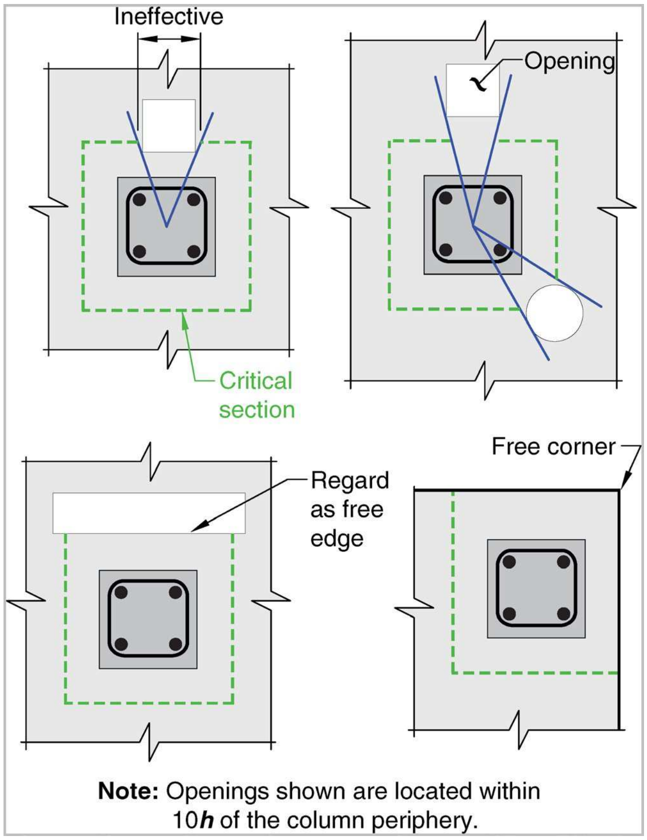

Fig. R22.6.4.3—Effect of openings and free edges (effective perimeter shown with dashed lines).

R22.6.5 *Two-way shear strength provided by concrete*

R22.6.5.2 Equation (22.6.5.2a) is the basic ACI 318 concentric punching shear equation for steel- reinforced concrete slabs, multiplied by the factor $2.5k_{cr}$, which accounts for the axial stiffness of the GFRP reinforcement.

Experimental evidence (Matthys and Taerwe 2000; El-Ghandour et al. 2003; Ospina et al. 2003; Dulude et al. 2013) shows that the axial stiffness of the GFRP reinforcement, as well as the concrete strength, significantly affect the concentric punching shear response of interior GFRP-reinforced concrete two-way slabs. Test results of isolated GFRP-reinforced concrete two-way slab specimens subjected to uniform gravity loading indicate that an increase in the top GFRP mat stiffness increases punching shear capacity and decreases the ultimate slab deflection. A statistical evaluation of test results reveal that the one-way shear design model proposed by Tureyen and Frosch (2003), which accounts for reinforcement stiffness, can be modified (Ospina 2005) to account for the shear transfer in two-way concrete slabs. The modification leads to Eq. (22.6.5.2a), which can be used to calculate the concentric punching shear

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capacity of GFRP-reinforced concrete two-way concrete slabs that are either supported by interior columns or subjected to concentrated loads that are either square or circular in shape. Experimental evidence has shown that Eq. (22.6.5.2a) can be applied to two-way concrete slabs supported by edge columns (El-Gendy and El-Salakawy 2020).

As discussed in R22.5.5.1 for one-way shear, Eq. (22.6.5.2a) may lead to unreasonably low estimates of shear capacity for lightly reinforced concrete members such as slabs and foundations and, thus, Eq. (22.6.5.2b) provides a lower limit on the shear capacity of the concrete. In effect, Eq. (22.6.5.2b) provides a lower bound of 0.16 on k_{cr} in Eq. (22.6.5.2a) (Nanni et al. 2014).

The parameter k_{cr} is the ratio of the depth of the elastic neutral axis to the longitudinal reinforcement depth and may be evaluated for slabs using the expression developed for rectangular sections in Eq. (R22.5.5.1a), with ρ_f equal to the average of the slab reinforcement ratios calculated across the width defined by the critical punching shear perimeter (El-Gendy and El-Salakawy 2020).

Experimental evidence indicates that the measured concrete shear strength of two-way members without shear reinforcement does not increase in direct proportion with member thickness. This phenomenon is referred to as the "size effect". The modification factor λ_s accounts for the dependence of the two-way shear strength of slabs on effective depth. For steel-reinforced concrete two-way slabs with $d > 250$ mm, the size effect defined in Table 22.5.5.1.3 reduces the shear strength of two-way slabs below the traditional value of $0.33\sqrt{f_c'}\,b_o d$ (Hawkins and Ospina 2017; Dönmez and Bažant 2017). A similar trend is expected for GFRP-reinforced concrete two-way slabs, with the shear strength decreasing below $0.83k_{cr}\sqrt{f_c'}$ for GFRP-reinforced concrete slabs with increasing thickness.

22.6.5.3 Intentionally left blank.

22.6.5.4 Intentionally left blank.

22.6.5.5 Intentionally left blank.

22.6.6 *Maximum shear for two-way members with shear reinforcement*—Out of scope

22.6.7 *Two-way shear strength provided by single- or multiple-leg stirrups*—Out of scope

22.6.8 *Two-way shear strength provided by headed shear stud reinforcement*—Out of scope

22.6.9 *Design provisions for two-way members with shearheads*—Out of scope

22.7—Torsional strength

R22.7—Torsional strength

The design for torsion in this section is based on a thin-walled tube space truss analogy. A beam subjected to torsion

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is idealized as a thin-walled tube with the core concrete cross section in a solid beam neglected as shown in Fig. R22.7(a). Once a reinforced concrete beam has cracked in torsion, its torsional strength is provided primarily by closed stirrups and longitudinal bars located near the surface of the member. In the thin-walled tube analogy, the strength is assumed to be provided by the outer skin of the cross section roughly centered on the closed stirrups.

In a closed thin-walled tube, the product of the shear stress τ and the wall thickness t at any point in the perimeter is known as the shear flow, $q = \tau t$. The shear flow q due to torsion acts as shown in Fig. R22.7(a) and is constant at all points around the perimeter of the tube. The path along which it acts extends around the tube at midthickness of the walls of the tube. At any point along the perimeter of the tube, the shear stress due to torsion is $\tau = T/(2A_o t)$, where A_o is the gross area enclosed by the shear flow path, shown shaded in Fig. R22.7(b), and t is the thickness of the wall at the point where τ is being calculated.

The concrete contribution to torsional strength is ignored, and in cases of combined shear and torsion, the concrete contribution to shear strength does not need to be reduced. The design procedure is derived and compared with steel-reinforced concrete test results in MacGregor and Ghoneim (1995) and Hsu (1997), and confirmed for GFRP-reinforced concrete in Mohamed and Benmokrane (2015).

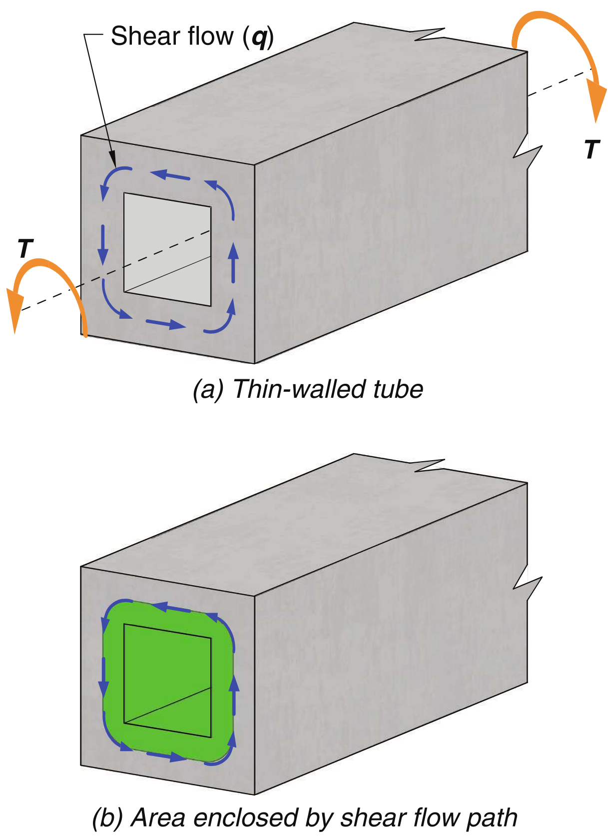

Fig. R22.7—(a) Thin-walled tube; and (b) area enclosed by shear flow path.

CODE

22.7.1 *General*

22.7.1.1 This section shall apply to solid members if $T_u \geq \phi T_{th}$, where ϕ is given in Chapter 21 and threshold torsion T_{th} is given in 22.7.4. If $T_u < \phi T_{th}$, it shall be permitted to neglect torsional effects.

22.7.1.2 Nominal torsional strength in solid members shall be calculated in accordance with 22.7.6.

22.7.2 *Limiting material strengths*

22.7.2.1 The value of $\sqrt{f_c'}$ used to calculate T_{th} and T_{cr} shall not exceed 0.69 MPa.

22.7.2.2 The value of f_{ft} for transverse torsional reinforcement shall not exceed the limits in 20.2.2.6.

22.7.3 *Factored design torsion*

22.7.3.1 If $T_u \geq \phi T_{cr}$ and T_u is required to maintain equilibrium, the member shall be designed to resist T_u.

22.7.3.2 In a statically indeterminate structure where $T_u \geq \phi T_{cr}$ and a reduction of T_u can occur due to redistribution of internal forces after torsional cracking, it shall be permitted to reduce T_u to ϕT_{cr}, where the cracking torsion T_{cr} is calculated in accordance with 22.7.5.

22.7.3.3 If T_u is redistributed in accordance with 22.7.3.2, the factored moments and shears used for design of the adjoining members shall be in equilibrium with the reduced torsion.

COMMENTARY

R22.7.1 *General*

R22.7.1.1 Torsional moments that do not exceed the threshold torsion T_{th} will not cause a structurally significant reduction in either flexural or shear strength and can be ignored. This Code does not address hollow members in torsion, other than to define the threshold torsion below which torsional effects can be neglected.

R22.7.2 *Limiting material strengths*

R22.7.2.1 Because of a lack of test data and practical experience with concretes having compressive strengths greater than 69 MPa, the Code imposes a maximum value of 0.69 MPa on $\sqrt{f_c'}$ for use in the calculation of torsional strength.

R22.7.2.2 The stress level in the transverse torsional reinforcement is limited to control diagonal crack widths at service loads and to avoid failure at the bent portion of the GFRP stirrup (Mohamed and Benmokrane 2015), similar to what is required for shear. Refer to R22.5.3.3.

R22.7.3 *Factored design torsion*

In designing for torsion in reinforced concrete structures, two conditions may be identified (Collins and Lampert 1973; Hsu and Burton 1974):

(a) The torsional moment cannot be reduced by redistribution of internal forces (22.7.3.1). This type of torsion is referred to as equilibrium torsion because the torsional moment is required for the structure to be in equilibrium.

For this condition, illustrated in Fig. R22.7.3(a), torsional reinforcement must be provided to resist the total design torsional moments.

(b) The torsional moment can be reduced by redistribution of internal forces after cracking (22.7.3.2) if the torsion results from the member twisting to maintain compatibility of deformations. This type of torsion is referred to as compatibility torsion. The force redistribution results from cracking of the concrete and does not depend on the ability of the reinforcement to yield.

For this condition, illustrated in Fig. R22.7.3(b), the torsional stiffness before cracking corresponds to that of the uncracked section according to St. Venant's theory. At torsional cracking, however, a large twist occurs under an essentially constant torsional moment, resulting in a large redistribution of forces in the structure (Collins and Lampert 1973; Hsu and Burton 1974). The cracking torsional moment under combined shear, moment, and torsion corresponds to a principal tensile stress somewhat less than the $\mathbf{0.33}\sqrt{f_c'}$ used in R22.7.5.

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If the torsional moment exceeds the cracking torsional moment (22.7.3.2), a maximum factored torsional moment equal to the cracking torsional moment may be assumed to occur at the critical sections near the faces of the supports. The maximum factored torsional moment has been established to limit the width of torsional cracks.

Provision 22.7.3.2 applies to typical and regular framing conditions. With layouts that impose significant torsional rotations within a limited length of the member, such as a large torsional moment located close to a stiff column, or a column that rotates in the reverse directions because of other loading, a more detailed analysis is advisable.

If the factored torsional moment from an elastic analysis based on uncracked section properties is between ϕT_{th} and ϕT_{cr}, torsional reinforcement should be designed to resist the calculated torsional moments.

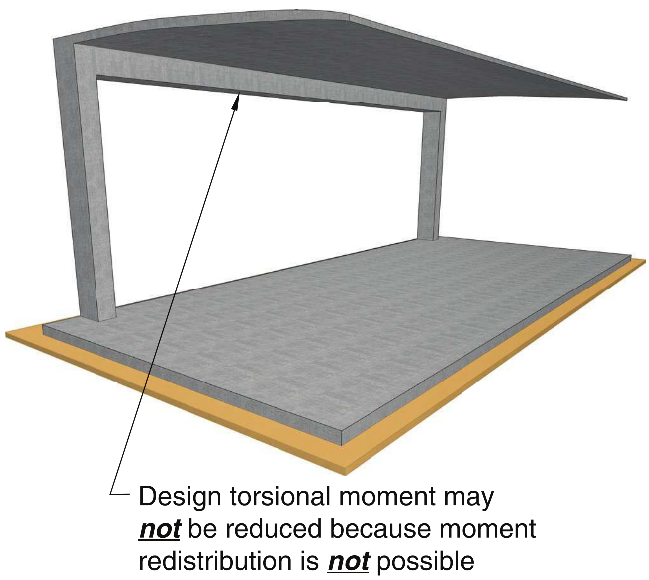

Fig. R22.7.3a—Equilibrium torsion, the design torsional moment may not be reduced (22.7.3.1).

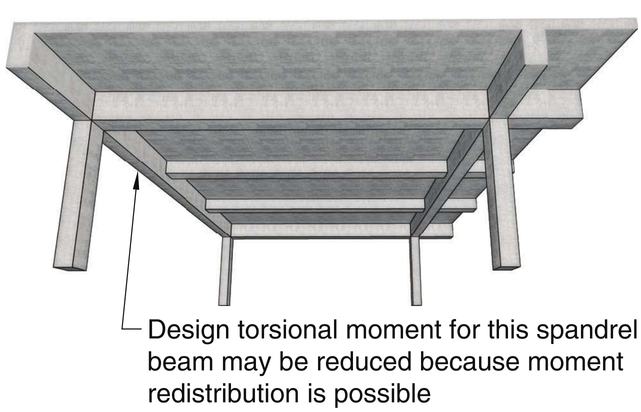

Fig. R22.7.3b—Compatibility torsion, the design torsional moment may be reduced (22.7.3.2).

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22.7.4 *Threshold torsion*

22.7.4.1 Threshold torsion T_{th} shall be calculated in accordance with Table 22.7.4.1a for solid cross sections and Table 22.7.4.1b for hollow cross sections, where N_u is positive for compression and negative for tension.

Table 22.7.4.1a—Threshold torsion for solid cross sections

Type of member	T_{th}	
Member not subjected to axial force	$0.083\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$	(a)
Member subjected to axial force	$0.083\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{0.33A_g\sqrt{f_c'}}}$	(b)

Table 22.7.4.1b—Threshold torsion for hollow cross sections

Type of member	T_{th}	
Member not subjected to axial force	$0.083\sqrt{f_c'}\left(\frac{A_g^2}{p_{cp}}\right)$	(a)
Member subjected to axial force	$0.083\sqrt{f_c'}\left(\frac{A_g^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{0.33A_g\sqrt{f_c'}}}$	(b)

22.7.5 *Cracking torsion*

22.7.5.1 Cracking torsion T_{cr} shall be calculated in accordance with Table 22.7.5.1 for solid cross sections, where N_u is positive for compression and negative for tension.

COMMENTARY

R22.7.4 *Threshold torsion*

The threshold torsion is defined as one-fourth the cracking torsional moment T_{cr}. For sections of solid members, the interaction between the cracking torsional moment and the inclined cracking shear is approximately circular or elliptical. For such a relationship, a threshold torsional moment of T_{th}, as used in 22.7.4.1, corresponds to a reduction of less than 5% in the inclined cracking shear, which is considered negligible.

For torsion, a hollow section is defined as having one or more longitudinal voids, such as a single-cell or multiple-cell box girder. Small longitudinal voids, that result in $A_g/A_{cp} \geq 0.95$, can be ignored when calculating T_{th}. The interaction between torsional cracking and shear cracking for hollow sections is assumed to vary from the elliptical relationship for members with small voids, to a straight-line relationship for thin-walled sections with large voids. For a straight-line interaction, a torsional moment of T_{th} would cause a reduction in the inclined cracking shear of approximately 25%, which was considered significant. Therefore, the expressions for solid sections are modified by the factor $(A_g/A_{cp})^2$ to develop the expressions for hollow sections. Tests of solid and hollow beams (Hsu 1968) indicate that the cracking torsional moment of a hollow section is approximately (A_g/A_{cp}) times the cracking torsional moment of a solid section with the same outside dimensions. An additional multiplier of (A_g/A_{cp}) reflects the transition from the circular interaction between the inclined cracking loads in shear and torsion for solid members, to the approximately linear interaction for thin-walled hollow sections.

R22.7.5 *Cracking torsion*

The cracking torsional moment under pure torsion, T_{cr}, is derived by replacing the actual section with an equivalent thin-walled tube with a wall thickness t prior to cracking of $0.75A_{cp}/p_{cp}$ and an area enclosed by the wall centerline A_o equal to $2A_{cp}/3$. Cracking is assumed to occur when the principal tensile stress reaches $0.33\sqrt{f_c'}$. The stress at cracking, $0.33\sqrt{f_c'}$, has purposely been taken as a lower bound value. In a nonprestressed beam loaded with torsion alone, the principal tensile stress is equated to the torsional shear stress, $\tau = T/(2A_o t)$. Thus, cracking occurs when τ reaches $0.33\sqrt{f_c'}$, giving the cracking torsional moment T_{cr} as defined by expression (a) in Table 22.7.5.1.

If the factored torsional moment exceeds ϕT_{cr} in a statically indeterminate structure, a maximum factored torsional moment equal to ϕT_{cr} may be assumed to occur at critical sections near the faces of the supports. This limit has been established to control the width of the torsional cracks.

R22.7.5.1 Due to a lack of published research, this Code does not address hollow members in torsion, other than to define the threshold torsion below which torsional effects can be neglected.

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Table 22.7.5.1—Cracking torsion

Type of member	T_{cr}	
Member not subjected to axial force	$0.33\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$	(a)
Member subjected to axial force	$0.33\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{0.33A_g\sqrt{f_c'}}}$	(c)

22.7.6 *Torsional strength*

22.7.6.1 T_n shall be the lesser of (a) and (b):

(a) $$T_n = \frac{2A_oA_{ft}f_{ft}}{s} \quad (22.7.6.1a)$$

(b) $$T_n = \frac{2A_oA_{fl}f_{fu}}{p_h} \quad (22.7.6.1b)$$

where A_o shall be determined by analysis, A_{ft} is the area of one leg of a closed stirrup resisting torsion; A_{ft} is the area of longitudinal torsional reinforcement; and p_h is the perimeter of the centerline of the outermost closed stirrup.

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R22.7.6 *Torsional strength*

The torsional design strength ϕT_n must equal or exceed the torsional moment T_u due to factored loads. In the calculation of T_n, all the torsion is assumed to be resisted by stirrups and longitudinal reinforcement, neglecting any concrete contribution to torsional strength. At the same time, the nominal shear strength provided by concrete, V_c, is assumed to be unchanged by the presence of torsion.

R22.7.6.1 Equation (22.7.6.1a) is based on the space truss analogy shown in Fig. R22.7.6.1a with compression diagonals at an angle of 45 degrees (Mohamed and Benmokrane 2015), assuming the concrete resists no tension. After torsional cracking develops, the torsional strength is provided mainly by closed stirrups, longitudinal reinforcement, and compression diagonals. The concrete outside these stirrups is relatively ineffective. For this reason, A_o, the gross area enclosed by the shear flow path around the perimeter of the tube, is defined after cracking in terms of A_{oh}, the area enclosed by the centerline of the outermost closed transverse torsional reinforcement.

The shear flow q in the walls of the tube, discussed in R22.7, can be resolved into the shear forces V_1 to V_4 acting in the individual sides of the tube or space truss, as shown in Fig. R22.7.6.1a.

As shown in Fig. R22.7.6.1b, on a given wall of the tube, the shear flow V_i is resisted by a diagonal compression component, $D_i = V_i/\sin 45$, in the concrete. An axial tension force, $N_i = V_i(\cot 45)$, is required in the longitudinal reinforcement to complete the resolution of V_i.

Because the shear flow due to torsion is constant at all points around the perimeter of the tube, the resultants of D_i and N_i act through the midheight of side i. As a result, half of N_i can be assumed to be resisted by each of the top and bottom chords as shown. Longitudinal reinforcement with a strength $A_{fl}f_{fu}$ is required to resist the sum of the N_i forces, $\sum N_i$, acting in all the walls of the tube.

In the derivation of Eq. (22.7.6.1b), axial tension forces are summed along the sides of the area A_o. These sides form a perimeter length p_o approximately equal to the length of the line joining the centers of the bars in the corners of the tube. For ease in calculation, this has been replaced with the perimeter of the closed stirrups, p_h.

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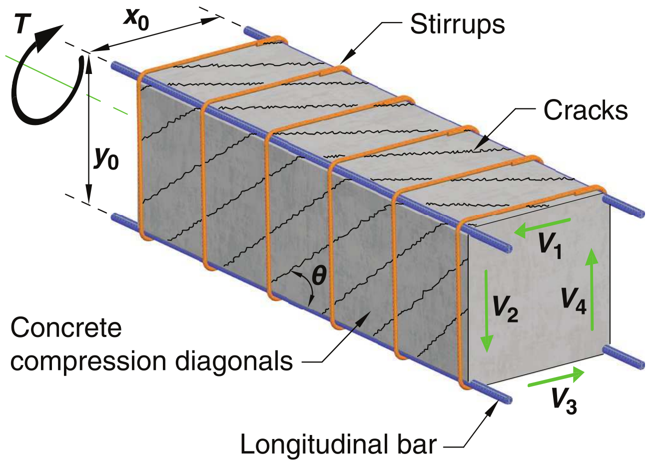

Fig. R22.7.6.1a—Space truss analogy.

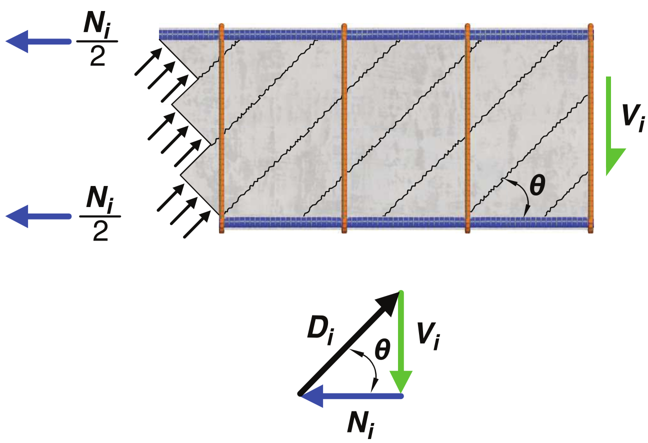

Fig. R22.7.6.1b—Resolution of shear force $\mathbf{V_i}$ *into diagonal compression force* $\mathbf{D_i}$ *and axial tension force* $\mathbf{N_i}$ *in one wall of tube.*

=**22.7.6.1.1** In Eq. (22.7.6.1a) and (22.7.6.1b), it shall be permitted to take A_o equal to $\mathbf{0.85}A_{oh}$.

R22.7.6.1.1 The area A_{oh} is shown in Fig. R22.7.6.1.1 for various cross sections. In I-, T-, L-shaped, or circular sections, A_{oh} is taken as that area enclosed by the outermost transverse reinforcement.

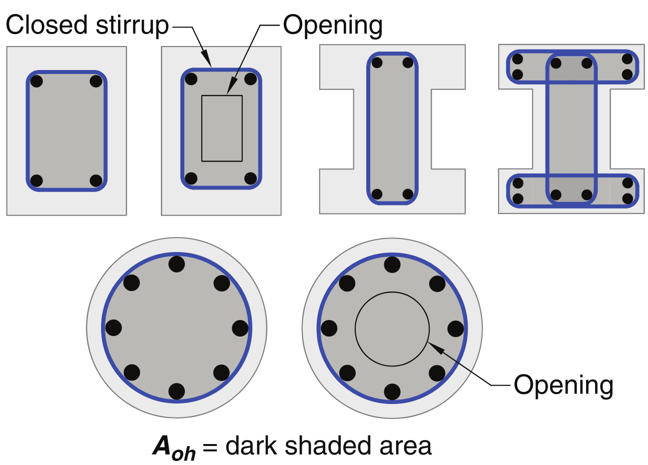

Fig. R22.7.6.1.1—Definition of $\mathbf{A_{oh}}$.

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22.7.7 *Cross-sectional limits*

22.7.7.1 Cross-sectional dimensions shall be selected such that for solid sections:

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u p_h}{1.7A_{oh}^2}\right)^2} \leq \phi(0.2 f_c') \qquad (22.7.7.1)$$

22.7.7.2 Intentionally left blank.

22.8—Bearing

22.8.1 *General*

22.8.1.1 This section shall apply to the calculation of bearing strength of concrete members.

COMMENTARY

R22.7.7 *Cross-sectional limits*

R22.7.7.1 The size of a cross section is limited to minimize the potential for crushing of the web concrete due to inclined compressive stresses from shear and torsion. In Eq. (22.7.7.1), the two terms on the left-hand side are the shear stresses due to shear and torsion. The sum of these stresses may not exceed the limit intended to control web crushing, similar to the limiting strength given in 22.5.1.2 for shear without torsion. In a solid section, the shear stresses due to torsion act in the tubular outside section while the shear stresses due to V_u are spread across the width of the section, as shown in Fig. R22.7.7.1. For this reason, stresses are combined in Eq. (22.7.7.1) using the square root of the sum of the squares rather than by direct addition. Limiting the strain in the GFRP shear reinforcement to control diagonal cracking is addressed in 22.7.2.2.

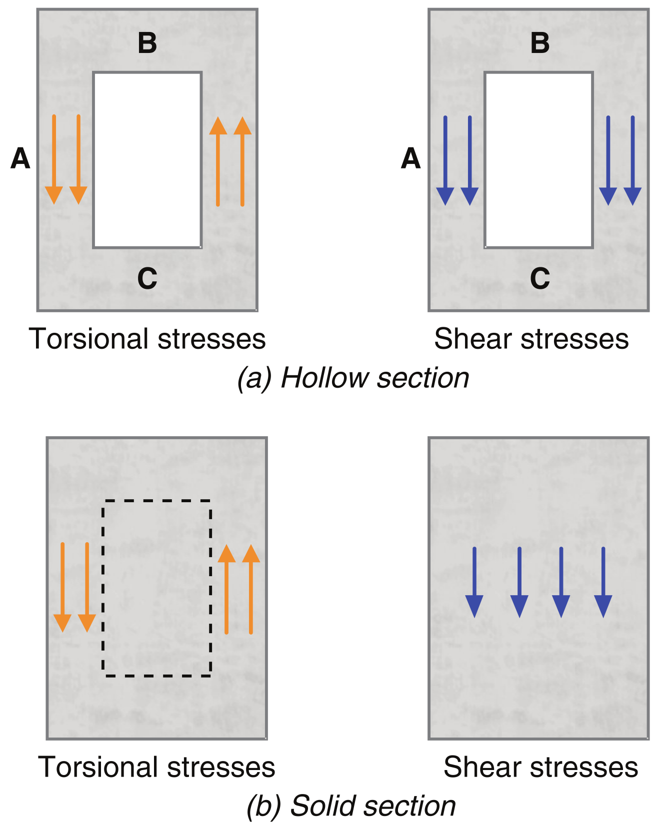

Fig. R22.7.7.1—Addition of torsional and shear stresses.

R22.8—Bearing

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22.8.2 *Required strength*

22.8.2.1 Factored compressive force transferred through bearing shall be calculated in accordance with the factored load combinations defined in Chapter 5 and analysis procedures defined in Chapter 6.

22.8.3 *Design strength*

22.8.3.1 Design bearing strength shall satisfy:

$$\phi B_n \geq B_u \tag{22.8.3.1}$$

for each applicable factored load combination.

22.8.3.2 Nominal bearing strength, B_n, shall be calculated in accordance with Table 22.8.3.2, where A_1 is the loaded area and A_2 is the area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having its upper base equal to the loaded area. The sides of the pyramid, cone, or tapered wedge shall be sloped 1 vertical to 2 horizontal.

Table 22.8.3.2—Nominal bearing strength

Geometry of bearing area	B_n		
Supporting surface is wider on all sides than the loaded are	Lesser of (a) and (b)	$\sqrt{\frac{A_2}{A_1}}\left(0.85f_c'(A_1)\right)$	(a)
		$2(0.85f_c'(A_1))$	(b)
Other cases	$0.85f_c'(A_1)$		(c)

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R22.8.3 *Design strength*

R22.8.3.2 The permissible bearing stress of $\mathbf{0.85}\boldsymbol{f_c'}$ is based on tests reported in (Hawkins 1968). Where the supporting area is wider than the loaded area on all sides, the surrounding concrete confines the bearing area, resulting in an increase in bearing strength. No minimum depth is given for the support, which will most likely be controlled by the punching shear requirements of 22.6.

A_1 is the loaded area but not greater than the bearing plate or bearing cross-sectional area.

Where the top of the support is sloped or stepped, advantage may still be taken of the condition that the supporting member is larger than the loaded area, provided the supporting member does not slope at too great an angle. Figure R22.8.3.2 illustrates the application of the frustum to find A_2 for a support under vertical load transfer.

Adequate bearing strength needs to be provided for cases where the compression force transfer is in a direction other than normal to the bearing surface. For such cases, this section applies to the normal component and the tangential component needs to be transferred by other methods, such as by anchor bolts or shear lugs.

The frustum should not be confused with the path by which a load spreads out as it progresses downward through the support. Such a load path would have steeper sides. However, the frustum described has somewhat flat side slopes to ensure that there is concrete immediately surrounding the zone of high stress at the bearing.

Where tensile forces occur in the plane of bearing, it may be desirable to reduce the allowable bearing stress, provide confinement reinforcement, or both. Guidelines are provided in the *PCI Design Handbook* for precast and prestressed concrete (PCI MNL 120-4).

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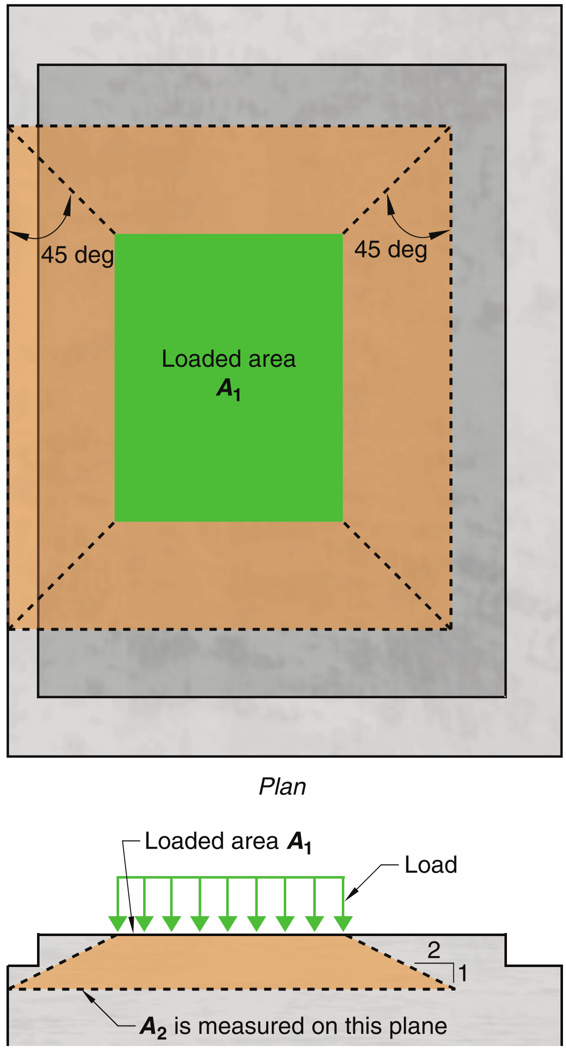

Fig. R22.8.3.2—Application of frustum to find $\mathbf{A_2}$ *in stepped or sloped supports.*

22.9—Shear friction—Out of scope

R22.9—Shear friction—Out of scope

Shear friction is not covered in this Code due to a lack of sufficient published research on shear friction in GFRP-reinforced concrete members. The factors for shear friction in steel-reinforced concrete members are primarily empirically determined. These factors have not yet been determined when GFRP reinforcement is crossing the potential slip plane. Information on shear friction in GFRP-reinforced concrete members is provided in Alkatan (2016).

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CHAPTER 23—STRUT-AND-TIE METHOD—NOT ADDRESSED

COMMENTARY

CHAPTER R23—STRUT-AND-TIE METHOD—NOT ADDRESSED

This Code does not cover strut-and-tie models. As formulated for steel-reinforced concrete, strut-and-tie models satisfy the lower-bound theory of plasticity and are therefore most appropriate when considering elastic-perfectly plastic behavior. These models are generally not appropriate for assessing behavior in the elastic realm and are therefore inappropriate for use with GFRP bars. Some design provisions, such as the Canadian Standard S806-12, permit strut-and-tie modeling with significant limitations.

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CHAPTER 24—SERVICEABILITY REQUIREMENTS

24.1—Scope

24.1.1 This chapter shall apply to member design for minimum serviceability, including (a) through (d):

(a) Deflections due to service-level gravity loads (24.2)

(b) Distribution of flexural reinforcement in one-way slabs and beams to control cracking (24.3)

(c) Shrinkage and temperature reinforcement (24.4)

(d) Permissible tensile stresses in GFRP reinforcement (24.6)

24.2—Deflections due to service-level gravity loads

=**24.2.1** Members subjected to flexure shall be designed with adequate stiffness to limit deflections or deformations that adversely affect strength or serviceability of a structure.

CHAPTER R24—SERVICEABILITY REQUIREMENTS

R24.1—Scope

This chapter prescribes serviceability requirements that are referenced by other chapters of the Code, or are otherwise applicable to provide adequate performance of structural members. This chapter does not stand on its own as a complete and cohesive compilation of serviceability requirements for the design of structural members. This chapter has no specific requirements for vibrations.

Steel-reinforced cast-in-place floor systems designed in accordance with the minimum thickness and deflection requirements of Sections 7.3, 8.3, 9.3, and 24.2 of ACI 318 have generally been found, through experience, to provide vibration performance suitable for human comfort under typical service conditions. However, there may be situations where serviceability conditions are not satisfied—for example:

(a) Long spans and open floor plans

(b) Floors with strict vibration performance requirements such as precision manufacturing and laboratory spaces

(c) Facilities subject to rhythmic loadings or vibrating mechanical equipment

GFRP-reinforced concrete and steel-reinforced concrete members will have similar vibrational characteristics. Given that the deflection limits in this Code are identical to those in ACI 318 and that steel reinforcement behaves elastically at service load levels, flexural stiffness, mass, and damping behavior will be similar for GFRP-reinforced concrete and steel-reinforced concrete under the same boundary and loading conditions.

Guidance on the consideration of vibrations in the design of floor systems and the evaluation of vibration frequency and amplitude for steel-reinforced concrete floor systems is contained in the *PCI Design Handbook* (PCI MNL 120), ATC Design Guide 1 (Applied Technology Council 1999), Mast (2001), Fanella and Mota (2014), and Wilford and Young (2006). An example application is described by West et al. (2008).

R24.1.1 Serviceability criteria such as deflections and crack control often govern the design of GFRP-reinforced concrete members (Nanni 1993a; Bischoff 2005; Veysey and Bischoff 2013).

R24.2—Deflections due to service-level gravity loads

This section is concerned only with deflections or deformations that may occur at service load levels. When time-dependent deflections are calculated, only the dead load and those portions of other loads that are sustained need be considered.

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Table 24.2.2—Maximum permissible calculated deflections

Member	Condition		Deflection to be considered	Deflection limitation
Flat roofs	Not supporting or attached to nonstructural elements likely to be damaged by large deflections		Immediate deflection due to maximum of L_r, S, and R	$\ell/180$[1]
Floors			Immediate deflection due to L	$\ell/360$
Roof or floors	Supporting or attached to nonstructural elements	Likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements, which is the sum of the time-dependent deflection due to all sustained loads and the immediate deflection due to any additional live load[2]	$\ell/480$[3]
		Not likely to be damaged by large deflections		$\ell/240$[4]

[1]Limit not intended to safeguard against ponding. Ponding shall be checked by calculations of deflection, including added deflections due to ponded water, and considering time-dependent effects of sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

[2]Time-dependent deflection shall be calculated in accordance with 24.2.4, but shall be permitted to be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be calculated on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

[3]Limit shall be permitted to be exceeded if measures are taken to prevent damage to supported or attached elements.

[4]Limit shall not exceed tolerance provided for nonstructural elements.

GFRP-reinforced concrete members tend to have larger deflections than steel-reinforced concrete members of similar size, shape, and reinforcement ratio because of the lower stiffness associated with commercially available GFRP reinforcement (Bakis et al. 2002). Therefore, this Code does not permit control of deflections by satisfying minimum thickness requirements. Estimated deflections must be computed and compared to limiting values. However, guidance for minimum thicknesses to aid in establishing initial member proportions in the design process are available (ACI 440.1R; Veysey and Bischoff 2011, 2013). Member dimensions may need to be revised based on the limits of calculated deflections.

Deflections are required to be calculated by 24.2.3 through 24.2.5. Calculated deflections are limited to the values in Table 24.2.2.

24.2.2 Deflections calculated in accordance with 24.2.3 through 24.2.5 shall not exceed the limits in Table 24.2.2.

R24.2.2 It should be noted that the limitations given in Table 24.2.2 relate only to supported or attached nonstructural elements. For those structures in which structural members are likely to be affected by deflection or deformation of members to which they are attached in such a manner as to affect adversely the strength of the structure, these deflections and the resulting forces should be considered explicitly in the analysis and design of the structures as required by 24.2.1 (ACI 209R-92).

When time-dependent deflections are calculated, the portion of the deflection before attachment of the nonstructural elements may be deducted. In making this correction, use may be made of the curve in Fig. R24.2.4.1 for members of usual sizes and shapes.

24.2.3 *Calculation of immediate deflections*

R24.2.3 *Calculation of immediate deflections*

24.2.3.1 Immediate deflections shall be calculated using methods or formulas for elastic deflections, considering effects of cracking and reinforcement on member stiffness.

R24.2.3.1 For calculation of immediate deflections of uncracked prismatic members, the usual methods or formulas for elastic deflections may be used with a constant value of E_cI_g along the length of the member. However, if the member is expected to crack at one or more sections, or

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if its depth varies along the span, a more rigorous calculation becomes necessary.

=**24.2.3.2** Effect of variation of cross-sectional properties, such as haunches, shall be considered when calculating deflections.

=**24.2.3.3** Deflections in two-way slab systems shall be calculated taking into account size and shape of the panel, conditions of support, and nature of restraints at the panel edges.

R24.2.3.3 The calculation of deflections for two-way slabs is challenging even if linear elastic behavior can be assumed. For immediate deflections, the values of E_c and I_e specified in 24.2.3.4 and 24.2.3.5, respectively, may be used in lieu of a more refined procedure. As an approximation, I_e may be taken as an average of values computed for the short and long directions of the slab, with the appropriate respective service moment M_a considered in each direction. Other procedures may be used if they result in predictions of deflection in reasonable agreement with the results of comprehensive tests.

=**24.2.3.4** Modulus of elasticity, E_c, shall be permitted to be calculated in accordance with 19.2.2.

24.2.3.5 Unless obtained by a more comprehensive analysis, effective moment of inertia, I_e, shall be calculated in accordance with Table 24.2.3.5 using

$$M_{cr} = \frac{f_r I_g}{y_t} \tag{24.2.3.5a}$$

$$\gamma = 1.72 - 0.72\left(\frac{0.8M_{cr}}{M_a}\right) \tag{24.2.3.5b}$$

Table 24.2.3.5—Effective moment of inertia, I_e

Service moment	Effective moment of inertia, I_e, in.^{4}
$M_a \leq 0.8M_{cr}$	I_g
$M_a > 0.8M_{cr}$	$I_e = \dfrac{I_{cr}}{1-\gamma\left(\dfrac{0.8M_{cr}}{M_a}\right)^2\left(1-\dfrac{I_{cr}}{I_g}\right)}$

R24.2.3.5 The overall flexural stiffness of a cracked member varies between E_cI_g and E_cI_{cr}, depending on the magnitude of the applied service moment and the extent of cracking along the member. Branson (1965) introduced the concept of an effective moment of inertia I_e to allow for a gradual transition from I_g to I_{cr}. This approach accounts for two different phenomena: the effect of concrete tension stiffening and the variation of E_cI along the member.

As demonstrated by Bischoff (2005), Branson's equation overestimates flexural stiffness if the I_g/I_{cr} ratio is greater than approximately 3 or 4. This corresponds to most GFRP-reinforced concrete flexural members that typically have an I_g/I_{cr} ratio between 5 and 25. It is for this reason that past research on deflection of GFRP-reinforced concrete beams (Yost et al. 2003) has shown that Branson's equation underestimates deflection, particularly for members with a high I_g/I_{cr} ratio.

The presented approach is equivalent to a weighted average of flexibility ($1/E_cI$) which better represents the deflection response of members with discrete cracks along their length (Bischoff and Scanlon 2007) and provides reasonable estimates of deflection for GFRP-reinforced concrete beams and one-way slabs (Bischoff et al. 2009).

The equation for effective moment of inertia in Table 24.2.3.5 is the section-based expression proposed by Bischoff (2005), modified to include an additional factor γ to account for the variation in stiffness as determined from the integration of curvature over the member length. The factor is dependent on load and boundary conditions and accounts for the length of the uncracked regions of the member and for the change in stiffness in the cracked regions. In place of a more comprehensive analysis, the value of γ resulting from integrating the curvature over the length of a simply supported beam with uniformly distributed load may be

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used as a reasonably conservative approximation for other support and loading conditions. This is the value given in Eq. (24.2.3.5a). Values for γ for simple and cantilever beams under other loading conditions are available in Bischoff and Gross (2011a).

M_{cr} is multiplied by a reduction factor to account for restraint that can reduce the effective cracking moment as well as to account for reduced tensile strength of concrete during construction that can lead to cracking that later affects service deflections (Scanlon and Bischoff 2008). As the lower stiffness of GFRP reinforcement provides less restraint to shrinkage than occurs with steel reinforcement, the reduction factor for GFRP reinforcement ($0.8M_{cr}$) is larger than for steel reinforcement ($2/3M_{cr}$) (Bischoff and Gross 2011b).

24.2.3.6 For slabs and beams with both ends continuous, I_e shall be permitted to be taken as the weighted average of values given by Eq. (24.2.3.6), where I_{e+}, I_{e1-}, and I_{e2-} are obtained from Table 24.2.3.5 for the critical positive and negative moment sections.

$$I_e = 0.70I_{e+} + 0.15(I_{e1-} + I_{e2-}) \qquad (24.2.3.6)$$

R24.2.3.6 For spans with both ends continuous, ACI 435R suggests that the effective moment of inertia for steel-reinforced concrete members may be approximated using the weighted average from Eq. (24.2.3.6). The validity of this method has been confirmed for GFRP-reinforced concrete (DeSimone 2009).

24.2.3.7 For prismatic one-way slabs and beams, I_e shall be permitted to be taken as the value obtained from Table 24.2.3.5 using the maximum service load moment M_a in the member.

R24.2.3.7 Although variation in member stiffness along the span influences member deflections, the deflection behavior of prismatic GFRP-reinforced concrete members is affected most by the section behavior at the location of the maximum moment along the span (DeSimone 2009). The member will crack first at this location and exhibit a very large decrease in stiffness (or increase in flexibility) because of the high I_g/I_{cr} ratios associated with GFRP-reinforced concrete. This large drop in stiffness has the most pronounced effect on the overall response.

24.2.3.8 Intentionally left blank.

24.2.3.9 Intentionally left blank.

24.2.4 *Calculation of time-dependent deflections*

R24.2.4 *Calculation of time-dependent deflections*

24.2.4.1 *GFRP-reinforced concrete members*

R24.2.4.1 *GFRP-reinforced concrete members*

24.2.4.1.1 Unless obtained from a more comprehensive analysis, additional time-dependent deflection resulting from creep and shrinkage of flexural members shall be calculated as the product of the immediate deflection caused by sustained load and the factor λ_Δ.

$$\lambda_\Delta = 0.6\xi \qquad (24.2.4.1.1)$$

24.2.4.1.2 Intentionally left blank.

24.2.4.1.3 In Eq. (24.2.4.1.1), values of the time-dependent factor for sustained loads, ξ, shall be in accordance with Table 24.2.4.1.3.

Shrinkage and creep cause time-dependent deflections in addition to the elastic deflections that occur when loads are first placed on the structure. Such deflections are influenced by temperature, humidity, curing conditions, age at time of loading, amount of compression reinforcement, and magnitude of the sustained load. The expression given in this section is considered satisfactory for use with the Code procedures for the calculation of immediate deflections, and with the limits given in Table 24.2.2. The deflection calculated in accordance with this section is the additional time-dependent deflection due to the dead load and those portions of other loads that will be sustained for a sufficient period to cause significant time-dependent deflections.

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Table 24.2.4.1.3—Time-dependent factor for sustained loads

Sustained load duration, months	Time-dependent factor ξ
3	1.0
6	1.2
12	1.4
60 or more	2.0

24.2.4.2 *Prestressed members*—Out of scope

24.2.5 *Calculation of deflections of composite concrete construction*

24.2.5.1 If composite concrete flexural members are shored during construction so that, after removal of temporary supports, the 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 calculation of deflections.

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Equation (24.2.4.1.1) is a modification to the equation developed in Branson (1971) and used in ACI 318. Research has shown that GFRP compression reinforcement does reduce deflections due to creep (Walkup et al. 2017), although not as effectively as does steel compression reinforcement. However, the long-term deflection multiplier given in Eq. (24.2.4.1.1) does not account for the presence of GFRP compression reinforcement because such reinforcement is not typically used in GFRP-reinforced concrete flexural members and because the decrease in creep due to the presence of GFRP in the compression zone is small. ξ = **2.0** represents a nominal time-dependent factor for a 5-year duration of loading for steel-reinforced concrete. The curve in Fig. R24.2.4.1 may be used to estimate values of ξ for loading periods less than 5 years. Experimental studies (Brown 1997; Gross et al. 2006; Hall and Ghali 2000; Youssef et al. 2009a,b; Mias et al. 2013a,b; Walkup et al. 2017) have shown that the time-dependent deflection, when considered as a multiple of the instantaneous deflection, is lower for GFRP-reinforced concrete than for steel-reinforced concrete. As a result, the values of the multiplier are reduced to 60% of ξ.

Because available data on time-dependent deflections of two-way slabs are too limited to justify more elaborate procedures, calculation of the additional time-dependent deflection for two-way construction in accordance with Eq. (24.2.4.1.1) is required to use the multipliers given in 24.2.4.1.3.

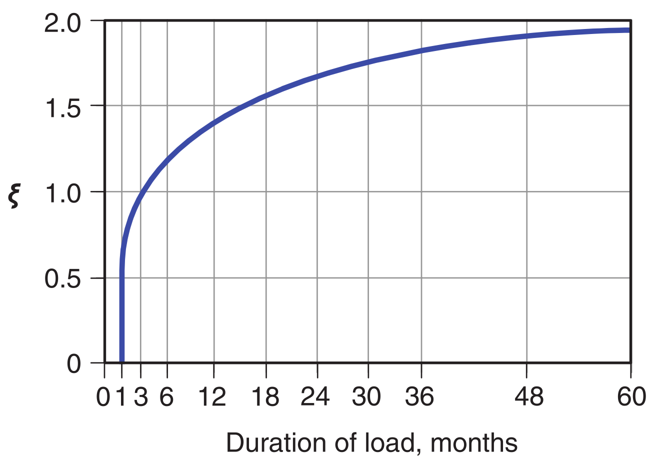

Fig. R24.2.4.1—Mulipliers for time-dependent deflections.

R24.2.5 *Calculation of deflections of composite concrete construction*

Composite concrete members are designed to meet the horizontal shear strength requirements of 16.4. Because few tests have been made to study the immediate and time-dependent deflections of composite members, the requirements given in this section are based on the judgment of ACI Committee 318 and on experience. In 22.3.3.3, it is

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=**24.2.5.2** If composite concrete flexural members are not shored during construction, the magnitude and duration of load before and after composite action becomes effective shall be considered in calculating time-dependent deflections.

24.2.5.3 Deflections resulting from differential shrinkage of precast and cast-in-place components shall be considered.

24.3—Distribution of GFRP flexural reinforcement in one-way slabs and beams

24.3.1 Reinforcement shall be distributed to control flexural cracking in tension zones of slabs and beams reinforced for flexure in one direction only.

24.3.2 Spacing of reinforcement closest to the tension face shall not exceed the limits in Equations (24.3.2a) and (24.3.2b), where c_c is the least distance from surface of reinforcement to the tension face. Calculated stress in reinforcement at service, f_{fs}, shall be in accordance with 24.3.2.1, and the bond factor k_b shall be in accordance with 24.3.2.3.

$$s \leq \frac{0.81E_f}{f_{fs}k_b} - 2.5c_c \tag{24.3.2a}$$

and not greater than

$$s \leq 0.66\frac{E_f}{f_{fs}k_b} \tag{24.3.2b}$$

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stated that distinction need not be made between shored and unshored members. This refers to strength calculations, not to deflections. Construction documents should indicate whether composite concrete design is based on shored or unshored construction, as required by 26.11.1.1.

R24.3—Distribution of GFRP flexural reinforcement in one-way slabs and beams

R24.3.1 Where service loads result in high strains in the reinforcement, visible cracks should be expected, and steps should be taken in detailing of the reinforcement to control cracking. For reasons of durability and appearance, many fine cracks are preferable to a few wide cracks. Detailing practices limiting bar spacing will usually lead to adequate crack control. Extensive laboratory work on steel-reinforced concrete (Gergely and Lutz 1968; Kaar 1966; Base et al. 1966) and GFRP-reinforced concrete (El-Nemr et al. 2013, 2016) has demonstrated that crack width at service loads is proportional to reinforcement strain. The significant variables reflecting reinforcement detailing were found to be thickness of concrete cover and the spacing of reinforcement. Bond characteristics of GFRP bars also affect crack width and spacing.

Crack width is inherently subject to wide scatter even in careful laboratory work and is influenced by shrinkage and other time-dependent effects. Research has shown that crack widths increase over time for members under sustained load (Gross et al. 2009). Improved crack control is obtained where the reinforcement is well distributed over the zone of maximum concrete tension. Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

R24.3.2 The spacing of reinforcement is limited to control cracking using a procedure developed by Ospina and Bakis (2007) based on modifications to the work done by Frosch (1999) for steel-reinforced concrete. Crack widths in structures are highly variable. The Code provisions for spacing are intended to limit surface cracks to a width that is generally acceptable in practice but may vary widely in a given structure. Selection of the limiting crack width depends on the intended use of the structure. For comparison purposes, the crack control provisions for steel reinforcement in ACI 318 correspond to a maximum crack width that is approximately 0.46 mm, regardless of exposure condition. From a practical perspective, acceptable crack widths in GFRP-reinforced concrete members may need to be larger than those in steel-reinforced concrete members. In situations where crack widths are limited for aesthetic reasons, limiting crack widths in the range of 0.41 to 0.71 mm is generally acceptable. The maximum bar spacing limits given in Eq. (24.3.2) are based on limiting crack width to 0.71 mm. In cases where the licensed design professional believes a more restrictive maximum allowable crack width is appropriate, the 0.032 and 0.026 coefficients may be linearly adjusted. Only tension reinforcement nearest the tension face need be considered in selecting the value of c_c used in calculating spacing requirements.

In situations where the maximum bar spacing limit given in Eq. (24.3.2) yields smaller than practical bar spacing for a given diameter bar, as may be the case for interior slabs, the licensed design professional should consider using a smaller diameter bar to provide the required area of reinforcement.

24.3.2.1 Stress f_{fs} in reinforcement closest to the tension face at service loads shall be calculated based on an elastic cracked section analysis using the unfactored service moment M_s.

24.3.2.2 The bar stress f_{fs} shall satisfy Eq. (24.3.2.2), where β_{cr} is the ratio of the distance from the elastic cracked section neutral axis to the extreme tension fiber to the distance from the elastic cracked section neutral axis to the centroid of the longitudinal tensile reinforcement. Calculation of stress in reinforcement at service, f_{fs}, shall be in accordance with 24.3.2.1, and the bond factor k_b shall be in accordance with 24.3.2.3.

$$f_{fs} \leq \frac{0.36E_f}{d_c\beta_{cr}k_b} \quad (24.3.2.2)$$

R24.3.2.2 Equation (24.3.2.2) is based on limiting the computed crack width to 0.028 in. In cases where the licensed design professional believes a more restrictive maximum allowable crack width is appropriate, the 0.014 coefficient may be linearly adjusted. If the limit on bar stress is not satisfied, then the reinforcement stress may have to be reduced by increasing the amount of tensile reinforcement, adjusting the cross-section dimensions, or changing the material properties.

24.3.2.3 The bond factor for GFRP reinforcing bars k_b shall be taken as 1.2.

R24.3.2.3 The bond factor k_b is a coefficient that accounts for the degree of bond between the GFRP bar and the surrounding concrete. Shield et al. (2019) found k_b values varied between 0.69 and 1.61 based on an examination of available crack width data in the literature. A k_b value of 1.2 was selected so that the crack widths would be no larger than 0.7 mm approximately 70% of the time for all GFRP bar surface types.

24.3.3 If there is only one bar nearest to the extreme tension face, the width of the extreme tension face shall not exceed s determined in accordance with Eq. (24.3.2).

24.3.4 If the flanges of a T-beam is in tension, the portion of the flexural tension reinforcement not located over the beam web shall be distributed within the lesser of the effective flange width, as defined in accordance with 6.3.2 and $\ell_n/10$. If $\ell_n/10$ controls, additional longitudinal reinforcement satisfying 24.4.3.1 shall be provided in the outer portions of the flange.

R24.3.4 In T-beams, distribution of the negative moment reinforcement for control of cracking should take into account two considerations: 1) wide spacing of the reinforcement across the full effective width of flange may cause some wide cracks to form in the slab near the web; and 2) close spacing near the web leaves the outer regions of the flange unprotected. The one-tenth limitation is to guard against a spacing that is too wide, with some additional reinforcement required to protect the outer portions of the flange.

For T-beams designed to resist negative moments due to gravity and wind loads, all tensile reinforcement required for strength is located within the lesser of the effective flange width and $\ell_n/10$. Common practice is to place more than half of the reinforcement over the beam web.

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24.3.5 The spacing of flexural reinforcement in one-way slabs and beams subject to fatigue or designed to be watertight shall be selected based on investigations and precautions specific to those conditions and shall not exceed the limits of 24.3.2.

24.4—GFRP shrinkage and temperature reinforcement

24.4.1 Reinforcement to resist shrinkage and temperature stresses shall be provided in one-way slabs in the direction perpendicular to the flexural reinforcement in accordance with 24.4.3.

24.4.2 If shrinkage and temperature movements are restrained, the effects of $\boldsymbol{T}$ shall be considered in accordance with 5.3.6.

24.4.3 *GFRP reinforcement*

24.4.3.1 Reinforcement to resist shrinkage and temperature stresses shall conform to 20.2.1.4 and shall be in accordance with 24.4.3.2 through 24.4.3.5.

24.4.3.2 The ratio of shrinkage and temperature reinforcement area to gross concrete area shall not be less than $\mathbf{140/}\boldsymbol{E_f}$.

R24.4—GFRP shrinkage and temperature reinforcement

R24.4.1 Shrinkage and temperature reinforcement is required at right angles to the principal reinforcement to minimize cracking and to tie the structure together to ensure it is acting as assumed in the design. The provisions of this section are intended for structural slabs only; they are not intended for slabs-on-ground.

R24.4.2 The area of shrinkage and temperature reinforcement required by 24.4.3.2 should be satisfactory where shrinkage and temperature movements are permitted to occur. Where structural walls or columns provide significant restraint to shrinkage and temperature movements, the restraint of volume changes causes tension in slabs, as well as displacements, shear forces, and flexural moments in columns or walls. In these cases, it may be necessary to increase the amount of slab reinforcement required by 24.4.3.2 due to the shrinkage and thermal effects in both principal directions (PCI MNL 120; Gilbert 1992). Top and bottom reinforcement are both effective in controlling cracks. Control strips during the construction period, which permit initial shrinkage to occur without causing an increase in stress, are also effective in reducing cracks caused by restraint.

Topping slabs also experience tension due to restraint of differential shrinkage between the topping and the precast elements or metal deck (which has zero shrinkage) that should be considered in reinforcing the slab. Consideration should be given to strain demands on reinforcement crossing joints of precast elements where most of the restraint is likely to be relieved.

R24.4.3 *GFRP reinforcement*

R24.4.3.2 The ratio of GFRP bar area to gross concrete area required by 24.4.3.2 provides the same force capacity as does the 0.0018 ratio required by ACI 318 for Grade 60 steel reinforcement and corresponds to a 0.7 mm estimated crack width at service load levels (Shield et al. 2019). The 0.0018 ratio for Grade 420 steel reinforcement is empirical but has been used satisfactorily for many years. The resulting area of GFRP reinforcement may be distributed near the top or bottom of the slab or may be distributed between the two faces of the slab as deemed appropriate for specific conditions.

24.4.3.3 The spacing of shrinkage and temperature reinforcement shall not exceed the lesser of $3h$ and 300 mm.

24.4.3.4 At all sections where required, reinforcement used to resist shrinkage and temperature stresses shall develop $0.006E_f$ in tension in accordance with 25.4.2.

R24.4.3.4 Splices and end anchorages of shrinkage and temperature reinforcement are to be designed to develop the tensile stress that corresponds to a 0.006 tensile strain in the GFRP reinforcement in accordance with Chapter 25. The strain that corresponds to a 0.028 in. crack width at service loads effectively limits the tensile strain at ultimate to 0.006 (Shield et al. 2019).

24.4.3.5 For one-way precast slabs and wall panels, shrinkage and temperature reinforcement is not required in the direction perpendicular to the flexural reinforcement if (a) through (c) are satisfied.

(a) Precast members are not wider than 3.7 m

(b) Precast members are not mechanically connected to cause restraint in the transverse direction

(c) Reinforcement is not required to resist transverse flexural stresses

R24.4.3.5 For precast concrete members not wider than 3.7 m, there is usually no need to provide reinforcement to withstand shrinkage and temperature stresses in the short direction. The 3.7 m width is less than that in which shrinkage and temperature stresses can build up to a magnitude requiring reinforcement. In addition, much of the shrinkage occurs before the members are tied into the structure. Once in the final structure, the members are usually not as rigidly connected transversely as monolithic concrete, thus, the transverse restraint stresses due to both shrinkage and temperature change are significantly reduced.

The waiver does not apply where reinforcement is required to resist flexural stresses, such as in thin flanges of precast single and double tees.

24.4.4 *Prestressed reinforcement*—Out of scope

24.5—Permissible stresses in prestressed concrete flexural members—Out of scope

R24.5—Permissible stresses in prestressed concrete flexural members—Out of scope

Specification of permissible stresses in prestressed concrete flexural members are not covered in this Code because this Code does not cover GFRP prestressed concrete members.

24.6—Permissible tensile stresses in GFRP reinforcement

R24.6—Permissible tensile stresses in GFRP reinforcement

The service load stress levels in GFRP reinforcement should be limited to avoid creep-rupture failure under sustained stresses. Because these stress levels will be within the elastic range of the member, the stresses can be computed through an elastic cracked section analysis.

24.6.1 Sustained stress $f_{fs,sus}$ in GFRP reinforcement closest to the tension face due to the sustained portion of service loads shall be calculated based on an elastic cracked section analysis using the unfactored sustained service moment $M_{s,sus}$.

R24.6.1 GFRP reinforcing bars subjected to a constant tension over time can suddenly fail after a time period called the endurance time. This phenomenon is known as creep rupture (or static fatigue). As the ratio of the sustained tensile stress to the short-term strength of the GFRP bar increases, endurance time decreases. The creep rupture endurance time can irreversibly decrease under sufficiently adverse conditions such as high temperature, ultraviolet radiation exposure, high alkalinity, wet and dry cycles, freezing-and-thawing cycles, and abrasion of the reinforcement at crack locations where partial bond slip occurs. Test methods for the experimental characterization of creep rupture behavior appear in JSCE (1997b) and ASTM D7337.

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To avoid failure of a GFRP-reinforced concrete member due to creep rupture, the stress limit given in 24.6.2 is imposed. For the case of a flexural member, the stress level in the reinforcement can be computed using Eq. (R24.6.1), where $\boldsymbol{M_{s,sus}}$ is equal to the unfactored moment due to all sustained loads (dead loads and the sustained portion of the live load). The cracked moment of inertia $\boldsymbol{I_{cr}}$, and the ratio of the depth of the elastic neutral axis to the effective depth, $\boldsymbol{k_{cr}}$, are computed for the cracked transformed section using an elastic analysis.

$$f_{fs,sus} = \frac{n_f d(1-k_{cr})}{I_{cr}} M_{s,sus} \qquad \text{(R24.6.1)}$$

24.6.2 GFRP reinforcement shall be proportioned such that $\boldsymbol{f_{fs,sus}}$ does not exceed $\boldsymbol{0.30f_{fu}}$.

R24.6.2 The value of safe sustained stress level recommended in ACI 440.1R for GFRP bars was selected based on studies conducted with first-generation GFRP bars. The technology has improved significantly in terms of fibers, resins, and manufacturing process. More recent tests on different size bars from a variety of manufacturers (Keller et al. 2017; Sayed-Ahmed et al. 2017; Benmokrane et al. 2019) show that the creep-rupture stress limit of GFRP bars is higher than the limit given in ACI 440.1R. The limit for a safe sustained stress level is set at 30% of $\boldsymbol{f_{fu}}$.

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CHAPTER 25—GFRP REINFORCEMENT DETAILS

25.1—Scope

25.1.1 This chapter shall apply to reinforcement details, including:

(a) Minimum spacing
(b) Standard hooks and crossties
(c) Development of reinforcement
(d) Splices
(e) Transverse reinforcement

25.1.2 Intentionally left blank.

25.2—Minimum spacing of GFRP reinforcement

25.2.1 For parallel reinforcement in a horizontal layer, clear spacing shall be at least the greatest of 25 mm, d_b, and $(4/3)d_{agg}$.

25.2.2 For parallel reinforcement placed in two or more horizontal layers, reinforcement in the upper layers shall be placed directly above reinforcement in the bottom layer with a clear spacing between layers of at least 25 mm.

⁼**25.2.3** For longitudinal reinforcement in columns, pedestals, struts, and boundary elements in walls, clear spacing between bars shall be at least the greatest of 38 mm, $1.5d_b$, and $(4/3)d_{agg}$.

25.2.4 Intentionally left blank.

25.2.5 Intentionally left blank.

25.2.6 Intentionally left blank.

25.2.7 Intentionally left blank.

25.2.8 Intentionally left blank.

25.2.9 Intentionally left blank.

25.2.10 Intentionally left blank.

25.3—Standard hooks, crossties, and minimum inside bend diameters

25.3.1 Standard hooks for the development of bars in tension shall conform to Table 25.3.1.

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CHAPTER R25—GFRP REINFORCEMENT DETAILS

R25.1—Scope

All provisions in the Code relating to bar diameter (and area) are based on the nominal dimensions of the reinforcement as given in ASTM D7957.

R25.1.1 In addition to the requirements in this chapter that affect detailing of reinforcement, detailing specific to particular members is given in the corresponding member chapters. Additional detailing associated with structural integrity requirements is covered in 4.10.

R25.2—Minimum spacing of GFRP reinforcement

The minimum limits are set to permit concrete to flow readily into spaces between bars and between bars and forms without honeycombs, and to ensure against concentration of bars on a line that may cause shear or shrinkage cracking. Use of nominal bar diameter to define minimum spacing permits a uniform criterion for all bar sizes. The size limitations on aggregates were translated to minimum spacing requirements, and are provided to promote proper encasement of reinforcement and to minimize honeycombing. The limitations associated with aggregate size need not be satisfied if, in the judgment of the licensed design professional, the workability and methods of consolidation of the concrete are such that the concrete can be placed without creating honeycombs or voids.

The development lengths required to achieve the design stress in the GFRP bars given in 25.4 are a function of the bar spacing and cover. As a result, it may be desirable to use larger than minimum bar spacing or cover in some cases.

R25.3—Standard hooks, crossties, and minimum inside bend diameters

R25.3.1 Standard bends in reinforcing bars are described in terms of the inside diameter of bend because the inside bend diameter is easier to measure than the radius of bend. Bends are incorporated during manufacturing of the GFRP bar and cannot be made after the resin in the bar is cured. The primary factors affecting the minimum bend diameter are manufacturability of the bend and avoidance of crushing the concrete inside the bend. Hooks in GFRP bars cannot

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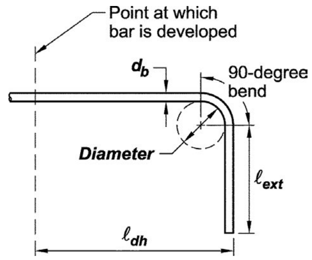
Table 25.3.1—Standard hook geometry for development of GFRP bars in tension

Type of standard hook	GFRP bar size	Minimum inside bend diameter, in.	Straight extension* ℓ_{ext}, in.	Type of standard hook
90-degree hook	No. M6 through No. M25	Refer to ASTM D7957	$12d_b$	

*A standard hook for bars in tension includes the inside bend diameter specified by ASTM D7957 and straight extension length defined in Table 25.3.1. It shall be permitted to use a longer straight extension at the end of a hook. A longer extension shall not be considered to increase the anchorage capacity of the hook.

plastically deform and unbend in use, hence bend angles greater than 90 degrees provide little if any improvement over 90-degree bends.

25.3.2 Minimum inside bend diameters for bars used as transverse reinforcement and standard hooks for bars used to anchor stirrups and ties shall conform to Table 25.3.1. Standard hooks shall enclose longitudinal reinforcement.

25.3.3 Intentionally left blank.

25.3.4 Intentionally left blank.

25.3.5 Crossties shall be in accordance with (a) through (c):

(a) Crosstie shall be continuous between ends

(b) There shall be a standard hook at both ends with bend of 90 degrees

(c) Hooks shall engage peripheral longitudinal bars

25.4—Development of GFRP reinforcement

25.4.1 *General*

25.4.1.1 Calculated tension or compression in reinforcement at each section of a member shall be developed on each side of that section by embedment length, hook, mechanical device, or a combination thereof.

R25.4—Development of GFRP reinforcement

R25.4.1 *General*

R25.4.1.1 The development length concept is based on the attainable average bond stress over the length of embedment of the reinforcement (ACI Committee 408 1966). Development lengths are required because of the tendency of highly stressed bars to split relatively thin sections of restraining concrete. A single bar embedded in a mass of concrete should not require as great a development length; although a row of bars, even in mass concrete, can create a weakened plane with longitudinal splitting along the plane of the bars.

In application, the development length concept requires minimum lengths or extensions of reinforcement beyond all points of peak stress in the reinforcement. Such peak stresses generally occur at the points of maximum stress and points where reinforcement is bent or terminated. From a point of peak stress in reinforcement, some length of reinforcement or anchorage is necessary to develop the stress. This development length or anchorage is necessary on both sides of such peak stress points. Often the reinforcement continues for a considerable distance on one side of a critical stress point so that calculations need involve only the other side, for example, the negative moment reinforcement continuing through a support to the middle of the next span.

25.4.1.2 Hooks shall not be used to develop bars in compression.

R25.4.1.2 Hooks are ineffective in compression. No data are available to demonstrate that hooks can reduce development length in compression.

25.4.1.3 Development lengths do not require a strength reduction factor ϕ.

R25.4.1.3 The strength reduction factor ϕ is not used in the development length and lap splice length equations.

25.4.1.4 The values of $\sqrt{f_c'}$ used to calculate development length shall not exceed 0.69 MPa.

R25.4.1.4 Darwin et al. (1996) show that the force developed in a deformed steel bar in development and lap splice tests increases at a lesser rate than $\sqrt{f_c'}$ with increasing compressive strength. Using $\sqrt{f_c'}$, however, is sufficiently accurate for values of $\sqrt{f_c'}$ up to 0.69 MPa, and because of the long standing use of the $\sqrt{f_c'}$ in design, ACI Committee 318 has chosen not to change the exponent applied to the compressive strength used to calculate development and lap splice lengths, but rather to set an upper limit of 0.69 MPa on $\sqrt{f_c'}$.

25.4.2 *Development of GFRP bars in tension*

R25.4.2 *Development of GFRP bars in tension*

25.4.2.1 Development length ℓ_d for bars in tension shall be the greater of (a), (b), and (c):

(a) Length calculated in accordance with 25.4.2.4 using the applicable modification factors of 25.4.2.4

(b) $20d_b$

(c) 300 mm

25.4.2.2 Intentionally left blank.

25.4.2.3 Intentionally left blank.

25.4.2.4 For GFRP bars, ℓ_d shall be calculated by:

$$\ell_d = \frac{d_b\left(\dfrac{f_{fr}}{0.083\sqrt{f_c'}} - 340\right)\psi_t}{13.6 + \dfrac{c_b}{d_b}} \qquad (25.4.2.4)$$

in which the term c_b/d_b shall not be taken greater than 3.5, and f_{fr} is the stress in the bar required to develop the full nominal sectional capacity.

R25.4.2.4 GFRP bars do not yield, they are linear elastic until fracture. Therefore, the concept of development length takes on a different meaning for GFRP bars than for steel bars. Instead of determining the length required to reach f_{fu}, which is rarely required, Eq. (25.4.2.4) is used to determine an embedment length to reach the required stress in the GFRP bar at the controlling limit state. For the GFRP rupture limit state f_{fr} will equal f_{fu}, but for the more commonly occurring limit state of concrete crushing the required stress in the bar at ultimate will be less than f_{fu}.

Equation (25.4.2.4), based on the work of Wambeke and Shield (2006), includes the effects of all variables controlling the development of stress in a straight GFRP bar in tension. In Eq. (25.4.2.4), c_b is a factor that represents the least of the side cover, the cover over the bar (in both cases measured to the center of the bar), or one-half the center-to-center spacing of the bars. ψ_t is the reinforcement location factor to reflect the effect of the casting position (that is, formerly denoted as "top bar effect"). A limit of 3.5 is placed on the term c_b/d_b. If c_b/d_b is less than 3.5, splitting failures are likely to occur. For values above 3.5, a pullout failure is expected, and an increase in cover is unlikely to increase the anchorage

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capacity. The presence of GFRP confining reinforcement has not been shown to lead to a measurable increase in the developable GFRP bar stress for a given embedment length.

25.4.2.5 For the calculation of ℓ_d, the reinforcement location factor ψ_t, shall be 1.5 if more than 12 in. of fresh concrete is placed below horizontal reinforcement being developed and 1.0 for all other cases.

R25.4.2.5 The reinforcement location or casting position factor ψ_t accounts for the position of the reinforcement in freshly placed concrete. The factor 1.5 is based on research (Wambeke and Shield 2006; Ehsani et al. 1996a).

25.4.3 *Development of GFRP standard hooks in tension*

R25.4.3 *Development of GFRP standard hooks in tension*

25.4.3.1 Development length ℓ_{dh} for bars in tension terminating in a standard hook shall be the greater of (a) through (c):

(a) $$\ell_{dh} = \begin{cases} 165\dfrac{d_b}{\sqrt{f_c'}} & \text{for } f_{fu} \leq 520 \text{ MPa} \\ \dfrac{f_{fu}}{3.1}\dfrac{d_b}{\sqrt{f_c'}} & \text{for } 520 \text{ MPa} < f_{fu} < 1030 \text{ MPa} \\ 330\dfrac{d_b}{\sqrt{f_c'}} & \text{for } f_{fu} \geq 1030 \text{ MPa} \end{cases}$$

(b) $\mathbf{12}\boldsymbol{d_b}$

(c) 230 mm

R25.4.3.1 Equations for the development length of GFRP bars terminating in standard hooks are based on the work of Ehsani et al. (1995, 1996b). The provisions for hooked bars are only applicable to standard hooks (refer to 25.3.1).

25.4.3.2 If the cover normal to the plane of the hook exceeds 64 mm and the cover extension beyond the hook is at least 50 mm, ℓ_{dh} calculated in accordance with 25.4.3.1 is permitted to be multiplied by 0.7.

R25.4.3.2 Unlike straight bar development, no distinction is made for casting position.

25.4.3.3 Intentionally left blank.

25.4.4 *Development of headed deformed bars in tension*—Out of scope

25.4.5 *Development of mechanically anchored GFRP bars in tension*

R25.4.5 *Development of mechanically anchored GFRP bars in tension*

25.4.5.1 Any mechanical attachment or device capable of developing $\mathbf{1.25}\boldsymbol{f_{fu}}$ of GFRP bars shall be permitted, provided it is approved by the building official in accordance with 1.10. Development of bars in tension shall be permitted to consist of a combination of mechanical anchorage plus additional embedment length of the bars between the critical section and the mechanical attachment or device.

R25.4.5.1 Anchorage of GFRP bars through the use of mechanical devices within concrete may be used if tests demonstrate the ability of the mechanical device to develop or anchor the desired force in the bar, as described in this provision.

25.4.5.2 The durability characteristics of the anchorage system shall not be less than the durability characteristics for GFRP bars prescribed in ASTM D7957.

R25.4.5.2 Annex F of CSA S807-19 includes a normative test to assess the durability characteristics of headed GFRP bars. Although this Code does not cover headed bars, the test method described in CSA S807-19 Annex F may be modified to determine the durability characteristics of anchorage systems in an alkaline environment.

25.4.6 *Development of welded deformed wire reinforcement in tension*—Out of scope

25.4.7 *Development of welded plain wire reinforcement in tension*—Out of scope

25.4.8 *Development of pretensioned seven-wire strands in tension*—Out of scope

25.4.9 *Development of GFRP bars in compression*

R25.4.9 *Development of GFRP bars in compression*

25.4.9.1 Development length ℓ_{dc} for bars in compression shall be the length calculated in accordance with 25.4.2.

R25.4.9.1 There is no experimental data on the development length of GFRP bars in compression. However, estimating the development length of GFRP bars in compression using expressions for the development length in tension is conservative. The weakening effect of flexural tension cracks is not present for bars in compression, and usually end bearing of the bars on the concrete is beneficial. Therefore, development lengths in compression should not be longer than those specified for tension.

25.4.9.2 Intentionally left blank.

25.4.9.3 Intentionally left blank.

25.4.10 *Reduction of development length for excess GFRP reinforcement*

R25.4.10 *Reduction of development length for excess GFRP reinforcement*

25.4.10.1 Reduction of development lengths defined in 25.4.2.1(a), 25.4.3.1(a), and 25.4.9.1 shall be permitted by use of the ratio $(A_{f,required})/(A_{f,provided})$, except where prohibited by 25.4.10.2. The modified development lengths shall not be less than the respective minimums specified in 25.4.2.1(b), 25.4.2.1(c), 25.4.3.1(b), and 25.4.3.1(c).

R25.4.10.1 A reduction in development length is permitted in limited circumstances if excess reinforcement is provided. The reinforcement stress f_{fr} that is to be developed in 25.4.2.4 is the stress that allows the section to reach its full flexural capacity, M_n. In many cases, the amount of flexural reinforcement will be controlled by serviceability requirements, providing ϕM_n well in excess of M_u. In these cases, a reduction in the development length is permitted because the development of bar stress along the embedment length increases at a rate greater than linear (Wambeke and Shield 2006).

25.4.10.2 A reduction of development length in accordance with 25.4.10.1 is not permitted for (a) through (c).

(a) At locations where anchorage or development for f_{fu} is required

(b) Where bars are required to be continuous

(c) For mechanically anchored reinforcement

R25.4.10.2 The reduction factor based on area is not to be used in those cases where anchorage development for full strength f_{fu} is required. For example, the excess reinforcement factor does not apply for development of shrinkage and temperature reinforcement according to 24.4.3.4 or for development of reinforcement provided according to 8.7.4.2, 9.7.7, and 9.8.1.6.

25.5—Splices

25.5.1 *General*

R25.5—Splices

R25.5.1 *General*

Lap splice lengths of longitudinal reinforcement in columns should be calculated in accordance with 10.7.5 and this section.

25.5.1.1 Lap splices shall not be permitted for bars larger than No. M32.

R25.5.1.1 Because of lack of adequate experimental data on lap splices of GFRP bars larger than No. M32 in compression and in tension, lap splicing of these bar sizes is prohibited.

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=**25.5.1.2** For contact lap splices, minimum clear spacing between the contact lap splice and adjacent splices or bars shall be in accordance with the requirements for individual bars in 25.2.1.

=**25.5.1.3** For noncontact splices in flexural members, the transverse center-to-center spacing of spliced bars shall not exceed the lesser of one-fifth the required lap splice length and 6 in.

25.5.1.4 Intentionally left blank.

25.5.1.5 Intentionally left blank.

25.5.2 *Lap splice lengths of GFRP bars in tension*

25.5.2.1 Tension lap splice length ℓ_{st} for bars in tension shall be in accordance with Table 25.5.2.1, where ℓ_d shall be in accordance with 25.4.2.1(a).

25.5.2.2 If bars of different size are lap spliced in tension, ℓ_{st} shall be ℓ_{st} of the larger bar.

25.5.3 *Lap splice lengths of welded deformed wire reinforcement in tension*—Out of scope

25.5.4 *Lap splice lengths of welded plain wire reinforcement in tension*—Out of scope

25.5.5 *Lap splice lengths of GFRP bars in compression*

25.5.5.1 Compression lap splice length shall be calculated in accordance with 25.5.2.

25.5.5.2 Intentionally left blank.

25.5.5.3 Intentionally left blank.

25.5.5.4 Intentionally left blank.

25.5.6 *End-bearing splices of deformed bars in compression*—Out of scope

25.5.7 *Mechanical splices of GFRP bars in tension or compression*

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R25.5.1.3 If individual bars in noncontact lap splices are too widely spaced, an unreinforced section is created. Forcing a potential crack to follow a zigzag line (5-to-1 slope) is considered a minimum precaution.

R25.5.5 *Lap splice lengths of GFRP bars in compression*

GFRP bond research has been primarily related to bars in tension. Bond behavior of GFRP compression bars is not complicated by the problem of transverse tension cracking and thus, the use of the same provisions for compression splices as for tension splices is conservative.

Lap splice requirements particular to columns are provided in Chapter 10.

R25.5.7 *Mechanical splices of GFRP bars in tension or compression*

Table 25.5.2.1—Lap splice lengths of GFRP bars in tension

$A_{f,provided}/A_{f,required}$* over length of splice	Maximum percent of A_f spliced within required lap length	Splice type	ℓ_{st}	
≥ 2.0	50	Class A	Greater of:	$1.0\ell_d$, $20d_b$, and 300 mm
	100	Class B	Greater of:	$1.3\ell_d$, $20d_b$, and 300 mm
< 2.0	All cases	Class B		

*Ratio of area of reinforcement provided to area of reinforcement required by analysis at splice location.

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25.5.7.1 A mechanical splice shall develop in tension or compression, as required, at least $\mathbf{1.25}f_{fu}$ of the GFRP bar.

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R25.5.7.1 The maximum reinforcement stress used in design under the Code is the specified design tensile strength defined in 20.2.2.3. Because rupture is a brittle failure, the 25% increase above the specified design tensile strength was selected as both an adequate minimum for safety and a practicable maximum for economy.

25.5.7.2 Intentionally left blank.

25.5.7.3 Mechanical splices need not be staggered.

R25.5.7.3 Although mechanical splices need not be staggered, staggering is encouraged and may be necessary for constructability to provide enough space around the splice for installation or to meet the clear spacing requirements.

25.5.7.4 Mechanical splices shall not contain any parts that are susceptible to corrosion.

25.6—Bundled reinforcement—Out of scope

R25.6—Bundled reinforcement—Out of scope

Bundled bars are not covered by this Code due to a lack of sufficient published research on this topic; however, the use of bundled GFRP bars has been studied by Asadian et al. (2019).

25.7—GFRP transverse reinforcement

25.7.1 *GFRP stirrups*

25.7.1.1 Stirrups shall extend as close to the compression and tension surfaces of the member as cover requirements and proximity of other reinforcement permits and shall be anchored at each end. Where used as shear reinforcement, stirrups shall extend a distance $\boldsymbol{d}$ from extreme compression fiber.

R25.7—GFRP transverse reinforcement

R25.7.1 *GFRP stirrups*

R25.7.1.1 Stirrup legs should be extended as close as practicable to the compression face of the member because, near ultimate load, the flexural tension cracks penetrate deeply toward the compression zone.

It is essential that shear and torsional reinforcement be adequately anchored at each end to be fully effective on either side of any potential inclined crack. This generally requires a hook or bend at the end of the reinforcement as provided by this section.

25.7.1.2 Between anchored ends, each bend in the continuous portion of a single or multiple leg stirrup shall enclose a longitudinal bar.

25.7.1.3 Anchorage of each stirrup leg shall be in accordance with (a) or (b):

(a) standard hook around longitudinal reinforcement at both ends

(b) lap of at least $\mathbf{1.3}\boldsymbol{\ell_d}$ per 25.4.2.1 with f_{fr} equal to f_{ft} on one end and standard hook around longitudinal reinforcement at the other end.

R25.7.1.3 GFRP stirrups typically have different shapes than those usually used in steel-reinforced concrete. Common geometry for GFRP stirrups is shown in Fig. R25.7.1.3a through R25.7.1.3c representing two U-shaped bars inserted from the side, a single bar with four 90-degree bends, and a continuous closed stirrup that has no ends. Where GFRP stirrups used for shear reinforcement take the form of two U-shaped bars inserted from the sides as shown in Fig. R25.7.1.3a, anchorage is provided by the standard hooks at the ends of the bars and there is no requirement to overlap the tails of the U-shaped bars. ACI 318 provisions for bond of hooked steel bars cannot be applied directly to GFRP bars because of the different mechanical properties and bond behavior. The tensile force in a vertical GFRP stirrup leg is primarily transferred to the concrete through the tail beyond the hook. Ehsani et al. (1995) found that for a tail length beyond $\mathbf{12}\boldsymbol{d_b}$, as required by 25.3.1, there is no significant slip-

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page. Alternatively, GFRP stirrups used for shear reinforcement can be formed with an overlap length on one face as shown in Fig. R25.7.1.3b. For the lapped stirrups, anchorage for the side with the overlap is provided by the bend at one end and the overlap at the other end.

For the continuous-closed GFRP stirrups shown in Fig. R25.7.1.3c, anchorage is provided for all four legs by the 90-degree bends around longitudinal reinforcement at each corner. There is no equivalent to the continuous-closed GFRP stirrup for steel.

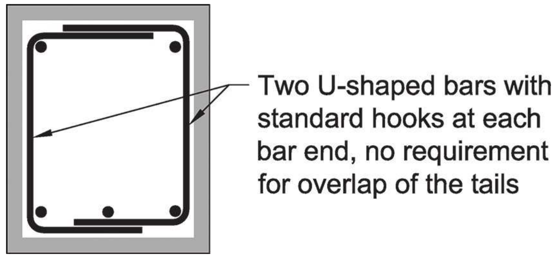

Fig. R25.7.1.3a—Anchorage provided by standard hooks.

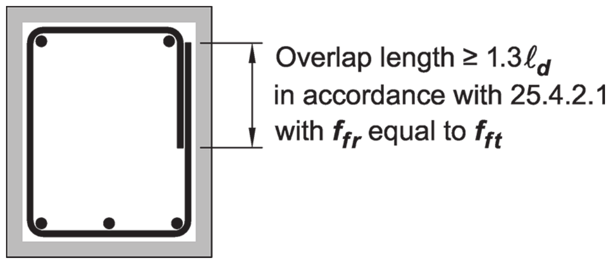

Fig. R25.7.1.3b—Anchorage provided by overlap length.

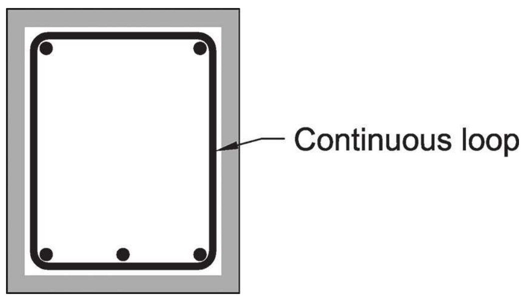

Fig. R25.7.1.3c—Anchorage provided by continuous-closed GFRP stirrups.

25.7.1.4 Intentionally left blank.

25.7.1.5 Intentionally left blank.

25.7.1.6 Stirrups used for torsion or integrity reinforcement shall be closed stirrups perpendicular to the axis of the member. Each end of every stirrup leg shall be anchored with 90-degree standard hooks around a longitudinal bar.

R25.7.1.6 Both longitudinal and closed transverse reinforcement are required to resist the diagonal tension stresses due to torsion. The stirrups should be closed because inclined cracking due to torsion may occur on all faces of a member.

In the case of sections subjected primarily to torsion, the concrete side cover to the stirrups spalls off at high torsional

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25.7.1.6.1 GFRP stirrups used for torsion or integrity reinforcement shall be permitted to be made up of two pieces of reinforcement. A C-shaped stirrup anchored according to 25.7.1.3a shall be closed by either a U-shaped or a C-shaped stirrup anchored according to 25.7.1.3a.

25.7.2 *GFRP ties*

25.7.2.1 Ties shall consist of a closed loop of bar with spacing in accordance with (a) and (b):

(a) Clear spacing of at least $\mathbf{(4/3)}\boldsymbol{d_{agg}}$

(b) Center-to-center spacing shall not exceed the least of $\mathbf{12}\boldsymbol{d_b}$ of longitudinal bar, $\mathbf{24}\boldsymbol{d_b}$ of tie bar, and smallest dimension of member

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moments (Mitchell and Collins 1976). This renders lapped-spliced stirrups ineffective, leading to a premature torsional failure (Behera and Rajagopalan 1969). In such cases, closed stirrups should not be made up of pairs of U-stirrups lapping one another. Individual stirrups that have no ends (termed continuous-closed stirrups) as shown in Fig. R25.7.1.3c and spirals satisfy the requirements of 25.7.1.6. There is no equivalent to the continuous-closed stirrup for steel.

R25.7.1.6.1 Figures R25.7.1.6.1a and R25.7.1.6.1b illustrates GFRP stirrup details that satisfy the requirements of 25.7.1.6.1. Tests conducted on full-scale RC beams reinforced longitudinally with GFRP bars and transversely by C-shaped GFRP stirrups as illustrated in Fig. R25.7.1.6b indicated that the beams reached the ultimate torsional strength without stirrup anchorage failure (Mohamed and Benmokrane 2015).

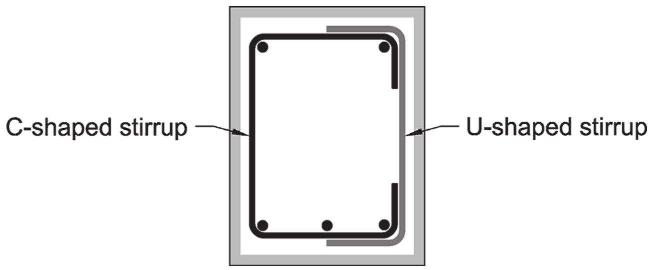

Fig. R25.7.1.6.1a—C-shaped GFRP stirrup closed by a U-shaped stirrup.

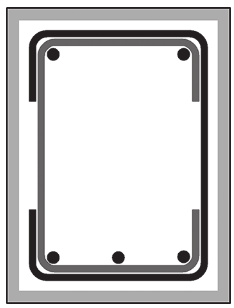

Fig. R25.7.1.6.1b—C-shaped GFRP stirrup closed by a C-shaped GFRP stirrup.

R25.7.2 *GFRP ties*

R25.7.2.1 The modulus of elasticity of GFRP bars is lower than that of steel bars, requiring closer support of the longitudinal GFRP bars to prevent buckling. The spacing between ties can be related to the diameter of the longitudinal bars by a simplified model that assumes that the bar is a compressive member simply supported between the supports provided by the ties (Jawaheri Zadeh and Nanni 2013). Neglecting the lateral support from the concrete cover leads to $\boldsymbol{s_{max}} \approx \mathbf{17.5}\boldsymbol{d_b}$, for the yielding of steel bars prior to buckling, which is in good agreement with ACI 318 ($\boldsymbol{s_{max}} = \mathbf{16}\boldsymbol{d_b}$). The same analysis leads to $\boldsymbol{s_{max}} \approx \mathbf{14}\boldsymbol{d_b}$ to avoid GFRP buckling prior

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to concrete crushing. Thus, a conservative value of 12 longitudinal-bar diameters, as also confirmed by experimental evidence (Guerin et al. 2018b; De Luca et al. 2010), has been adopted. Tie spacing as related to the diameter of tie bars is also reduced to achieve a desired level of concrete confinement due to the reduction in GFRP modulus of elasticity compared to steel. Additional provisions for minimum spacing are specified in 10.7.6.

25.7.2.2 Diameter of tie bar shall be at least No. M10.

R25.7.2.2 These provisions apply to crossties as well as ties. GFRP bars larger than No. M32 are not covered by this Code.

25.7.2.3 Rectilinear ties shall be arranged to satisfy (a), (b), and (c):

(a) 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.

(b) Each bar shall have less than 150 mm clear on each side along the tie from a laterally supported bar.

(c) If ties are constructed from pairs of overlapping U-shapes, overlaps at ends of adjacent rectilinear ties shall be staggered around the perimeter.

R25.7.2.3 The maximum permissible included angle of 135 degrees and the exemption of bars located within 6 in. clear on each side along the tie from adequately tied bars is illustrated in Fig. R25.7.2.3a. Adequate staggering of ties is illustrated in Fig. R25.7.2.3b.

25.7.2.3.1 Anchorage of rectilinear ties shall be provided by standard hooks that conform to 25.3.2 and engage a longitudinal bar. The minimum overlap of bar ends shall be the greater of $\mathbf{20}\boldsymbol{d_b}$ or 150 mm.

R25.7.2.3.1 Longer overlaps may be specified, provided they do not interfere with tie bends. An overlap of $\mathbf{20}\boldsymbol{d_b}$ provides at least as much overlap length as that required for Class B splices in 25.5.2.1 for stresses up to $\mathbf{0.005}\boldsymbol{E_f}$ in No. M13 and smaller bars.

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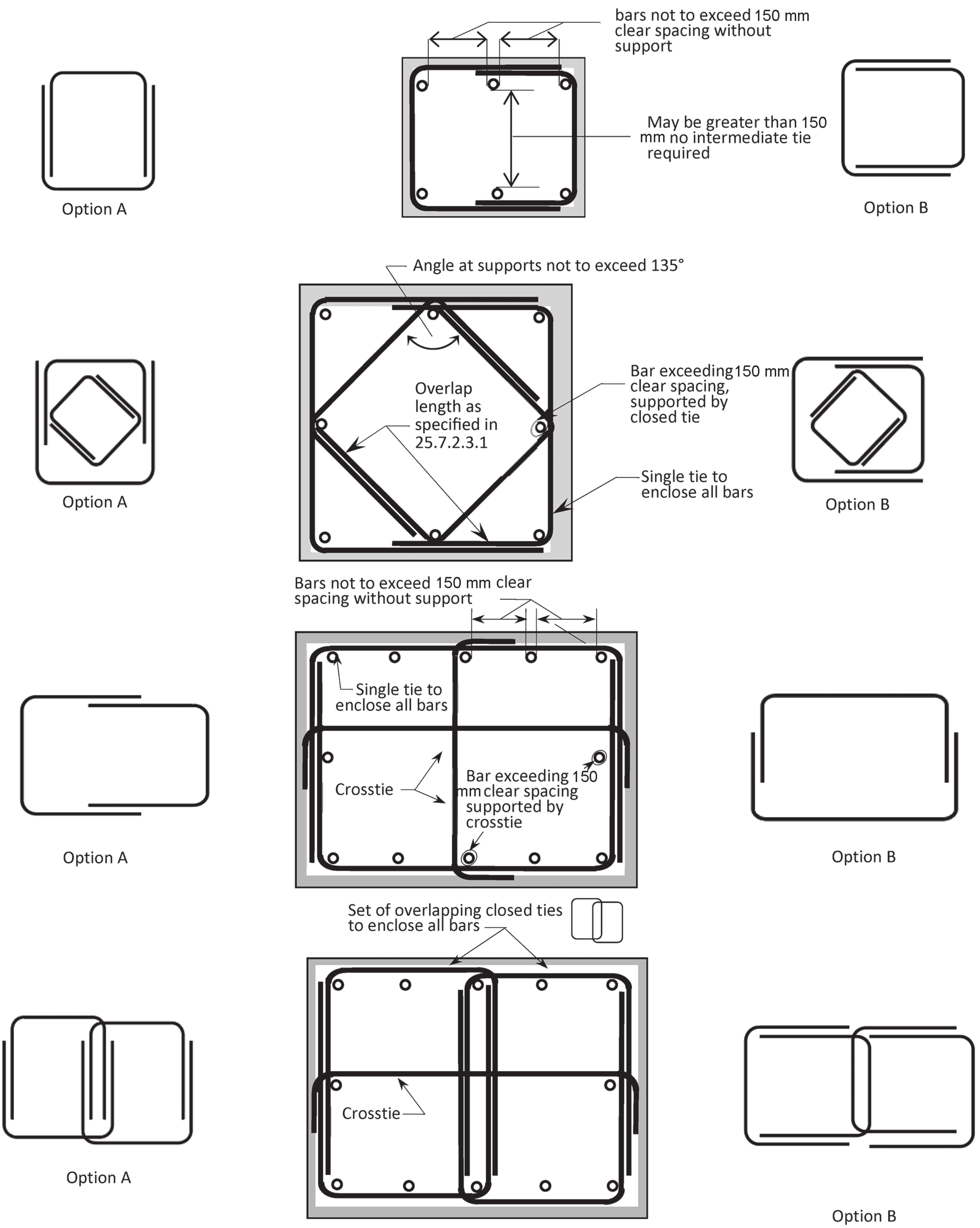

Fig. R25.7.2.3a—Illustrations to clarify measurements between laterally supported column bars and rectilinear tie anchorage.

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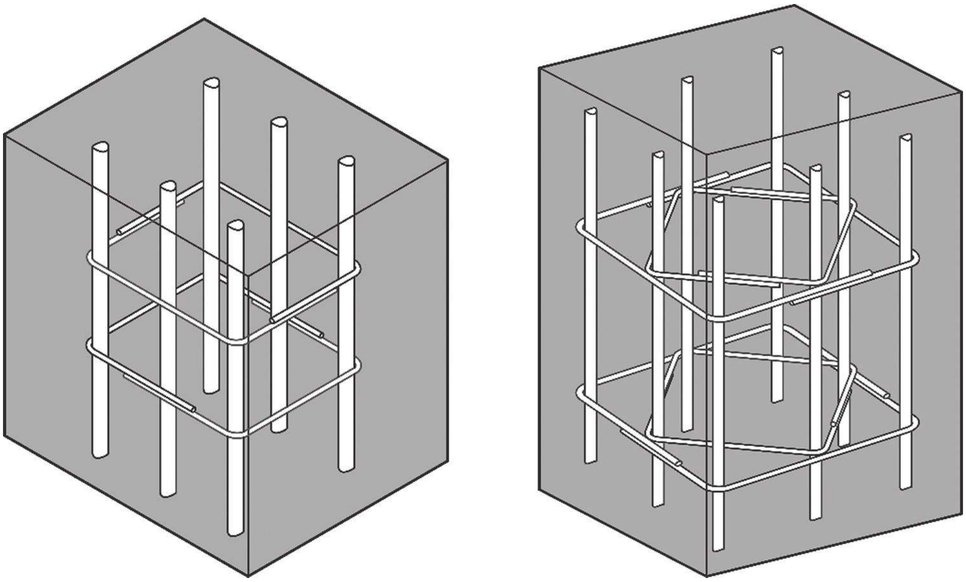

Fig. R25.7.2.3b—Illustrations to clarify staggering of ties.

25.7.2.4 Circular ties shall be permitted where longitudinal bars are located around the perimeter of a circle. Circular ties shall be continuous-closed ties or shall be anchored in accordance with 25.7.2.4.1.

25.7.2.4.1 Anchorage of individual circular ties shall be in accordance with (a) or (b):

(a) Ends shall overlap by at least 150 mm and terminate with standard hooks in accordance with 25.3.2 that engage a longitudinal bar

(b) Ends shall overlap by at least $\mathbf{20}\boldsymbol{d_b}$ with a crosstie placed to intersect the overlap.

R25.7.2.4.1 Vertical splitting and loss of tie restraint are possible where the overlapped ends of adjacent circular ties are anchored at a single longitudinal bar. Adjacent circular ties should not engage the same longitudinal bar with end hook anchorages (refer to Fig. R25.7.2.4.1). Test results indicate that using overlap length equal to 20 times the diameter of the closed GFRP ties without standard hooks was sufficient to avoid pullout or slippage failure within the closed GFRP ties in columns tested under axial compression load (Mohamed et al. 2014a). The provisions of 25.5.2.1 set 20 diameters as the minimum requirement for overlap length. An overlap of $\mathbf{20}\boldsymbol{d_b}$ provides at least as much overlap length as that required for class B splices in 25.5.2.1 for stresses up to $\mathbf{0.005}\boldsymbol{E_f}$ in No. M13 and smaller bars. Illustration of the crosstie placed to intersect the overlap in 25.7.2.4.1(b) is shown in Fig. R25.7.2.4.1. The purpose of the crosstie is to hold the circular tie in place and prevent buckling of the longitudinal bars in the event of concrete cover spalling. Alternatively, continuous-closed circular ties can be used, which do not require any special detailing for anchorage.

25.7.2.4.2 Overlaps at ends of adjacent circular ties shall be staggered around the perimeter enclosing the longitudinal bars.

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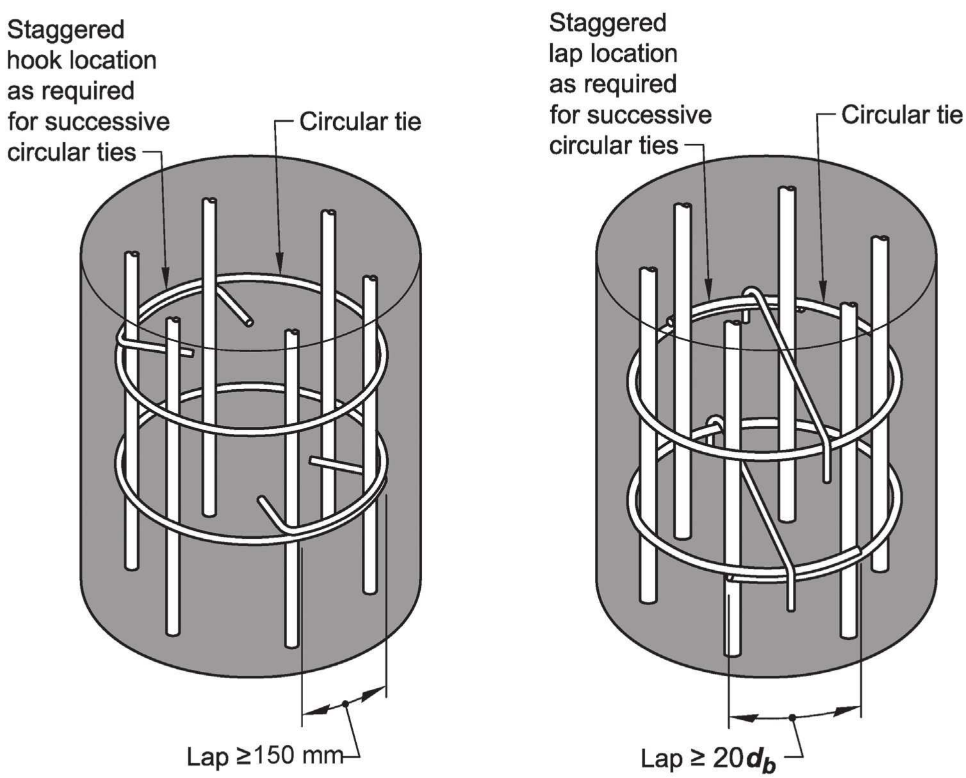

Fig. R25.7.2.4.1—Circular tie anchorage.

25.7.2.5 Ties to resist torsion shall be perpendicular to the axis of the member and shall conform to the requirements of 25.7.1.6.

R25.7.2.5 Closed ties may also be used to resist torsion in columns.

25.7.3 *Spirals*

R25.7.3 *Spirals*

25.7.3.1 Spirals shall consist of evenly spaced continuous bar with clear spacing conforming to (a) and (b):

(a) At least the greater of 25 mm and **(4/3)*d_{agg}***

(b) Not greater than 75 mm

R25.7.3.1 Spirals should be held firmly in place, at proper pitch and alignment, to prevent displacement during concrete placement.

25.7.3.2 Spiral bar diameter shall be at least 10 mm.

R25.7.3.2 For practical considerations, the minimum diameter of spiral reinforcement is 10 mm (No. M10 bar). GFRP bar without surface enhancements (for example, sand coating or wrapping) is suitable for GFRP spirals.

25.7.3.3 Volumetric spiral reinforcement ratio ρ_s shall satisfy Eq. (25.7.3.3).

$$\rho_s \geq 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{0.004E_f} \qquad (25.7.3.3)$$

R25.7.3.3 The effect of spiral reinforcement in increasing the strength of the concrete within the core is not fully realized until the column has been subjected to a load and deformation sufficient to cause the concrete shell outside the core to spall off. The amount of spiral reinforcement required by Eq. (25.7.3.3) is intended to provide additional load-carrying capacity for concentrically loaded columns equal to or slightly greater than the strength lost when the shell spalls off. The derivation of Eq. (25.7.3.3) for steel reinforcement is given by Richart (1933). In this derivation, the stress in the steel spiral is assumed to correspond to a strain of 0.005. The stress in the GFRP spiral is conservatively assumed to correspond to a strain of 0.004 (Afifi et al. 2015).

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=**25.7.3.4** Spirals shall be anchored by 1-1/2 extra turns of spiral bar at each end.

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=**R25.7.3.4** Spiral anchorage is illustrated in Fig. R25.7.3.4.

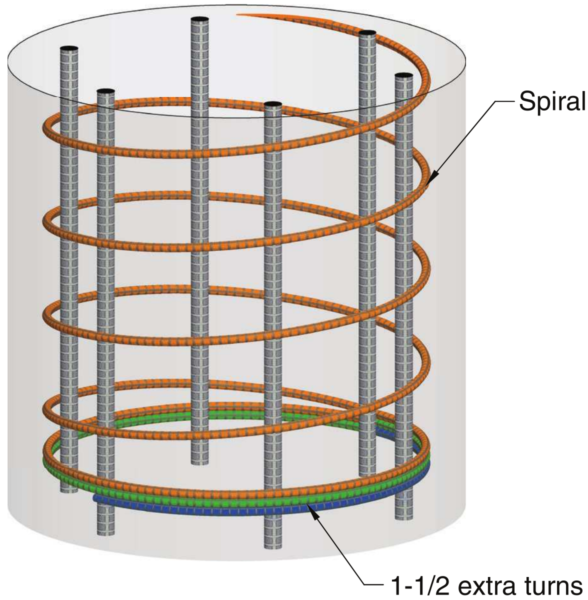

Fig. R25.7.3.4—Spiral anchorage.

25.7.3.5 Intentionally left blank.

25.7.3.6 Spiral lap splices shall be at least the greater of one full turn and $72d_b$.

R25.7.3.6 The minimum 300 mm splice length typically imposed in the case of steel spirals never governs against the other requirements in the case of GFRP spirals.

25.7.4 *Hoops*—Out of scope

25.8—Post-tensioning anchorages and couplers—Out of scope

R25.8—Post-tensioning anchorages and couplers—Out of scope

Specification of post-tensioned anchorages and couplers are not covered in this Code because this Code does not cover GFRP prestressed concrete members.

25.9—Anchorage zones for post-tensioned tendons—Out of scope

R25.9—Anchorage zones for post-tensioned tendons—Out of scope

Anchorage zones for post-tensioned tendons are not covered in this Code because this Code does not cover GFRP prestressed concrete members.

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CHAPTER 26—CONSTRUCTION DOCUMENTS AND INSPECTION

26.1—Scope

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CHAPTER R26—CONSTRUCTION DOCUMENTS AND INSPECTION

R26.1—Scope

This chapter establishes the minimum requirements for information that must be included in the construction documents as applicable to the project. The requirements include information developed in the structural design that must be conveyed to the contractor, provisions directing the contractor on required quality, and inspection requirements to verify compliance with the construction documents. All provisions relating to construction have been gathered into this chapter for use by the licensed design professional.

This chapter is directed to the licensed design professional responsible for incorporating project requirements into the construction documents. The construction documents should contain all of the necessary design and construction requirements for the contractor to achieve compliance with the Code. It is not intended that the Contractor will need to read and interpret the Code.

A general reference in the construction documents requiring compliance with this Code is to be avoided because the contractor is rarely in a position to accept responsibility for design details or construction requirements that depend on detailed knowledge of the design. References to specific Code provisions should be avoided as well because it is the intention of the Code that all necessary provisions be included in the construction documents. For example, references to specific provisions within Chapter 26 are expected to be replaced with the appropriate references within the project construction documents. Reference to ACI and ASTM standards as well as to other documents is expected.

This chapter includes provisions for some of the information that is to be in the construction documents. This chapter is not intended as an all-inclusive list; additional items may be applicable to the Work or required by the building official. ACI 440.5 is a reference construction specification that is written to be consistent with the requirements of this Code.

It is recognized that there are situations, such as those in precast structures, where design and detailing of portions of the Work are delegated to specialty engineers or contractors who may retain the services of a specialty engineer. Such specialty engineers should be licensed design professionals who are sufficiently knowledgeable in the design and construction of the structural items being delegated for design.

Chapter 26 is organized as shown below:

Section	Coverage
26.1	Scope
26.2	Design criteria
26.3	Member information
26.4	Concrete materials and mixture requirements
26.5	Concrete production and construction
26.6	GFRP reinforcement materials and construction requirements
26.8	Embedments
26.9	Additional requirements for precast concrete
26.11	Formwork

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26.12	Concrete evaluation and acceptance
26.13	Inspection

26.1.1 This chapter addresses (a) through (c):

(a) Design information that the licensed design professional shall specify in the construction documents, if applicable.

(b) Compliance requirements that the licensed design professional shall specify in the construction documents, if applicable.

R26.1.1(a) and (b) Except for the inspection requirements of 26.13, the provisions of this chapter are organized by design information and compliance requirements.

Design information is project specific and developed during the structural design. It describes the basis of the design or provides information regarding the construction of the Work. Only design information that is applicable to the Work need be provided.

Compliance requirements are general provisions that provide a minimum acceptable level of quality for construction of the Work. It is not the intent of the Code to require the licensed design professional to incorporate verbatim the compliance requirements into the construction documents. Some of these requirements may not be applicable to a specific project.

Construction documents that incorporate the minimum applicable compliance requirements of this chapter are considered to comply with the Code, even if the requirements are stated differently, exceed these minimum requirements, or provide more detail.

(c) Inspection requirements that the licensed design professional shall specify in the construction documents, if applicable.

R26.1.1(c) Section 26.13 provides inspection provisions to be used in the absence of general building code inspection provisions. These inspection requirements are intended to provide verification that the Work complies with the construction documents.

The inspection requirements of the governing jurisdiction or the general building code take precedence over those included in this chapter; refer to 26.13.1. ACI 311.4R provides guidance for inspection of concrete construction, and ACI 311.6M is a reference specification for testing services for ready mixed concrete.

26.2—Design criteria

26.2.1 Design information:

(a) Name and year of issue of the Code, general building code, and any supplements governing design.

(b) Loads used in design.

(c) Design work delegated to the contractor including applicable design criteria.

R26.2—Design criteria

R26.2.1(a) and (b) Reference to the applicable version of the documents that govern the design including essential loading information, such as gravity and lateral loading, is to be included in the construction documents.

R26.2.1(c) Examples of design criteria include dimensions, loads, and other assumptions used during design that may affect the delegated portion of the Work.

26.2.2 Compliance requirements:

(a) Design work delegated to the contractor shall be performed by a specialty engineer.

(b) The contractor's specialty engineer, relying on the documents identifying the portion of design work assigned, shall produce design work that is compatible with the construction documents and the design criteria provided by the licensed design professional in charge of the design work.

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(c) The contractor shall submit necessary information to the licensed design professional to confirm that the specialty engineer complied with the documents identifying the portion of the design work assigned.

26.3—Member information

26.3.1 Design information:

(a) Member size, location, and related tolerances

(b) Intentionally left blank

(c) Identify structural members for which modulus of elasticity testing of concrete mixtures is required

26.3.2 Intentionally left blank.

26.4—Concrete materials and mixture requirements

26.4.1 *Concrete materials*

26.4.1.1 *Cementitious materials*

⁼**26.4.1.1.1** Compliance requirements:

(a) Cementitious materials shall conform to the specifications in Table 26.4.1.1.1(a), except as permitted in 26.4.1.1.1(b)

Table 26.4.1.1.1(a)—Specifications for cementitious materials

Cementitious material	Specification
Portland cement	ASTM C150
Blended hydraulic cements	ASTM C595, excluding Type IS (≥70) and Type IT ($S \geq 70$)
Expansive hydraulic cement	ASTM C845
Hydraulic cement	ASTM C1157
Fly ash and natural pozzolan	ASTM C618
Slag cement	ASTM C989
Silica fume	ASTM C1240

(b) Alternative cements shall be permitted if approved by the licensed design professional and the building official. Approval shall be based upon test data documenting that the proposed concrete mixture made with the alternative cement meets the performance requirements for the application including structural, fire, and durability.

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R26.3—Member information

⁼**R26.3.1(a)** Construction tolerances for member size and location can be incorporated in construction documents by reference to ACI 117M for cast-in-place construction or to ACI ITG-7 for precast construction. Specific project tolerances that are more restrictive or that are not covered in these references should also be included in the construction documents.

R26.4—Concrete materials and mixture requirements

⁼**R26.4.1.1.1(b)** Provisions for strength and durability in Chapter 19 and many requirements in Chapter 26 are based on test data and experience using concretes made with cementitious materials meeting the specifications in Table 26.4.1.1.1(a).

Some alternative cements may not be suitable for use in structural concrete covered by this Code. Therefore, requirements are included for evaluating the suitability of alternative cements. Recommendations for concrete properties to be evaluated are discussed in Becker et al. (2019), ITG-10R, and ITG-10.1R.

In addition to test data, documentation of prior successful use of the proposed alternative cement in structural concrete

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for conditions with essentially equivalent performance requirements as those of the project can be helpful to the licensed design professional determining whether to allow use of the material. As with all new technologies, a project owner should be informed of the risks and rewards.

26.4.1.2 *Aggregates*

R26.4.1.2 *Aggregates*

26.4.1.2.1 Compliance requirements:

(a) Aggregates shall conform to ASTM C33.

(b) Aggregates not conforming to ASTM C33 are permitted if they have been shown by test or actual service to produce concrete of adequate strength and durability and are approved by the building official.

R26.4.1.2.1(b) Aggregates conforming to ASTM specifications are not always economically available and, in some instances, materials that do not conform to ASTM C33 may have a documented history of satisfactory performance under similar exposure. Such nonconforming materials are permitted if acceptable evidence of satisfactory performance is provided. Generally, aggregates conforming to the designated specifications should be used.

(c) Crushed hydraulic-cement concrete or recycled aggregate shall be permitted if approved by the licensed design professional and the building official based on documentation that demonstrates compliance with (1) and (2).

(1) Concrete incorporating the specific aggregate proposed for the Work has been demonstrated to provide the mechanical properties and durability required in structural design.

(2) A testing program to verify aggregate consistency and a quality control program to achieve consistency of properties of the concrete are conducted throughout the duration of the project.

R26.4.1.2.1(c) This Code requires that concrete made with crushed hydraulic-cement concrete or recycled aggregate be specifically approved for use in a particular project. Properties of fresh and hardened concrete made with these aggregates are influenced by the nature, quality, and variability of the source concrete that is crushed to produce aggregate; nature and variability of the waste-stream from which recycled aggregate is extracted; and the grading, proportions, and uniformity of the resulting aggregate.

ASTM C33 notes that use of such aggregates "may require some additional precautions." These precautions include that any such aggregates meet the durability requirements of ASTM C33 and that the proposed concrete mixture meets the durability requirements of the Exposure Classes assigned for the Work. Areas of special concern include evidence of alkali-silica reactivity and sulfate content of concrete. Additionally, properties of concrete made with crushed hydraulic-cement concrete or recycled aggregate can be significantly more variable than those of comparable concretes made with conventional normalweight aggregates (Bezerra Cabral et al. 2010).

This Code requires explicit documentation to verify that concrete made with crushed hydraulic-cement concrete or recycled aggregate can consistently provide the mechanical properties and durability required in design. Such properties may have been calculated or assumed in the design process, but may not have been specified in contract documents. Specific criteria for approval of concrete made with recycled aggregates including crushed hydraulic-cement concrete are expected to be unique to each project and set of exposure conditions. The project-specific test program and acceptance criteria should be established by the licensed design professional.

ACI 555R provides information on issues that should be considered in verifying required performance.

26.4.1.3 *Mineral fillers*

R26.4.1.3 *Mineral fillers*

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26.4.1.3.1 Compliance requirements:
(a) Mineral fillers shall conform to ASTM C1797.

26.4.1.4 *Water*

26.4.1.4.1 Compliance requirements:
(a) Mixing water shall conform to ASTM C1602.

26.4.1.5 *Admixtures*

26.4.1.5.1 Compliance requirements:
(a) Admixtures shall conform to (1) through (3):
  (1) Water reduction and setting time modification: ASTM C494.
  (2) Producing flowing concrete: ASTM C1017.
  (3) Air entrainment: ASTM C260.

(b) Admixtures that do not conform to the specifications in 26.4.1.5.1(a) shall be subject to prior review by the licensed design professional.
(c) Admixtures used in concrete containing expansive cements conforming to ASTM C845 shall be compatible with the cement and produce no deleterious effects.

26.4.1.6 *Steel fiber reinforcement*—Not applicable

26.4.1.7 *Packaged, preblended, dry, combined materials for shotcrete*—Out of scope

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R26.4.1.3.1(a) Mineral fillers are finely ground products derived from aggregate that can be used in self-consolidating concrete or in any concrete mixture to improve the properties of fresh and hardened concrete by optimizing particle packing. ASTM C1797 defines Types A and B mineral fillers derived from carbonate aggregate and Type C mineral fillers derived from quarried stone of any mineralogy. Refer to 26.4.2 for restrictions to use of carbonate-based mineral filler in concrete exposed to sulfates.

R26.4.1.4 *Water*

R26.4.1.4.1 Almost any natural water that is potable and has no pronounced taste or odor is satisfactory as mixing water for making concrete. Excessive impurities in mixing water may affect setting time, concrete strength, and volume stability, and may also cause efflorescence.

ASTM C1602 allows the use of potable water without testing and includes methods for qualifying nonpotable sources of water, such as from concrete production operations, with consideration of effects on setting time and strength. Testing frequencies are established to ensure continued monitoring of water quality.

ASTM C1602 includes optional limits for chlorides, sulfates, alkalis, and solids in mixing water that can be invoked if appropriate.

R26.4.1.5 *Admixtures*

R26.4.1.5.1(a) ASTM C494 includes Type S—specific performance admixtures—that can be specified if performance characteristics not listed in 26.4.1.5.1(a) are desired, such as viscosity-modifying admixtures. The basic requirement for a Type S admixture is that it will not have adverse effects on the properties of concrete when tested in accordance with ASTM C494. Meeting the requirements of Type S does not ensure that the admixture will perform its described function. The manufacturer of an admixture presented as conforming to Type S should also be required to provide data that the product will meet the performance claimed.

R26.4.1.5.1(c) In some cases, the use of admixtures in concrete containing ASTM C845 expansive cements has resulted in reduced levels of expansion or increased shrinkage values. Refer to ACI 223R.

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26.4.2 *Concrete mixture requirements*

26.4.2.1 Design information:

(a) Requirements (1) through (12) for each concrete mixture, based on assigned exposure classes or design of members:

(1) Minimum specified compressive strength of concrete, f_c'.

(2) Minimum modulus of elasticity of concrete, E_c, if specified in accordance with 19.2.2.2.

(3) Test age, if different from 28 days, for demonstrating compliance with f_c' and E_c if specified.

(4) Maximum *w*/*cm* applicable to most restrictive assigned durability exposure class from 19.3.2.1.

(5) Nominal maximum size of coarse aggregate not to exceed the least of (i), (ii), and (iii):

(i) one-fifth the narrowest dimension between sides of forms

(ii) one-third the depth of slabs

(iii) three-fourths the minimum specified clear spacing between individual reinforcing bars

These limitations shall not apply if, in the judgment of the licensed design professional, workability and methods of consolidation are such that concrete can be placed without honeycombs or voids.

(6) Applicable air content for Exposure Category F from 19.3.3.1.

(7) For members assigned to Exposure Class F3, indicate that concrete mixtures shall meet the limits on supplementary cementitious materials in Table 26.4.2.2(b).

(8) For members assigned to Exposure Class S1, S2, or S3, indicate that mineral fillers derived from carbonate aggregate are prohibited unless approved by the licensed design professional.

(9) Applicable cementitious materials for Exposure Category S from 19.3.2.1.

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R26.4.2 *Concrete mixture requirements*

R26.4.2.1(a) The requirements for each concrete mixture used for the Work are to be stated in the construction documents. These are determined from applicable concrete design requirements in 19.2 and durability requirements in 19.3. The most restrictive requirements that apply are to be stated.

R26.4.2.1(a)(4) In accordance with Table 19.3.2.1, the *w*/*cm* is based on all cementitious and supplementary cementitious materials in the concrete mixture. The *w*/*cm* of concrete made with alternative cements may not reflect the strength and durability characteristics of the concrete made with portland cement and supplementary cementitious materials permitted in Table 26.4.1.1.1(a). As noted in R26.4.1.1.1(b), it is imperative that testing be conducted to determine the performance of concrete made with alternative cements and to develop appropriate project specification.

R26.4.2.1(a)(5) The size limitations on aggregates are provided to facilitate placement of concrete around the reinforcement without honeycombing due to blockage by closely-spaced reinforcement. It is the intent of the Code that the licensed design professional select the appropriate nominal maximum size aggregate and include this value in the construction documents for each concrete mixture. Because maximum aggregate size can impact concrete properties such as shrinkage, and also the cost of concrete, the largest aggregate size consistent with the requirements of 26.4.2.1 should be permitted. Increasing aggregate size will only decrease shrinkage if there is a concurrent reduction in paste volume.

R26.4.2.1(a)(6) ASTM C94 and ASTM C685 include a tolerance for air content as delivered of ±1.5 percentage points.

R26.4.2.1(a)(8) If concrete members are assigned to Exposure Class S1, S2, or S3, the use of mineral fillers derived from carbonate aggregate in concrete mixtures can result in a form of sulfate attack. Information is provided in ACI 201.2R. ASTM C1797 Type C mineral fillers that are derived from noncarbonate quarried stone can be used in concrete exposed to sulfates. If the quantity of Type A, B, or C mineral filler derived from carbonate aggregate proposed for use is such that the total calcium carbonate content from cement and mineral filler is equal to or less than 15 percent by mass of the cementitious materials, then sulfate resistance can be evaluated by ASTM C1012 to comply with the expansion criteria in Table 26.4.2.2(c).

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(10) For members assigned to Exposure Category S, indicate if alternative combinations of cementitious materials qualified in accordance with 26.4.2.2(c) are permitted.

(11) Members in which calcium chloride is prohibited because of assignment to Exposure Class S2 or S3.

(12) For members assigned to Exposure Class W1 or W2, requirements for the evaluation of the potential for alkali-aggregate reactivity.

(b) At the option of the licensed design professional, exposure classes based on the severity of the anticipated exposure of members.

(c) The required compressive strength at designated stages of construction for each part of the structure designed by the licensed design professional.

26.4.2.2 Compliance requirements:

(a) The required compressive strength at designated stages of construction for each part of the structure not designed by the licensed design professional shall be submitted for review.

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=**R26.4.2.1(a)(12)** Members assigned to Exposure Class W1 or W2 are potentially susceptible to alkali-aggregate reaction. As noted in ASTM C1778, alkali-aggregate reaction (AAR) can occur between the alkali hydroxides in the pore solution of concrete and certain components found in some aggregates. Two types of AAR are recognized depending on the nature of the reactive component: alkali-silica reaction (ASR), which involves various types of reactive siliceous minerals; and alkali-carbonate reaction (ACR), which involves certain types of aggregates that contain dolomite. Both types of reaction can result in expansion and cracking of concrete elements under prolonged exposure to moisture, leading to a reduction in the structural strength and service life of a concrete structure. Options for mitigating ASR, including use of supplementary cementitious materials or limiting alkali content of the concrete, are provided in ASTM C1778. ACR can only be prevented by not using the reactive aggregate.

=**R26.4.2.1(b)** Durability requirements for concrete are based on exposure classification of members as given in 19.3. Therefore, the exposure classes applicable to the members establish the basis for the requirements for concrete mixtures. Section 19.3.1 requires the licensed design professional to assign exposure classes for different members in the structure. Concrete mixtures should be specified accordingly, but the Code does not require the assigned exposure classes to be explicitly stated in the construction documents. If the licensed design professional is requiring the contractor to determine concrete properties by specifying ACI 301M, the assigned exposure classes for all members will need to be stated explicitly in the construction documents.

R26.4.2.1(c) If design or construction requirements dictate that in-place strength of concrete be achieved at specific ages or stages of construction, these requirements should be stated explicitly in the construction documents. Typical stages of construction when the required compressive strength of concrete needs to be specified include at removal of formwork and shores. Additionally, required compressive strength of concrete should be specified for precast concrete at stripping from the forms and during handling, shipping, and erection.

For portions of the structure that are not designed by the licensed design professional, refer to 26.4.2.2(a).

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(b) For members identified in construction documents as subject to cycles of freezing and thawing and application of deicing chemicals; supplementary cementitious materials, including fly ash and natural pozzolans; silica fume; and slag cement, shall not exceed the maximum percentage allowed in Table 26.4.2.2(b) and shall satisfy (1) and (2).

(1) Supplementary cementitious materials, including fly ash and natural pozzolans, silica fume, and slag cement, used in the manufacture of ASTM C595 and C1157 blended cements shall be included in assessing compliance with the limits in Table 26.4.2.2(b).

(2) The individual limits in Table 26.4.2.2(b) shall apply regardless of the number of cementitious materials in a concrete mixture.

Table 26.4.2.2(b)—Limits on cementitious materials for concrete assigned to Exposure Class F3

Supplementary cementitious materials	Maximum percent of total cementitious materials by mass
Fly ash or natural pozzolans conforming to ASTM C618	25
Slag cement conforming to ASTM C989	50
Silica fume conforming to ASTM C1240	10
Total of fly ash or natural pozzolans and silica fume	35
Total of fly ash or natural pozzolans, slag cement, and silica fume	50

(c) For concrete mixtures for members identified in construction documents to be exposed to sulfate, alternative combinations of cementitious materials to those specified in 26.4.2.1(a)(9) are permitted if tests for sulfate resistance satisfy the criteria in Table 26.4.2.2(c).

Table 26.4.2.2(c)—Requirements for establishing suitability of combinations of cementitious materials for Exposure Category S

Exposure class		Maximum length change for tests in accordance with ASTM C1012, %		
		At 6 months	At 12 months	At 18 months
S1		0.10	No requirement	No requirement
S2		0.05	0.10*	No requirement
S3	Option 1	No requirement	No requirement	0.10
	Option 2	0.05	0.10	No requirement

*The 12-month expansion limit applies only if the measured expansion exceeds the 6-month maximum expansion limit.

(d) For concrete identified as being exposed to water in service, evidence shall be submitted that the concrete mixture complies with (1) and (2).

(1) Aggregates are not alkali-silica reactive or measures to mitigate alkali-silica reactivity have been established.

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R26.4.2.2(b) These limits on supplementary cementitious materials are applicable to concrete mixtures for members assigned to Exposure Class F3.

R26.4.2.2(c) Mixture requirements for Exposure Category S are given in 19.3.2.1. ASTM C1012 may be used to evaluate the sulfate resistance of concrete mixtures using alternative combinations of cementitious materials to those listed in Table 19.3.2.1 for all classes of sulfate exposure. More detailed guidance on qualification of such mixtures using ASTM C1012 is given in ACI 201.2R. The expansion criteria in Table 26.4.2.2(c) for testing according to ASTM C1012 are the same as those in ASTM C595 and C1157 for moderate sulfate resistance (Optional Designation MS) in Exposure Class S1 and for high sulfate resistance (Optional Designation HS) in Exposure Class S2 and Exposure Class S3 Option 2. The 18-month expansion limit only applies for Exposure Class S3, Option 1.

R26.4.2.2(d) Documentation that the potential for AAR has been evaluated can be provided by the concrete supplier. ASTM C1778 provides methods and criteria for determining the reactivity of aggregates and guidance for reducing the risk of deleterious alkali-aggregate reactions in concrete.

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(2) Aggregates are not alkali-carbonate reactive.

26.4.3 *Proportioning of concrete mixtures*

26.4.3.1 Compliance requirements:

(a) Concrete mixture proportions shall be established so that the concrete satisfies (1) through (3):

(1) Can be placed readily without segregation into forms and fully encase reinforcement.

(2) Meets durability requirements given in the construction documents.

(3) Conforms to strength test requirements for standard-cured specimens.

(4) Conforms to modulus of elasticity requirements (i) through (iii) for mixtures requiring testing in accordance with construction documents.

(i) The modulus of elasticity shall be determined as the average modulus obtained from at least three cylinders made from the same sample of concrete and tested at 28 days or at test age designated for E_c.

(ii) Cylinders used to determine modulus of elasticity shall be made and cured in the laboratory in accordance with ASTM C192 and tested in accordance with ASTM C469.

(iii) Modulus of elasticity of a concrete mixture shall be acceptable if the measured value equals or exceeds the specified value.

(b) Concrete mixture proportions shall be established in accordance with Article 4.2.3 of ACI 301M or by an alternative method acceptable to the licensed design professional. Alternative methods shall have a probability of satisfying the strength requirements for acceptance tests of standard-cured specimens that meets or exceeds the probability associated with the method in Article 4.2.3 of ACI 301M. If Article 4.2.3 of ACI 301M is used, the strength test records used for establishing and documenting concrete mixture proportions shall not be more than 24 months old.

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R26.4.3 *Proportioning of concrete mixtures*

Statistical requirements for proportioning concrete are available in other ACI documents, such as ACI 301M and ACI 214R.

R26.4.3.1(a) This section provides requirements for developing mixture proportions. The concrete is required to be workable and to meet the durability and strength requirements of the Code. The term "without segregation" is intended to provide for a cohesive mixture in which aggregates remain well distributed while the concrete is in its fresh state. It is recognized that some segregation in the form of bleeding will occur. The required workability will depend on reinforcement congestion, member geometry, and the placement and consolidation methods to be used. Construction requirements of the contractor should be considered in establishing required workability of the concrete.

The Code does not include provisions for especially severe exposures, such as chemical contact, high temperatures, temporary freezing-and-thawing conditions during construction, abrasive conditions, alkali-aggregate reactions, or other unique durability considerations pertinent to the structure. The Code also does not address aesthetic considerations such as surface finishes. If applicable, these items should be covered specifically in the construction documents.

Strength test requirements for standard-cured specimens are given in 26.12.3.

R26.4.3.1(a)(4) Modulus of elasticity testing may be required for the development of concrete mixtures to verify that specified modulus of elasticity can be obtained. It is necessary to specify both E_c and test age. Testing to verify that the specified modulus of elasticity is being attained during construction is at the discretion of the licensed design professional, including specification of acceptance criteria.

Field testing may also be required by the local building official.

R26.4.3.1(b) Article 4.2.3 of ACI 301M contains the statistical procedures for selecting the required average strength. Alternatively, the concrete producer may provide evidence acceptable to the licensed design professional that the concrete can be proportioned by another method to meet the project requirements and the acceptance criteria of 26.12.3. The Code presumes that the probability of not meeting the acceptance criteria in 26.12.3 is not more than 1 in 100. Following the method of proportioning in ACI 301M will maintain this level of risk. A key factor in evaluating any proposed alternative proportioning method should be its ability to preserve this presumed level of risk. Refer to ACI 214R for additional information.

(c) The concrete materials used to develop the concrete mixture proportions shall correspond to those to be used in the proposed Work.

(d) If different concrete mixtures are to be used for different portions of proposed Work, each mixture shall comply with the concrete mixture requirements stated in the construction documents.

R26.4.3.1(d) If more than one concrete mixture is used for the project, each mixture is required to satisfy Code requirements. A change in concrete constituents, such as sources or types of cementitious materials, aggregates, or admixtures, is considered a different mixture. A minor change in mixture proportions made in response to field conditions is not considered a new mixture.

Concrete mixture requirements to be placed in the construction documents are given in 26.4.2.1(a).

26.4.4 *Documentation of concrete mixture characteristics*

R26.4.4 *Documentation of concrete mixture characteristics*

26.4.4.1 Compliance requirements:

(a) Documentation of concrete mixture characteristics shall be submitted for review by the licensed design professional before the mixture is used and before making changes to mixtures already in use. Evidence of the ability of the proposed mixture to comply with the concrete mixture requirements in the construction documents shall be included in the documentation. The evidence shall include records of consecutive strength tests, as defined in 26.12.1.1, of the same concrete mixture used in previous projects or the results of laboratory trial batches of the proposed mixture.

R26.4.4.1(a) Review of the proposed concrete mixture is necessary to ensure that it is appropriate for the project and meets all the requirements for strength and durability as established by the licensed design professional. The licensed design professional typically reviews the documentation on a proposed concrete mixture to evaluate the likelihood that the concrete will meet the strength-test acceptance requirements of 26.12.3 and that acceptable materials are used. The statistical principles discussed in ACI 214R can be useful in evaluating the likelihood that a proposed mixture will meet the strength-test requirements of 26.12.3.

Concrete mixture requirements to be placed in the construction documents are given in 26.4.2.1(a).

(b) If field or laboratory test data are not available, and $f_c' \leq$ **34 MPa**, concrete proportions shall be based on other experience or information, if approved by the licensed design professional. If $f_c' >$ **34 MPa**, test data documenting the characteristics of the proposed mixtures are required.

R26.4.4.1(b) If $f_c' \leq$ **34 MPa** and test data are not available, concrete mixture proportions should be established to produce a sufficiently high average strength such that the likelihood that the concrete would not meet the strength acceptance criteria would be acceptably low. Guidance on an appropriate average strength is provided in ACI 214R. The purpose of this provision is to allow construction to continue when there is an unexpected interruption in concrete supply and there is not sufficient time for testing and evaluation. It also applies for a small project where the cost of trial mixture data is not justified.

(c) It shall be permitted to modify mixtures during the course of the Work. Before using the modified mixture, evidence acceptable to the licensed design professional shall be submitted to demonstrate that the modified mixture complies with the concrete mixture requirements in the construction documents.

R26.4.4.1(c) It is sometimes necessary or beneficial to adjust concrete mixtures during the course of a project. Conditions that could result in mixture adjustments include changes in concrete materials, seasonal temperature fluctuations, or changes in conveying and placing methods. Additionally, an adjustment to a concrete mixture may be required or appropriate if strength tests are lower or higher than required.

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26.5—Concrete production and construction

26.5.1 *Concrete production*

26.5.1.1 Compliance requirements:

(a) Cementitious materials and aggregates shall be stored to prevent deterioration or contamination.

(b) Material that has deteriorated or has been contaminated shall not be used in concrete.

(c) Equipment for mixing and transporting concrete shall conform to ASTM C94 or ASTM C685.

(d) Ready-mixed and site-mixed concrete shall be batched, mixed, and delivered in accordance with ASTM C94 or ASTM C685.

26.5.2 *Concrete placement and consolidation*

26.5.2.1 Compliance requirements:

(a) Debris and ice shall be removed from spaces to be occupied by concrete before placement.

(b) Standing water shall be removed from place of deposit before concrete is placed unless a tremie is to be used or unless otherwise permitted by both the licensed design professional and the building official.

(c) Equipment used to convey concrete from the mixer to the location of final placement shall have capabilities to achieve the placement requirements.

(d) Concrete shall not be pumped through pipe made of aluminum or aluminum alloys.

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R26.5—Concrete production and construction

Detailed recommendations for mixing, handling, transporting, and placing concrete are given in ACI 304R.

R26.5.1 *Concrete production*

R26.5.1.1(c) ASTM C94 and ASTM C685 address operational requirements for equipment used to produce concrete.

R26.5.1.1(d) ASTM C94 is a specification for ready mixed concrete whereby materials are primarily measured by mass (weight) and production is by batches. This is the more common method of concrete production, and it is also used in precast concrete plants. ASTM C685 is a specification for concrete where materials are measured by volume and the production is by continuous mixing. These specifications include provisions for capacity of mixers, accuracy of measuring devices, batching accuracy, mixing and delivery, and tests for evaluating the uniformity of mixed concrete.

R26.5.2 *Concrete placement and consolidation*

R26.5.2.1(a) Forms need to be cleaned before beginning to place concrete. In particular, sawdust, nails, wood pieces, and other debris that may collect inside forms need to be removed.

R26.5.2.1(b) The tremie referred to in this provision is not a short tube or "elephant trunk." It is a full-depth pipe used in accordance with accepted procedures for placing concrete under water. Information regarding placing concrete using a tremie is given in ACI 304R.

R26.5.2.1(c) The Code requires the equipment for handling and transporting concrete to be capable of supplying concrete to the place of deposit continuously and reliably under all conditions and for all methods of placement. This applies to all placement methods, including pumps, belt conveyors, pneumatic systems, wheelbarrows, buggies, crane buckets, and tremies.

R26.5.2.1(d) Loss of strength can result if concrete is pumped through pipe made of aluminum or aluminum alloy. This loss is caused by the formation of hydrogen gas generated by the reaction between the cement alkalies and the aluminum eroded from the interior of the pipe surface. The strength reduction has been shown to be as much as 50 percent (Newlon and Ozol 1969). Hence, equipment made of aluminum or aluminum alloys should not be used for

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(e) Concrete shall be placed in accordance with (1) through (5):

(1) At a rate to provide an adequate supply of concrete at the location of placement.

(2) At a rate so concrete at all times has sufficient workability such that it can be consolidated by the intended methods.

(3) Without segregation or loss of materials.

(4) Without interruptions sufficient to permit loss of workability between successive placements that would result in cold joints.

(5) Deposited as near to its final location as practicable to avoid segregation due to rehandling or flowing.

(f) Concrete that has been contaminated or has lost its initial workability to the extent that it can no longer be consolidated by the intended methods shall not be used.

(g) Retempering concrete in accordance with the limits of ASTM C94 shall be permitted unless otherwise restricted by the licensed design professional.

(h) After starting, concreting shall be carried on as a continuous operation until the completion of a panel or section, as defined by its boundaries or predetermined joints.

(i) Concrete shall be consolidated by suitable means during placement and shall be worked around reinforcement and embedments and into corners of forms.

26.5.3 *Curing concrete*

=**26.5.3.1** Design information:

(a) If supplementary tests of field-cured specimens are required to verify adequacy of curing and protection, the number and size of test specimens and the frequency of these supplementary tests.

=**26.5.3.2** Compliance requirements:

(a) Concrete, other than high-early-strength, shall be maintained at a temperature of at least 10°C and in a moist condition for at least the first 7 days after placement, except if accelerated curing is used.

(b) High-early-strength concrete shall be maintained at a temperature of at least 10°C and in a moist condition for at

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pump lines, tremies, or chutes other than short chutes such as those used to convey concrete from a truck mixer.

=**R26.5.2.1(e)** Concrete should be available at a supply rate consistent with the capacity of the placement equipment and the placement crew. Concrete supplied at a faster rate than can be accommodated by placement equipment or crew can result in loss of workability of concrete in equipment waiting to discharge. Excessive delays in the supply of concrete can cause previous placements to stiffen and result in the formation of cold joints.

Each step in the handling and transporting of concrete needs to be controlled to maintain uniformity within a batch and from batch to batch. It is important to minimize segregation of the coarse aggregate from the mortar or of water from the other ingredients.

Rehandling and transferring concrete over large distances from delivery vehicles to the point of placement in the structure can cause segregation of materials. The Code therefore requires that concrete be deposited as close to its final location as possible. However, self-consolidating concrete mixtures can be developed to flow longer distances and maintain their stability with minimal segregation. Guidance on self-consolidating concrete is provided in ACI 237R.

=**R26.5.2.1(g)** ASTM C94 permits water addition to mixed concrete before concrete is discharged to bring it up to the specified slump range as long as prescribed limits on the maximum mixing time and *w/cm* are not violated.

=**R26.5.2.1(i)** Detailed recommendations for consolidation of concrete are given in ACI 309R. This guide presents information on the mechanism of consolidation and provides recommendations on equipment characteristics and procedures for various types of concrete mixtures.

R26.5.3 *Curing concrete*

=Detailed recommendations for curing concrete are given in ACI 308R. This guide presents basic principles of proper curing and describes the various methods, procedures, and materials for curing of concrete.

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least the first 3 days after placement, except if accelerated curing is used.

(c) Accelerated curing to accelerate strength gain and reduce time of curing is permitted using high-pressure steam, steam at atmospheric pressure, heat and moisture, or other process acceptable to the licensed design professional. If accelerated curing is used, (1) and (2) shall apply:

(1) Compressive strength at the load stage considered shall be at least the strength required at that load stage.

(2) Accelerated curing shall not impair the durability of the concrete.

(d) If required by the building official or licensed design professional, results of tests for cylinders made and cured in accordance with (1) and (2) shall be provided in addition to results of standard-cured cylinders.

(1) At least two 150 x 300 mm or at least three 100 x 200 mm cylinders to be field-cured shall be molded at the same time and from the same samples as standard-cured cylinders;

(2) Field-cured cylinders shall be cured in accordance with the field curing procedure of ASTM C31 and tested in accordance with ASTM C39.

(e) Procedures for protecting and curing concrete shall be considered adequate if (1) or (2) are satisfied:

(1) Average strength of field-cured cylinders at test age designated for determination of f_c' is equal to or at least 85% of that of companion standard-cured cylinders.

(2) Average strength of field-cured cylinders at test age exceeds f_c' by more than 3.4 MPa.

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R26.5.3.2(c) This section applies whenever an accelerated curing method is used, whether for precast or cast-in-place elements. EB-001.15, and PCI MNL 116, and PCI MNL 117 provide general information on accelerated curing. Accelerated curing procedures require careful attention to obtain uniform and satisfactory results. Preventing moisture loss during the curing is essential.

The compressive strength of accelerated-cured concrete is not as high at later ages as that of nominally identical concrete continuously cured under moist conditions at moderate temperatures. Also, the modulus of elasticity, E_c, of accelerated-cured specimens may vary from that of specimens moist-cured at normal temperatures.

Accelerated curing temperatures do not pose durability concerns to GFRP bars because the bar temperatures remain below the glass transition temperature of the bars. ASTM D7957 sets the minimum glass transition temperature for GFRP bars at 100°C.

R26.5.3.2(d) Strengths of cylinders cured under field conditions may be required to evaluate the adequacy of curing and protection of concrete in the structure.

The Code provides a specific criterion in 26.5.3.2(e) for judging the adequacy of curing and protection afforded to the structure. For a valid comparison, field-cured cylinders and companion standard-cured cylinders need to be made from the same sample. Field-cured cylinders are to be cured, as nearly as possible, under the same conditions as the structure. The field-cured cylinders should not be treated more favorably than the structural members they represent.

In evaluating test results of field-cured cylinders, it should be recognized that even if cylinders are protected in the same manner as the structure, they may not experience the same temperature history as the concrete in the structure. This different temperature history occurs because heat of hydration may be dissipated differently in a cylinder compared with the structural member.

R26.5.3.2(e) Research (Bloem 1968) has shown that the strength of cylinders protected and cured to simulate good field practice should be at least about 85% of standard-cured cylinders if both are tested at the age designated for f_c'. Thus, a value of 85% has been set as a rational basis for judging the adequacy of field curing. The comparison is made between the measured strengths of companion field-cured and standard-cured cylinders, not between the strength of field-cured cylinders and the specified value of f_c'. Test results for the field-cured cylinders are considered satisfactory, however, if the strength of field-cured cylinders exceeds f_c' by more than 3.4 MPa, even though they fail to reach 85% of the strength of companion standard-cured cylinders.

The 85% criterion is based on the assumption that concrete is maintained above 10°C and in a moist condition for at least the first 7 days after placement, or high-early-strength concrete is maintained above 10°C and in a moist condition for at least the first 3 days after placement.

If the field-cured cylinders do not provide satisfactory strength by this comparison, steps need to be taken to improve the curing. If the tests indicate a possible serious deficiency in strength of concrete in the structure, core tests may be required, with or without supplemental wet curing, to evaluate the structural adequacy, as provided in 26.12.6.

26.5.4 *Concreting in cold weather*

R26.5.4 *Concreting in cold weather*

Detailed recommendations for cold weather concreting are given in ACI 306R. Specification requirements for concreting in cold weather are provided in ACI 301M and ACI 306.1. If both ACI 301M and ACI 306.1 are referenced in construction documents, the governing requirements should be identified.

26.5.4.1 Design information:
(a) Temperature limits for concrete as delivered in cold weather.

R26.5.4.1(a) ASTM C94, ACI 306R, and ACI 301M contain requirements and recommendations for concrete temperature based on section size.

26.5.4.2 Compliance requirements:
(a) Adequate equipment shall be provided for heating concrete materials and protecting concrete during freezing or near-freezing weather.
(b) Frozen materials or materials containing ice shall not be used.
(c) Forms, fillers, and ground with which concrete is to come in contact shall be free from frost and ice.
(d) Concrete materials and production methods shall be selected so that the concrete temperature at delivery complies with the specified temperature limits.

26.5.5 *Concreting in hot weather*

R26.5.5 *Concreting in hot weather*

Detailed recommendations for hot weather concreting are given in ACI 305R. This guide identifies the hot weather factors that affect concrete properties and construction practices and recommends measures to eliminate or minimize undesirable effects. Specification requirements for concreting in hot weather are provided in ACI 301M and ACI 305.1M.

26.5.5.1 Design information:
(a) Temperature limits for concrete as delivered in hot weather.

R26.5.5.1(a) ACI 301M and ACI 305.1M limit the maximum concrete temperature to 35°C at the time of placement.

26.5.5.2 Compliance requirements:
(a) Concrete materials and production methods shall be selected so that the concrete temperature at delivery complies with the specified temperature limits.
(b) Handling, placing, protection, and curing procedures shall limit concrete temperatures or water evaporation that

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could reduce strength, serviceability, and durability of the member or structure.

26.5.6 *Construction, contraction, and isolation joints*

26.5.6.1 Design information:
(a) If required by the design, locations and details of construction, isolation, and contraction joints.
(b) Details required for transfer of shear and other forces through construction joints.
(c) Surface preparation, including intentional roughening of hardened concrete surfaces where concrete is to be placed against previously hardened concrete.
(d) Surface preparation including intentional roughening if composite topping slabs are to be cast in place on a precast floor or roof intended to act structurally with the precast members.

26.5.6.2 Compliance requirements:
(a) Joint locations or joint details not shown or that differ from those indicated in construction documents shall be submitted for review by the licensed design professional.
(b) Construction joints in floor and roof systems shall be located within the middle third of spans of slabs, beams, and girders unless otherwise approved by the licensed design professional.
(c) Construction joints in girders shall be offset a distance of at least two times the width of intersecting beams, measured from the face of the intersecting beam, unless otherwise approved by the licensed design professional.
(d) Construction joints shall be cleaned and laitance removed before new concrete is placed.
(e) Surface of concrete construction joints shall be intentionally roughened if specified.
(f) Immediately before new concrete is placed, construction joints shall be prewetted and standing water removed.

26.5.7 *Construction of concrete members*

26.5.7.1 Design information:
(a) Details required to accommodate dimensional changes resulting from creep, shrinkage, and temperature.
(b) Identify if a slab-on-ground is designed as a structural diaphragm.

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R26.5.6 *Construction, contraction, and isolation joints*

For the integrity of the structure, it is important that joints in the structure be located and constructed as required by the design. Any deviations from locations indicated in construction documents should be approved by the licensed design professional.

Construction or other joints should be located where they will cause the least weakness in the structure. Lateral force design may require additional consideration of joints during design.

R26.5.6.2(a) If the licensed design professional does not designate specific joint locations, the contractor should submit joint locations for construction to the licensed design professional for review to determine that the proposed locations do not impact the performance of the structure.

R26.5.7 *Construction of concrete members*

R26.5.7.1(b) A slab-on-ground may be designed to act as a structural diaphragm or to provide required ties between foundations. The construction documents should clearly identify any slab-on-ground that is a structural diaphragm, and state that saw cutting or joints are prohibited unless approved by the licensed design professional. Joints can affect the integrity of the slab and its ability to act as a structural diaphragm, unless structural repairs are made. Refer also to 26.5.7.2(d).

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(c) Details for construction of sloped or stepped footings designed to act as a unit.
(d) Locations where floor system and column concrete placements are required to be integrated during placement in accordance with 15.3.

26.5.7.2 Compliance requirements:
(a) Beams, girders, or slabs supported by columns or walls shall not be cast until concrete in the vertical support members is no longer plastic.
(b) Beams, girders, haunches, drop panels, shear caps, and capitals shall be placed monolithically as part of a slab system.
(c) At locations where floor system and column concrete placements are required to be integrated during placement, column concrete shall extend full depth of the floor system at least 600 mm into the floor system from face of column and be integrated with floor system concrete.
(d) Saw cutting or construction of joints that can affect the integrity of a slab-on-ground identified in the construction documents as structural diaphragms shall not be permitted unless specifically indicated or approved by the licensed design professional.

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R26.5.7.2(a) Delay in placing concrete in members supported by columns and walls is necessary to minimize potential cracking at the interface of the slab and supporting member caused by bleeding and settlement of plastic concrete in the supporting member.

R26.5.7.2(c) Application of the concrete placement procedure described in 15.3 may require placing of two different concrete mixtures in the floor system. It is the responsibility of the licensed design professional to indicate in the construction documents where the higher- and lower-strength concretes are to be placed.

R26.5.7.2(d) This restriction applies to slabs identified as structural diaphragms in 26.5.7.1(b).

26.6—GFRP reinforcement materials and construction requirements

26.6.1 *General*

26.6.1.1 Design information:
(a) ASTM designation of GFRP reinforcement.
(b) Type, size, minimum values for guaranteed ultimate tensile force and tensile modulus of elasticity, location requirements, detailing, and embedment length of reinforcement.
(c) Concrete cover to reinforcement.
(d) Location and length of lap splices.
(e) Type and location of mechanical splices.
(f) Type and location of end-bearing splices.

26.6.1.2 Compliance requirements:
(a) Material certification reports for GFRP reinforcement shall be submitted.
(b) GFRP bars shall be free of mechanical damage in excess of that permitted by ACI SPEC-440.5.

R26.6—GFRP reinforcement materials and construction equipment

R26.6.1 *General*

R26.6.1.1(d) Splices should, if possible, be located away from points of maximum tensile stress. The lap splice requirements of 25.5.2 encourage this practice.

R26.6.1.2(b) Mechanical damage is interpreted as breaks to the GFRP bar surface resulting after the manufacturing process. Examples include but are not limited to gouging, dragging, and crushing. Excess resin, loose spiral windings, and absence of sand coating on the inside radius at bends are examples of items not considered to be mechanical damage.

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(c) At the time concrete is placed, reinforcement shall be clean of ice, mud, oil, or other deleterious coatings that decrease bond.

26.6.2 *Placement*

26.6.2.1 Design information:

(a) Tolerances on location of reinforcement taking into consideration tolerances on ***d*** and specified concrete cover in accordance with Table 26.6.2.1(a).

Table 26.6.2.1(a)—Tolerances on *d* and specified cover

d, mm	Tolerance on *d*, mm	Tolerance on specified concrete cover, mm*	
≤ 200	±10	Smaller of:	–10
			– (1/3) · specified cover
> 200	±13	Smaller of:	–13
			– (1/3) · specified cover

*Tolerance for cover to formed soffits is –6 mm.

(b) Tolerance for longitudinal location of bends and ends of reinforcement in accordance with Table 26.6.2.1(b). The tolerance for specified concrete cover in Table 26.6.2.1(a) shall also apply at discontinuous ends of members.

Table 26.6.2.1(b)—Tolerances for longitudinal location of bends and ends of reinforcement

Location of bends or reinforcement ends	Tolerances, mm
Discontinuous ends of brackets and corbels	±13
Discontinuous ends of other members	±25
Other locations	±50

26.6.2.2 Compliance requirements:

(a) Reinforcement shall be placed within required tolerances and supported to prevent displacement beyond required tolerances during concrete placement.

(b) Spiral units shall be continuous bar placed with even spacing and without distortion beyond the tolerances for the specified dimensions.

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R26.6.2 *Placement*

R26.6.2.1 Generally accepted practice, as reflected in ACI 117M, has established tolerances on total depth (formwork or finish) and fabrication of closed ties, stirrups, spirals, and truss bent reinforcing bars. The licensed design professional should specify more restrictive tolerances than those permitted by the Code when necessary to minimize the accumulation of tolerances resulting in excessive reduction in effective depth or cover.

More restrictive tolerances have been placed on minimum clear distance to formed soffits because of their importance for durability and fire protection and because reinforcement is usually supported in such a manner that the specified tolerance is practical.

The Code permits a reinforcement placement tolerance on effective depth ***d*** that is directly related to the flexural and shear strength of the member. Because reinforcement is placed with respect to edges of members and formwork surfaces, ***d*** is not always conveniently measured in the field. This provision is included in the design information section because tolerances on ***d*** should be considered in member design. Placement tolerances for cover are also provided.

Tolerances for placement of reinforcement should be specified in accordance with ACI 117M unless stricter tolerances are required.

R26.6.2.2(a) GFRP reinforcement should be adequately supported in the forms to prevent displacement by concrete placement or workers. Mat reinforcement should be tied down to prevent floating. Beam stirrups should be supported on the bottom form of the beam by supports such as continuous longitudinal beam bolsters. If only the longitudinal beam bottom reinforcement is supported, construction traffic can dislodge the stirrups as well as any top beam reinforcement tied to the stirrups.

R26.6.2.2(b) Spirals should be held firmly in place, at proper pitch and alignment, to prevent displacement during concrete placement. The Code has traditionally required spacers to hold the fabricated spiral cage in place, but alternate methods of installation are also permitted. If spacers are used, the following may be used for guidance: for spiral bar smaller than 16 mm diameter, a minimum of two spacers should be used for spirals less than 500 mm in diameter, three spacers for spirals 500 to 750 mm in diameter, and four spacers for spirals greater than 750 mm in diameter. For spiral bar 16 mm diameter or larger, a minimum of three spacers should be used for spirals 600 mm or less in diameter, and four spacers for spirals greater than 600 mm in diameter.

(c) Splices of reinforcement shall be made only as permitted in the construction documents, or as authorized by the licensed design professional.

(d) For longitudinal column bars forming an end-bearing splice, the bearing of square cut ends shall be held in concentric contact.

R26.6.2.2(d) Experience with end-bearing splices in steel-reinforced concrete columns has been almost exclusively with vertical bars in columns. If bars are significantly inclined from the vertical, attention is required to ensure that adequate end-bearing contact can be achieved and maintained.

(e) 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.

R26.6.2.2(e) These tolerances represent practice based on tests of full-size members containing No. M57 steel bars.

26.6.3 *Bending*

R26.6.3 *Bending*

26.6.3.1 Compliance requirements:

(a) Bends in GFRP reinforcement shall be factory formed by the GFRP bar manufacturer prior to shipment.

(b) Field bending of GFRP reinforcement shall not be permitted.

R26.6.3.1 Bending of GFRP bars must be completed during the manufacturing process prior to full cure of the polymer resin.

26.6.4 *Welding*—Not applicable

26.7—Anchoring to concrete—Out of scope

R26.7—Anchorage to concrete—Out of scope

Anchoring to concrete is not covered in this Code due to a lack of ANSI-approved material specifications for GFRP headed studs, headed bolts, hooked bolts, and anchors.

26.8—Embedments

26.8.1 Design information:

(a) Type, size, details, and location of embedments designed by the licensed design professional.

(b) Reinforcement required to be placed perpendicular to pipe embedments.

(c) Specified concrete cover for pipe embedments with their fittings.

(d) Environmental protection for exposed embedments intended to be connected with future Work.

26.8.2 Compliance requirements:

(a) Type, size, details, and location of embedments not shown in the construction documents shall be submitted for review by the licensed design professional.

(b) Aluminum embedments shall be coated or covered to prevent aluminum-concrete reaction.

(c) Pipes and fittings not shown in the construction documents shall be designed to resist effects of the material, pressure, and temperature to which they will be subjected.

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(d) No liquid, gas, or vapor, except water not exceeding 32°C or 0.34 MPa pressure, shall be placed in the pipes until the concrete has attained its specified strength.
(e) In solid slabs, piping, except for radiant heating or snow melting, shall be placed between top and bottom reinforcement.
(f) Conduit and piping shall be fabricated and installed so that cutting or displacement of reinforcement from its specified location is not required.

26.9—Additional requirements for precast concrete

26.9.1 Design information:
(a) Dimensional tolerances for precast members and interfacing members.
(b) Details of lifting devices, embedments, and related reinforcement required to resist temporary loads from handling, storage, transportation, and erection, if designed by the licensed design professional.

26.9.2 Compliance requirements:
(a) Members shall be marked to indicate location and orientation in the structure and date of manufacture.
(b) Identification marks on members shall correspond to erection drawings.
(c) Design and details of lifting devices, embedments, and related reinforcement required to resist temporary loads from handling, storage, transportation, and erection shall be provided if not designed by the licensed design professional.
(d) During erection, precast members and structures shall be supported and braced to ensure proper alignment, strength, and stability until permanent connections are completed.
(e) If approved by the licensed design professional, items embedded while the concrete is in a plastic state shall satisfy (1) through (4):
(1) Embedded items shall protrude from the precast concrete members or remain exposed for inspection.
(2) Embedded items are not required to be hooked or tied to reinforcement within the concrete.

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R26.9—Additional requirements for precast concrete

R26.9.1(a) Design of precast members and connections is particularly sensitive to tolerances on the dimensions of individual members and on their location in the structure. To prevent misunderstanding, the tolerances used in design should be specified in the construction documents. Instead of specifying individual tolerances, the standard industry tolerances assumed in design may be specified. It is important to specify any deviations from standard industry tolerances.

The tolerances required by 26.6.2 are considered a minimum acceptable standard for reinforcement in precast concrete. Industry-standard product and erection tolerances are provided in ACI ITG-7. Interfacing tolerances for precast concrete with cast-in-place concrete are provided in ACI 117M.

R26.9.1(b) If the devices, embedments, or related reinforcement are not designed by the licensed design professional, these details should be provided in shop drawings in accordance with 26.9.2(c).

R26.9.2(c) Refer to R26.9.1(b). At the option of the licensed design professional, specifications can require that shop drawings, calculations, or both be submitted for the items included in this provision when their design is delegated to the contractor.

R26.9.2(d) All temporary erection connections, bracing, and shoring as well as the sequencing of removal of these items should be shown in construction documents or erection drawings, depending on the assignment of responsibility for the means and methods of construction.

R26.9.2(e) Many precast products are manufactured in such a way that it is difficult, if not impossible, to position reinforcement that protrudes from the concrete before the concrete is placed. Such items as ties for horizontal shear and inserts can be placed while the concrete is plastic, if proper precautions are taken. This provision is not applicable to reinforcement that is completely embedded, or to

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(3) Embedded items shall be maintained in the correct position while the concrete remains plastic.
(4) The concrete shall be consolidated around embedded items.

26.10—Additional requirements for prestressed concrete—Out of scope

26.11—Formwork

26.11.1 *Design of formwork*

26.11.1.1 Design information:
(a) Requirement for the contractor to design, fabricate, install, and remove formwork.
(b) Location of composite members requiring shoring.
(c) Requirements for removal of shoring of composite members.

26.11.1.2 Compliance requirements:
(a) Design of formwork shall consider (1) through (4):
(1) Method of concrete placement.
(2) Rate of concrete placement.
(3) Construction loads, including vertical, horizontal, and impact.
(4) Avoidance of damage to previously constructed members.
(b) Formwork fabrication and installation shall result in a final structure that conforms to shapes, lines, and dimensions of the members as required by the construction documents.
(c) Formwork shall be sufficiently tight to inhibit leakage of paste or mortar.
(d) Formwork shall be braced or tied together to maintain position and shape.

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embedded items that will be hooked or tied to embedded reinforcement.

R26.10—Additional requirements for prestressed concrete—Out of scope

No additional requirements for prestressed concrete are included because this Code does not cover GFRP prestressed concrete members.

R26.11—Formwork

R26.11.1 *Design of formwork*

Typically, the contractor is responsible for formwork design, and the Code provides the minimum formwork performance requirements necessary for public health and safety. Concrete formwork design, construction, and removal demands sound judgment and planning to achieve adequate safety. Detailed information on formwork for concrete is given in "Guide to Formwork for Concrete" (ACI 347). This guide is directed primarily to contractors for design, construction, materials for formwork, and forms for unusual structures, but it should aid the licensed design professional in preparing the construction documents.

Formwork for Concrete, ACI SP-4, is a practical handbook for contractors, engineers, and architects. It follows the guidelines established in ACI 347 and includes information on planning, building, and using formwork. It also includes tables, diagrams, and formulas for formwork design loads.

ACI 301M Section 2 provides specification requirements for design and construction of formwork.

R26.11.1.1 Section 24.2.5 covers the requirements pertaining to deflections of shored and unshored members.

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26.11.2 *Removal of formwork*

26.11.2.1 Compliance requirements:

(a) Before starting construction, the contractor shall develop a procedure and schedule for removal of formwork and installation of reshores and shall calculate the loads transferred to the structure during this process.

(b) Structural analysis and concrete strength requirements used in planning and implementing the formwork removal and reshore installation shall be furnished by the contractor to the licensed design professional and to the building official, when requested.

(c) No construction loads shall be placed on, nor any formwork removed from, any part of the structure under construction except when that portion of the structure in combination with remaining formwork has sufficient strength to support safely its weight and loads placed thereon and without impairing serviceability.

(d) Sufficient strength shall be demonstrated by structural analysis considering anticipated loads, strength of formwork, and an estimate of in-place concrete strength.

(e) The estimate of in-place concrete strength shall be based on tests of field-cured cylinders or on other procedures to evaluate concrete strength approved by the licensed design professional and, when requested, approved by the building official.

(f) Formwork shall be removed in such a manner not to impair safety and serviceability of the structure.

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R26.11.2 *Removal of formwork*

R26.11.2.1 In determining the time for removal of formwork, consideration should be given to the construction loads, in-place strength of concrete, and possibility of deflections greater than acceptable to the licensed design professional (ACI 347; ACI 347.2R). Construction loads may be greater than the specified live loads. Even though a structure may have adequate strength to support the applied loads at early ages, deflections can cause serviceability problems.

The removal of formwork for multistory construction should be a part of a planned procedure developed by the contractor that considers the temporary support of the entire structure as well as each individual member. Such a procedure should be planned before construction and should be based on a structural analysis taking into account at least (a) through (e):

(a) The structural system that exists at the various stages of construction, and the construction loads corresponding to those stages;

(b) The in-place strength of the concrete at the various stages during construction;

(c) The influence of deformations of the structure and shoring system on the distribution of dead loads and construction loads during the various stages of construction;

(d) The strength and spacing of shores or shoring systems used, as well as the method of shoring, bracing, shore removal, and reshoring including the minimum time interval between the various operations;

(e) Any other loading or condition that affects the safety or serviceability of the structure during construction.

ACI 347.2R provides information for shoring and reshoring multistory buildings.

R26.11.2.1(e) Evaluation of concrete strength during construction may be demonstrated by field-cured test cylinders or other procedures approved by the licensed design professional and, when requested, approved by the building official, such as (a) though (d):

(a) Tests of cast-in-place cylinders in accordance with ASTM C873. This method is limited to use for slabs where the depth of concrete is between 130 to 300 mm

(b) Penetration resistance in accordance with ASTM C803

(c) Pullout strength in accordance with ASTM C900

(d) Maturity index measurements and correlation in accordance with ASTM C1074

Procedures (b), (c), and (d) require sufficient data for the materials used in the Work to demonstrate correlation of measurements on the structure with the compressive strength of molded cylinders or drilled cores. ACI 228.1R discusses the use of these methods to evaluate the in-place strength of concrete

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(g) Concrete exposed by formwork removal shall have sufficient strength not to be damaged by the removal.
(h) Intentionally left blank.
(i) No construction loads exceeding the combination of superimposed dead load plus live load including reduction shall be placed on any unshored portion of the structure under construction, unless analysis indicates adequate strength to support such additional loads and without impairing serviceability.

COMMENTARY

⁼**R26.11.2.1(i)** The nominal live load specified on the drawings is frequently reduced for members supporting large floor areas, and the limit on construction loads needs to account for such reductions.

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26.12—Evaluation and acceptance of hardended concrete

26.12.1 *General*

26.12.1.1 Compliance requirements:
(a) Evaluation of hardened concrete shall be based on strength tests. A strength test is the average of the compressive strengths of at least two 150 x 300 mm cylinders or at least three 100 x 200 mm cylinders made from the same sample of concrete taken in accordance with ASTM C172 at the point of delivery, handled and standard-cured in accordanced with ASTM C31, and tested in accordance with ASTM C39 at 28 days or at test age designated for f_c'.

(b) Intentionally left blank.
(c) The testing agency performing acceptance testing shall comply with ASTM C1077.

COMMENTARY

R26.12—Evaluation and acceptance of hardended concrete

R26.12.1 *General*

⁼**R26.12.1.1(a)** Casting and testing more than the minimum number of specimens may be desirable in case it becomes necessary to discard an outlying individual cylinder strength in accordance with ACI 214R. If individual cylinder strengths are discarded in accordance with ACI 214R, a strength test is valid provided at least two individual 150 x 300 mm cylinder strengths or at least three 100 x 200 mm cylinder strengths are averaged. All individual cylinder strengths that are not discarded in accordance with ACI 214R are to be used to calculate the average strength. The size and number of specimens representing a strength test should be the same for each concrete mixture. The cylinder size should be agreed upon by the owner, licensed design professional, and testing agency before construction.

Testing three instead of two 100 x 200 mm cylinders preserves the confidence level of the average strength because 100 x 200 mm cylinders tend to have approximately 20% higher within-test variability than 150 x 300 mm cylinders (Carino et al. 1994).

Representative concrete samples for making strength-test specimens are obtained from concrete as delivered to the project site. For example, samples of concrete delivered in a truck mixer would be obtained from the truck chute at discharge. ASTM C172 provides requirements for sampling concrete from different equipment used in the production or transportation of concrete.

Note that the term “strength test” does not apply to results of tests on cylinders field cured in or on the structure as described in ASTM C31, nor does it apply to results of tests on cylinders from laboratory trial batches.

⁼**R26.12.1.1(c)** ASTM C1077 defines the duties, responsibilities, and minimum technical requirements of testing agency personnel and defines the technical requirements for equipment used in testing concrete and concrete aggregates. Agencies that test cylinders or cores to determine compliance with Code requirements should be accredited or inspected for conformance to the requirements of ASTM C1077 by a recognized evaluation authority.

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(d) Qualified field testing technicians shall perform tests on fresh concrete at the job site, prepare specimens for standard curing, prepare specimens for field curing, if required, and record the temperature of the fresh concrete when preparing specimens for strength tests.

(e) Qualified laboratory technicians shall perform required laboratory tests.

(f) All reports of acceptance tests shall be provided to the licensed design professional, contractor, concrete producer, and, if requested, to the owner and the building official.

26.12.2 *Frequency of testing*

26.12.2.1 Compliance requirements:

(a) Samples for preparing strength test specimens of each concrete mixture placed each day shall be taken in accordance with (1) through (3):

(1) At least once a day.

(2) At least once for each 115 m^3 of concrete.

(3) At least once for each 465 m^2 of surface area for slabs or walls.

(b) On a given project, if total volume of concrete is such that frequency of testing would provide fewer than five strength tests for a given concrete mixture, strength test specimens shall be made from at least five randomly selected batches or from each batch if fewer than five batches are used.

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R26.12.1.1(d) Technicians can become certified through testing and training programs that include written and performance examinations. Field technicians in charge of sampling concrete; testing for slump, density (unit weight), yield, air content, and temperature; and making and curing test specimens should be certified in accordance with the ACI Concrete Field Testing Technician—Grade 1 Certification Program (ACI CPP-610.1-18), or an equivalent program meeting the requirements of ASTM C1077.

R26.12.1.1(e) Concrete laboratory testing technicians performing strength testing should be certified in accordance with the ACI Concrete Laboratory Testing Technician—Level 1 Certification Program, the ACI Concrete Strength Testing Technician Certification Program (ACI CPP-620.2-12), or an equivalent program meeting the requirements of ASTM C1077.

R26.12.1.1(f) The Code requires testing reports to be distributed to the parties responsible for the design, construction, and approval of the Work. Such distribution of test reports should be indicated in contracts for inspection and testing services. Prompt distribution of testing reports allows for timely identification of either compliance or the need for corrective action. A complete record of testing allows the concrete producer to reliably establish appropriate mixture proportions for future work.

R26.12.2 *Frequency of testing*

R26.12.2.1(a) Concrete samples for preparing strength test specimens are to be taken on a strictly random basis if they are to measure properly the acceptability of the concrete. To be representative within the period of placement, the choice of sampling times, or the concrete batches to be sampled, is to be made on the basis of chance alone. Batches are not sampled on the basis of appearance, convenience, or other possibly biased criterion, because the statistical analyses will lose their validity. ASTM D3665 describes procedures for random selection of the batches to be tested. Specimens for one strength test (as defined in 26.12.1.1(a)) are to be made from a single batch, and ASTM C172 requires that the sample be taken only after all adjustments to the batch are made.

In calculating surface area, only one side of the slab or wall is considered. Criterion (3) will require more frequent sampling than once for each 115 m^3 placed if average wall or slab thickness is less than 245 mm.

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(c) If the total quantity of a given concrete mixture is less than 38 m^3, strength tests are not required if evidence of satisfactory strength is submitted to and approved by the building official.

26.12.3 *Acceptance criteria for standard-cured specimens*

=**26.12.3.1** Compliance requirements:

(a) Specimens for acceptance tests shall be in accordance with (1) and (2):

(1) Sampling of concrete for strength test specimens shall be in accordance with ASTM C172.

(2) Cylinders for strength tests shall be made and standard-cured in accordance with ASTM C31 and tested in accordance with ASTM C39.

(b) Strength level of a concrete mixture shall be acceptable if (1) and (2) are satisfied:

(1) Every arithmetic average of any three consecutive strength tests equals or exceeds f_c'.

(2) No strength test falls below f_c' by more than 3.4 MPa if f_c' is 34 MPa or less; or by more than $0.10f_c'$ if f_c' exceeds 34 MPa

(c) If either of the requirements of 26.12.3.1(b) are not satisfied, steps shall be taken to increase the average of subsequent strength results

(d) Requirements for investigating low strength-test results shall apply if the requirements of 26.12.3.1(b)(2) are not met

26.12.4 *Acceptance criteria for shotcrete*—Out of scope

26.12.5 *Acceptance criteria for density of lightweight concrete*—Out of scope

26.12.6 *Investigation of strength-test results*

=**26.12.6.1** Compliance requirements:

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R26.12.3 *Acceptance criteria for standard-cured specimens*

=**R26.12.3.1** Evaluation and acceptance of the concrete can be judged immediately, as test results are received during the course of the Work. Strength tests failing to meet these criteria will occur occasionally, with a probability of approximately once in 100 tests (ACI 214R) even though concrete strength and uniformity are satisfactory. Allowance should be made for such statistically expected variations in deciding whether the strength level being produced is adequate. The strength acceptance criteria of 26.12.3.1(b) apply to test results from either 100 x 200 mm or 150 x 300 mm test cylinders permitted in 26.12.1.1(a). The average difference (Carino et al. 1994) between test results obtained by the two specimen sizes is not considered to be significant in design.

=**R26.12.3.1(c)** The steps taken to increase the average level of subsequent strength test results will depend on the particular circumstances but could include one or more of (a) through (g):

(a) Increase in cementitious materials content;
(b) Reduction in or better control of water content;
(c) Use of a water-reducing admixture to improve the dispersion of cementitious materials;
(d) Other changes in mixture proportions;
(e) Reduction in delivery time;
(f) Closer control of air content;
(g) Improvement in the quality of the testing, including strict compliance with ASTM C172, ASTM C31, and ASTM C39.

Such changes in operating procedures or small changes in cementitious materials content or water content should not require a formal resubmission of mixture proportions; however, changes in sources of cement, aggregates, or admixtures need to be accompanied by evidence submitted to the licensed design professional that the average concrete strength level will be improved.

R26.12.6 *Investigation of strength-test results*

=**R26.12.6.1** Requirements are provided if strength tests have failed to meet the specified acceptance criteria, specifi-

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cally 26.12.3.1(b)(2) or if the average strengths of field-cured cylinders do not comply with 26.5.3.2(e). These requirements are applicable only for evaluation of in-place strength at the time of construction. Strength evaluation of existing structures is covered by Chapter 27. The building official should apply judgment as to the significance of low test results and whether they indicate need for concern. If further investigation is deemed necessary, such investigation may include in-place tests as described in ACI 228.1R or, in extreme cases, strength tests of cores taken from the structure.

In-place tests of concrete, such as probe penetration (ASTM C803), rebound hammer (ASTM C805), or pullout test (ASTM C900), may be useful in determining whether a portion of the structure actually contains low-strength concrete. Unless these in-place tests have been correlated with compressive strength using accepted procedures such as described in ACI 228.1R, they are of value primarily for comparisons within the same structure rather than as quantitative estimates of strength.

For cores, if required, conservative acceptance criteria are provided that should ensure structural adequacy for virtually any type of construction (Bloem 1965, 1968; Malhotra 1976, 1977). Lower strength may be tolerated under many circumstances, but this is a matter of judgment on the part of the licensed design professional and building official. If the strengths of cores obtained in accordance with 26.12.6.1(d) fail to comply with 26.12.6.1(e), it may be practicable, particularly in the case of floor or roof systems, for the building official to require a strength evaluation as described in Chapter 27. Short of a strength evaluation, 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 supplemental curing should be verified by further strength evaluation using procedures previously discussed.

The Code, as stated, concerns itself with achieving structural safety, and the requirements for investigation of low strength-test results (26.12.6) are aimed at that objective. It is not the function of the Code to assign responsibility for strength deficiencies.

(a) If any strength test of standard-cured cylinders falls below f_c' by more than the limit allowed for acceptance, or if tests of field-cured cylinders indicate deficiencies in protection and curing, steps shall be taken to ensure that structural adequacy of the structure is not jeopardized.

R26.12.6.1(a) If the strength of field-cured cylinders does not conform to 26.5.3.2(e), steps need to be taken to improve the curing. If supplemental in-place tests confirm a possible deficiency in strength of concrete in the structure, core tests may be required to evaluate structural adequacy.

(b) If the likelihood of low-strength concrete is confirmed and calculations indicate that structural adequacy is significantly reduced, tests of cores drilled from the area in question in accordance with ASTM C42 shall be permitted. In such cases, three cores shall be taken for each strength test that falls below f_c' by more than the limit allowed for acceptance.

(c) The licensed design professional or the building official shall be permitted to modify details of core tests as stated in ASTM C42.

R26.12.6.1(c) Some default requirements in ASTM C42 are permitted to be altered by the "specifier of the tests," who is defined in ASTM C42 as "the individual responsible for analysis or review and acceptance of core test results." For

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(d) Cores shall be obtained, moisture-conditioned by storage in watertight bags or containers, transported to the testing agency, and tested in accordance with ASTM C42. Cores shall be tested between 5 days after last being wetted and 7 days after coring unless otherwise approved by the licensed design professional. The specifier of tests referenced in ASTM C42 shall be the licensed design professional or building official.

(e) Concrete in an area represented by core tests shall be considered structurally adequate if (1) and (2) are satisfied:

(1) The average of three cores is equal to at least 85% of f_c'.

(2) No single core is less than 75% of f_c'.

(f) Additional testing of cores extracted from locations represented by erratic core strength results shall be permitted.

(g) If criteria for evaluating structural adequacy based on core strength results are not met, and if the structural adequacy remains in doubt, the responsible authority shall be permitted to order a strength evaluation in accordance with Chapter 27 for the questionable portion of the structure or take other appropriate action.

26.12.7 *Acceptance of steel fiber-reinforced concrete*—Not applicable

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the purposes of ACI 440.11, the "specifier of the tests" is the licensed design professional or the building official.

⁼**R26.12.6.1(d)** The use of a water-cooled core barrel or a water-cooled saw for end trimming results in a core with a moisture gradient between the exterior surface and the interior. This gradient lowers the apparent compressive strength of the core (Bartlett and MacGregor 1994). The requirement of at least 5 days between the time of last being wetted and time of testing provides time for the moisture gradient to be reduced. If a water-cooled saw is used for end trimming, the conditioning period begins when sawing is completed. The maximum time of 7 days between coring and testing is intended to ensure timely testing of cores if strength of concrete is in question. If end trimming with a water-cooled saw is necessary, it should be done within 2 days of drilling the core to meet the time limits established by the testing criterion.

Research (Bartlett and MacGregor 1994) has also shown that other moisture conditioning procedures, such as soaking or air drying, affect measured core strengths and result in conditions that are not representative of the in-place concrete. Therefore, to provide reproducible moisture conditions that are representative of in-place conditions, a standard moisture conditioning procedure that permits dissipation of moisture gradients is prescribed for cores. ASTM C42 permits the specifier of tests to modify the default duration of moisture conditioning before testing. The specifier of the tests, however, must be aware of the potential reduction in strength if cores are tested before moisture gradients are allowed to dissipate.

⁼**R26.12.6.1(e)** An average core strength of 85% of the specified strength is realistic (Bloem 1968). It is not realistic, however, to expect the average core strength to be equal to f_c', because of differences in the size of specimens, conditions of obtaining specimens, degree of consolidation, and curing conditions. The acceptance criteria for core strengths have been established with consideration that cores for investigating low strength-test results will typically be extracted at an age later than specified for f_c'. For the purpose of satisfying 26.12.6.1(e), this Code does not intend that core strengths be adjusted for the age of the cores.

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26.13—Inspection

26.13.1 *General*

26.13.1.1 Concrete construction shall be inspected as required by the general building code, and as a minimum, the inspection shall comply with the requirements provided in 26.13. In the absence of a general building code, concrete construction shall be inspected in accordance with the provisions of this Code.

26.13.1.2 Inspection of concrete construction shall be conducted by the licensed design professional responsible for the design, a person under the supervision of the licensed design professional, or a qualified inspector. The inspection shall verify conformance with construction documents throughout the various Work stages. If an inspector conducts inspection of formwork, concrete placement, reinforcement, and embedments, the inspector shall be certified.

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R26.13—Inspection

R26.13.1 *General*

The quality of concrete structures depends largely on workmanship in construction. The best materials and design practices will not be effective unless construction is performed well. Inspection is necessary to verify that construction is in accordance with construction documents. Proper performance of the structure depends on construction that accurately represents the design and meets Code requirements.

Some general building codes have incorporated inspection requirements based upon established procedures such as PCI Plant Certification.

R26.13.1.1 By inspection, this Code does not intend that the inspector should supervise the construction. Rather, it means the inspector should visit the project as necessary to observe the various stages of Work and determine that it is being performed in conformance with the construction documents. The frequency of inspections should follow 26.13.3 for items requiring continuous or periodic inspection.

Inspection does not relieve the contractor from the obligation to follow the construction documents and to provide the designated quality and quantity of materials and workmanship for all stages of the Work.

This Code prescribes minimum requirements for inspection of all structures within its scope. This Code is not a construction specification and any user of this Code may require higher standards of inspection than cited in the general building code or this Code if additional requirements are necessary. ACI 311.4R describes the recommended procedure for organizing and conducting concrete inspection of steel-reinforced concrete structures and serves as a guide to owners, architects, and engineers. ACI SP-2 describes methods of inspecting steel-reinforced concrete construction that are generally accepted as good practice and serves as a guide in matters not covered by construction documents.

R26.13.1.2 The licensed design professional responsible for the design is in the best position to determine if construction is in conformance with construction documents. However, if the licensed design professional responsible for the design is not retained, inspection of construction through other licensed design professionals or through separate inspection organizations with demonstrated capability for performing the inspection may be used.

Inspectors should establish their capability of performing inspection requirements by becoming certified to inspect and record the results of concrete construction, including pre-placement, placement, and post-placement through the ACI Concrete Construction Special Inspector Certification Program (ACI CPP-630.1-15), or equivalent.

In some jurisdictions, legislation has established registration or licensing procedures for persons performing certain inspection functions. The general building code should be

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reviewed or the building official should be consulted to determine if any such requirements exist within a specific jurisdiction. The building official may be contacted for clarification of the inspection requirements if not clearly identified in the general building code.

If inspection is conducted independently of the licensed design professional responsible for the design, it is recommended that the licensed design professional responsible for the design review inspection reports and observe portions of the Work to verify that the design requirements are properly executed.

Inspection reports should be distributed promptly to the owner, licensed design professional responsible for the design, contractor, appropriate subcontractors, appropriate suppliers, and the building official to allow timely identification of compliance or the need for corrective action.

Inspection responsibility and the degree of inspection required should be set forth in the contracts between the owner, architect, engineer, contractor, and inspector. Adequate resources should be provided to properly perform and oversee the inspection.

26.13.2 *Inspection reports*

=**26.13.2.1** Inspection reports shall document inspected items and be developed throughout each construction Work stage. Records of the inspection shall be preserved by the party performing the inspection for at least 2 years after completion of the project.

26.13.2.2 Inspection reports shall document (a) through (d):

(a) General progress of the Work.

(b) Any significant construction loadings on completed floors, members, or walls.

(c) The date and time of mixing, quantity of concrete placed, approximate placement location in the structure, and results of tests for fresh and hardened concrete properties for all concrete mixtures used in the Work.

(d) Concrete temperatures and protection given to concrete during placement and curing when the ambient temperature falls below 4.4°C or rises above 35°C.

R26.13.2 *Inspection reports*

=**R26.13.2.1** A record of inspection is required in case questions subsequently arise concerning the performance or safety of the structure or members. Photographs documenting construction progress are also desirable.

The general building code or other legal documents may require these records be preserved longer than 2 years after completion of the project.

=**R26.13.2.2(d)** The term "ambient temperature" means the temperature of the environment to which the concrete is directly exposed. Concrete temperature as used in this section may be taken as the surface temperature of the concrete. Surface temperatures may be determined by placing temperature sensors in contact with concrete surfaces or between concrete surfaces and covers used for curing, such as insulation blankets or plastic sheeting.

26.13.3 *Items requiring inspection*

=**26.13.3.1** Unless otherwise specified in the general building code, items shall be continuously or periodically inspected in accordance with 26.13.3.2 and 26.13.3.3, respectively.

R26.13.3 *Items requiring inspection*

=**R26.13.3.1** Table 1705 in Chapter 17 of the 2012 IBC was used to determine which items of Work require continuous or periodic inspection.

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26.13.3.2 Items requiring continuous inspection shall include concrete mixture for intended location prior to placement.

26.13.3.3 Items requiring periodic inspection shall include (a) through (e):

(a) Placement of reinforcement and embedments

(b) Curing method and duration of curing for each member

(c) Construction and removal of forms and reshoring

(d) Sequence of erection and connection of precast members

(e) Verification of in-place concrete strength before removal of shores and formwork from beams and structural slabs.

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R26.13.3.3(d) Some jurisdictions may require continuous inspection of sequence of erection and connection of precast members, and also may require inspection of the shoring, bracing, or other temporary measures.

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CHAPTER 27—STRENGTH EVALUATION OF EXISTING CONCRETE STRUCTURES

27.1—Scope

⁼**27.1.1** Provisions of this chapter shall apply to strength evaluation of existing structures by analytical means or by load testing.

27.2—General

⁼**27.2.1** If there is doubt that a part or all of a structure meets the safety requirements of this Code and the structure is to remain in service, a strength evaluation shall be carried out as required by the licensed design professional or building official.

⁼**27.2.2** If the effect of a strength deficiency is well understood and it is practical to measure the dimensions and determine the material properties of the members required for analysis, an analytical evaluation of strength based on this information is permitted. Required data shall be determined in accordance with 27.3.

⁼**27.2.3** If the effect of a strength deficiency is not well understood or it is not practical to measure the dimensions and determine the material properties of the members required for analysis, a load test is required in accordance with 27.4.

27.2.4 If uncertainty about the strength of part or all of a structure involves deterioration, and if the observed response during the load test satisfies the acceptance criteria in 27.5 for the selected load test procedure, the structure or part of the structure is permitted to remain in service for a time period specified by the licensed design professional. If deemed necessary by the licensed design professional, periodic reevaluations shall be conducted.

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CHAPTER R27—STRENGTH EVALUATION OF EXISTING STRUCTURES

R27.1—Scope

⁼**R27.1.1** Provisions of this chapter may be used to evaluate whether a structure or a portion of a structure satisfies the safety requirements of the Code. A strength evaluation may be required if the materials are considered to be deficient in quality, if there is evidence indicating faulty construction, if a building will be used for a new function, or if, for any reason, a structure or a portion of it does not appear to satisfy the requirements of the Code. In such cases, this chapter provides guidance for investigating the safety of the structure. This chapter does not cover load testing for the approval of new design or construction methods. Acceptance of alternative materials or systems is covered in 1.10.

R27.2—General

⁼**R27.2.1** If a load test is described as part of the strength evaluation process, it is desirable for all parties to agree on the region to be loaded, the magnitude of the load, the load test procedure, and acceptance criteria before any load tests are conducted. If the safety concerns are related to an assemblage of members or an entire structure, it is not feasible to load test every member and section. In such cases, it is appropriate that an investigation plan be developed to address the specific safety concerns.

⁼**R27.2.2** Strength considerations related to axial load, flexure, and combined axial load and flexure are well understood. There are reliable theories relating strength and short-term displacement to load in terms of member dimensional and material data. To determine the strength of the structure by analysis, calculations should be based on data gathered on the actual dimensions of the structure, properties of the materials in place, and all pertinent details.

⁼**R27.2.3** If the shear or bond strength of a member is critical in relation to the doubt expressed about safety, a test may be the most efficient solution to eliminate or confirm the doubt. A test may also be appropriate if it is not feasible to determine the material and dimensional properties required for analysis, even if the cause of the concern relates to flexure or axial load. Wherever possible and appropriate, the results of the load test should be supported by analysis.

⁼**R27.2.4** For a deteriorating structure, acceptance provided by the load test is, by necessity, limited in terms of future service life. In such cases, a periodic inspection program is useful. A program that involves physical tests and periodic inspection can justify a longer period in service. Another option for maintaining the structure in service, while the periodic inspection program continues, is to limit the live load to a level determined to be appropriate in accordance with 27.2.5. The length of the specified time period between inspections should be based on consideration of:

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a) the nature of the deterioration; b) environmental and load effects; c) service history of the structure; and d) scope of the periodic inspection program. At the end of a specified time period, further strength evaluation is required if the structure is to remain in service. With the agreement of all concerned parties, procedures may be devised for periodic testing that do not necessarily conform to the loading and acceptance criteria specified within this chapter.

27.2.5 If the structure under investigation does not satisfy conditions or criteria of 27.3 or 27.5, the structure shall be permitted for use at a lower load rating, based on the results of the load test or analysis, and if approved by the building official.

R27.2.5 Except for load tested members that have failed under a test (refer to 27.4.5), the building official may permit the use of a structure or member at a lower load rating that is judged to be safe and appropriate on the basis of the strength evaluation.

27.3—Analytical strength evaluation

27.3.1 *Verification of as-built condition*

27.3.1.1 As-built dimensions of members shall be field-verified at critical sections.

R27.3—Analytical strength evaluation

R27.3.1 *Verification of as-built condition*

R27.3.1.1 As-built dimensions at critical locations requiring field verification are those dimensions necessary to quantify the performance at those sections. Critical sections for different load effects, such as moment, shear force, and axial force, are locations where stresses caused by such effects reach their maximum value and as further defined for various member types in the Code. Additionally, critical sections may be defined by specific conditions in the structure being evaluated, such as localized member deterioration.

27.3.1.2 Locations and sizes of reinforcement shall be determined by measurement. It shall be permitted to base reinforcement locations on available drawings if field-verified at representative locations to confirm the information on the drawings.

R27.3.1.2 If investigating individual members, the amount, size, arrangement, and location of reinforcement designed to resist applied load should be determined at the critical sections. In structures with many critical sections, the frequency of measurements may be reduced if the field measurements are consistent.

27.3.1.3 If required, an estimated equivalent f_c' shall be based on analysis of results of cylinder tests from the original construction, tests of cores removed from the structure, or both sets of data. Original cylinder data and core test data shall be representative of the area of concern.

R27.3.1.3 Guidance on estimating equivalent f_c' from original cylinder data can be found in Bartlett (2012).

ACI Committee 214 has developed two methods for determining an equivalent f_c' from cores taken from an existing structure. These methods are described in ACI 214.4R and rely on statistical analysis techniques. The procedures described are only appropriate where the determination of an equivalent f_c' is necessary for the strength evaluation of an existing structure and should not be used to investigate low cylinder strength test results in new construction, which is considered in 26.12.6. The number of core tests may depend on the size of the structure and the sensitivity of structural safety to concrete strength.

27.3.1.4 The method for obtaining and testing cores shall be in accordance with ASTM C42.

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27.3.1.5 The properties of reinforcement are permitted to be based on tensile tests in accordance with ASTM D7205 of representative samples of the material in the structure.

27.3.2 *Strength reduction factors*

27.3.2.1 If dimensions, size, and location of reinforcement, and material properties are determined in accordance with 27.3.1, it is permitted to increase ϕ from the design values elsewhere in this Code; however, ϕ shall not exceed the limits in Table 27.3.2.1.

Table 27.3.2.1—Maximum permissible strength reduction factors

Strength	Classification	Maximum permissible ϕ
Flexure, axial, or both	Tension controlled	0.6
	Compression controlled	0.8
Shear, torsion, or both		0.8
Bearing		0.8

27.4—Strength evaluation by load test

27.4.1 Load tests shall be conducted monotonically in accordance with 27.5.

27.4.2 Load tests shall be conducted in a manner that provides for safety of life and the structure during the test.

27.4.3 Safety measures shall not interfere with the load test or affect the results.

27.4.4 The portion of the structure subject to the test load shall be at least 56 days old. If the owner of the structure, the contractor, the licensed design professional, and all other involved parties agree, it shall be permitted to perform the load test at an earlier age.

27.4.5 A precast member to be made composite with cast-in-place concrete shall be permitted to be tested in flexure as a precast member alone in accordance with (a) and (b):

(a) Test loads shall be applied only when calculations indicate the isolated precast member will not fail by compression or buckling.
(b) The test load, when applied to the precast member alone, shall induce the same total force in the tensile reinforcement as would be produced by loading the composite member with the test load in accordance with 27.4.6.

27.4.6 *Test load arrangement and load factors*

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R27.3.1.5 The number of tests required depends on the uniformity of the material within the structure and should be determined by the licensed design professional responsible for the evaluation. The properties of the reinforcement should include ultimate strength and elastic modulus.

R27.3.2 *Strength reduction factors*

R27.3.2.1 The strength reduction factors are larger than those defined in Chapter 21. These increased values are justified by the use of field-obtained material properties and actual in-place dimensions.

R27.4—Strength evaluation by load test

R27.4.4 Other involved parties may include building officials, concrete subcontractors, and persons with a future interest in the structure.

R27.4.6 *Test load arrangement and load factors*

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27.4.6.1 Test load arrangements shall be selected to maximize the load effects in the critical regions of the members being evaluated.

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R27.4.6.1 It is important to apply the load at locations so the effects on the suspected deficiency are a maximum and sharing of the applied load with unloaded members is minimized. In cases where it is shown by analysis that adjoining unloaded members will help resist some of the load, the test load should be adjusted to produce appropriate load effects in the critical region of the members being evaluated.

27.4.6.2 The total test load $\boldsymbol{T_t}$, including dead load already in place, shall be at least the greatest of (a), (b), and (c):

(a) $T_t = 1.0D_w + 1.1D_s + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$ (27.4.6.2a)

(b) $T_t = 1.0D_w + 1.1D_s + 1.0L + 1.6(L_r \text{ or } S \text{ or } R)$ (27.4.6.2b)

(c) $T_t = 1.3(D_w + D_s)$ (27.4.6.2c)

R27.4.6.2 Test loads are consistent with the requirements in ACI 437.2M for tests on a portion of a structure and for statically indeterminate structures. The test load separates the dead load into self-weight dead load and the superimposed dead load on the structure during the load test. ACI 437.1R provides additional discussion of test loads for concrete structures.

27.4.6.3 It is permitted to reduce $\boldsymbol{L}$ in 27.4.6.2 in accordance with the general building code.

R27.4.6.3 The live load $\boldsymbol{L}$ may be reduced as permitted by the general building code governing safety considerations for the structure. The test load should be increased to compensate for resistance provided by unloaded portions of the structure in question. The increase in test load is determined from analysis of the loading conditions in relation to the selected pass/fail criterion for the test.

27.4.6.4 The load factor on the live load $\boldsymbol{L}$ in 27.4.6.2(b) shall be permitted to be reduced to 0.5 except for parking structures, areas occupied as places of public assembly, or areas where $\boldsymbol{L}$ is greater than 490 kg/m^2.

27.4.6.5 Unless documentation or tests are available to confirm the density of normalweight concrete used in the structure, the density shall be taken as 2400 kg/m^3. For other types of concrete materials, the density shall be determined based upon test results or from other documentation.

R27.4.6.5 The calculation of $\boldsymbol{D_w}$ may include determination of the weight of bonded concrete materials, such as a topping slab to be placed on precast members, not present during a load test. $\boldsymbol{D_s}$ may also include the weight from structural framing members.

27.5—Monotonic load test procedure

27.5.1 *Test load application*

27.5.1.1 Total test load $\boldsymbol{T_t}$ shall be applied in at least four approximately equal increments.

27.5.1.2 Uniform $\boldsymbol{T_t}$ shall be applied in a manner that ensures uniform distribution of the load transmitted to the structure or portion of the structure being tested. Arching action in the test load apparatus shall be avoided.

27.5.1.3 After the final load increment is applied, $\boldsymbol{T_t}$ shall remain on the structure for at least 24 hours unless signs of distress, as noted in 27.5.3, are observed.

R27.5—Monotonic load test procedure

R27.5.1 *Test load application*

R27.5.1.1 Inspecting the area of the structure subject to test loading for signs of distress after each load increment is advisable (refer to R27.5.3.1).

R27.5.1.2 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, arching of bricks in contact would result in reduction of the load on the slab near the midspan of the slab.

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=**27.5.1.4** After all response measurements are recorded, the test load shall be removed as soon as practical.

27.5.2 *Response measurements*

=**27.5.2.1** Response measurements, such as deflection, strain, slip, and crack width, shall be made at locations where maximum response is expected. Additional measurements shall be made if required.

=**27.5.2.2** The initial value for all applicable response measurements shall be obtained not more than 1 hour before applying the first load increment.

=**27.5.2.3** A set of response measurements shall be recorded after each load increment is applied and after $\boldsymbol{T_t}$ has been applied on the structure for at least 24 hours.

=**27.5.2.4** A set of final response measurements shall be made 24 hours after $\boldsymbol{T_t}$ is removed.

27.5.3 *Acceptance criteria*

=**27.5.3.1** The portion of the structure tested shall show no spalling or crushing of concrete, or other evidence of failure.

=**27.5.3.2** Members tested shall not exhibit cracks indicating imminent shear failure.

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R27.5.3 *Acceptance criteria*

=**R27.5.3.1** Evidence of failure includes distress (cracking, spalling, or deflection) of such magnitude and extent that the observed result is obviously excessive and incompatible with the safety requirements of the structure. No simple rules have been developed for application to all types of structures and conditions. If sufficient damage has occurred so that the structure is considered to have failed that test, retesting is not permitted 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 structural strength and behavior are satisfactory. However, accurate prediction or measurement of crack widths in structural concrete members is not likely to be achieved under field conditions. It is advisable to establish criteria before the test relative to the types of cracks anticipated; where the cracks will be measured; how they will be measured; and approximate limits or criteria to evaluate new cracks or limits for the changes in crack width.

R27.5.3.2 Forces are transmitted across a shear crack plane by aggregate interlock at the interface of the crack that is enhanced by clamping action of transverse reinforcement and by dowel action of stirrups crossing the crack. The member is assumed to be approaching imminent shear failure when 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, exhibit

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excessive deformation, rupture, or display loss of anchorage so as to threaten their integrity.

27.5.3.3 In regions of members without transverse reinforcement, structural cracks inclined to the longitudinal axis and having a horizontal projection greater than the depth of the member shall be evaluated. For variable-depth members, the depth shall be measured at the midlength of the crack.

R27.5.3.3 Inclined cracks may lead to brittle failure of members without transverse reinforcement. Assessment of all inclined cracks is advisable where transverse reinforcement is not present.

27.5.3.4 In regions of anchorage and lap splices of reinforcement, short inclined cracks or horizontal cracks along the line of reinforcement shall be evaluated.

R27.5.3.4 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 impending brittle failure of the member if they are associated with the development of main reinforcement. It is important that their causes and consequences be evaluated.

27.5.3.5 Measured deflections shall satisfy:

$$\Delta_r \leq \frac{\Delta_1}{5} \qquad (27.5.3.5)$$

R27.5.3.5 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. The recovery criterion for GFRP-reinforced concrete members is more restrictive than for steel-reinforced concrete members due to the increased elastic recovery expected from GFRP reinforcement. Unlike steel reinforcement that yields, GFRP bars remain linear elastic up to failure and return to their original length upon unloading, reducing the amount of permanent deformation.

27.5.3.6 If the maximum deflection measured during the test, Δ_1, does not exceed the larger of 1.3 mm or ℓ_t**/2000**, the residual deflection requirements in 27.5.3.5 shall be permitted to be waived.

R27.5.3.6 In the case of a very stiff structure, 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 does not exceed the larger of 1.3 mm or ℓ_t**/2000**.

27.5.3.7 If 27.5.3.5 or 27.5.3.6 is not satisfied, it shall be permitted to repeat the load test, provided that the second load test begins no earlier than 72 hours after removal of externally applied loads from the first load test.

27.5.3.8 Portions of the structure tested in the second load test shall be considered acceptable if:

$$\Delta_r \leq \frac{\Delta_2}{6} \qquad (27.5.3.8)$$

APPENDIX—EQUIVALENCE BETWEEN SI-METRIC, MSK-METRIC, AND U.S. CUSTOMARY UNITS OF NONHOMOGENOUS EQUATIONS IN THE CODE

Provision number	SI-metric stress in MPa	mks-metric stress in kgf/cm²	U.S. Customary units stress in pounds per square inch (psi)
6.6.4.5.4	$M_{2,min} = P_u(15 + 0.03h)$	$M_{2,min} = P_u(1.5 + 0.03h)$	$M_{2,min} = P_u(0.6 + 0.03h)$
7.6.1.1	$A_{f,\min} = \frac{2.1}{f_{fu}} A_g$	$A_{f,\min} = \frac{21}{f_{fu}} A_g$	$A_{f,\min} = \frac{300}{f_{fu}} A_g$
9.6.1.2(a)	$\frac{0.41\sqrt{f_c'}}{f_{fu}} b_w d$	$\frac{1.3\sqrt{f_c'}}{f_{fu}} b_w d$	$\frac{4.9\sqrt{f_c'}}{f_{fu}} b_w d$
9.6.1.2(b)	$\frac{2.3}{f_{fu}} b_w d$	$\frac{23}{f_{fu}} b_w d$	$\frac{330}{f_{fu}} b_w d$
9.6.3.4(a)	$0.062\sqrt{f_c'} \frac{b_w}{f_{ft}} s$	$0.2\sqrt{f_c'} \frac{b_w}{f_{ft}} s$	$0.75\sqrt{f_c'} \frac{b_w}{f_{ft}} s$
9.6.3.4(b)	$0.35 \frac{b_w}{f_{ft}} s$	$3.5 \frac{b_w}{f_{ft}} s$	$50 \frac{b_w}{f_{ft}} s$
9.6.4.2(a)	$0.062\sqrt{f_c'} \frac{b_w}{f_{ft}}$	$0.2\sqrt{f_c'} \frac{b_w}{f_{ft}}$	$0.75\sqrt{f_c'} \frac{b_w}{f_{ft}}$
9.6.4.2(b)	$0.35 \frac{b_w}{f_{ft}}$	$3.5 \frac{b_w}{f_{ft}}$	$50 \frac{b_w}{f_{ft}}$
9.6.4.3(a)	$\frac{0.42\sqrt{f_c'}}{f_{fu}} A_{cp} - \left(\frac{A_{fv}}{s}\right) p_h \frac{f_{ft}}{f_{fu}}$	$\frac{1.33\sqrt{f_c'}}{f_{fu}} A_{cp} - \left(\frac{A_{fv}}{s}\right) p_h \frac{f_{ft}}{f_{fu}}$	$\frac{5\sqrt{f_c'}}{f_{fu}} A_{cp} - \left(\frac{A_{fv}}{s}\right) p_h \frac{f_{ft}}{f_{fu}}$
9.6.4.3(b)	$\frac{0.42\sqrt{f_c'}}{f_{fu}} A_{cp} - \left(\frac{0.175 b_w}{f_{fv}}\right) p_h \frac{f_{ft}}{f_{fu}}$	$\frac{1.33\sqrt{f_c'}}{f_{fu}} A_{cp} - \left(\frac{1.75 b_w}{f_{fv}}\right) p_h \frac{f_{ft}}{f_{fu}}$	$\frac{5\sqrt{f_c'}}{f_{fu}} A_{cp} - \left(\frac{25 b_w}{f_{fv}}\right) p_h \frac{f_{ft}}{f_{fu}}$
9.7.6.2.2	$0.33\sqrt{f_c'} b_w d$	$1.1\sqrt{f_c'} b_w d$	$4\sqrt{f_c'} b_w d$
10.6.2.2(a)	$0.062\sqrt{f_c'} \frac{b_w s}{f_{ft}}$	$0.2\sqrt{f_c'} \frac{b_w s}{f_{ft}}$	$0.75\sqrt{f_c'} \frac{b_w s}{f_{ft}}$
10.6.2.2(b)	$0.35 \frac{b_w s}{f_{ft}}$	$3.5 \frac{b_w s}{f_{ft}}$	$50 \frac{b_w s}{f_{ft}}$
15.4.2(a)	$0.062\sqrt{f_c'} \frac{b_w s}{f_{ft}}$	$0.2\sqrt{f_c'} \frac{b_w s}{f_{ft}}$	$0.75\sqrt{f_c'} \frac{b_w s}{f_{ft}}$

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15.4.2(b)	$0.35\dfrac{b_w s}{f_{ft}}$	$3.5\dfrac{b_w s}{f_{ft}}$	$50\dfrac{b_w s}{f_{ft}}$
19.2.2.1(a)	$E_c = w_c^{1.5}0.043\sqrt{f_c'}$	$E_c = w_c^{1.5}0.14\sqrt{f_c'}$	$E_c = w_c^{1.5}33\sqrt{f_c'}$
19.2.2.1(b)	$E_c = 4700\sqrt{f_c'}$	$E_c = 15{,}100\sqrt{f_c'}$	$E_c = 57{,}000\sqrt{f_c'}$
19.2.3.1	$f_r = 0.62\sqrt{f_c'}$	$f_r = 2\sqrt{f_c'}$	$f_r = 7.5\sqrt{f_c'}$
22.2.2.4.3(b)	$0.85-\dfrac{0.05(f_c'-28)}{7}$	$0.85-\dfrac{0.05(f_c'-280)}{70}$	$0.85-\dfrac{0.05(f_c'-4000)}{1000}$
22.5.5.1(a)	$V_c = 0.42\lambda_s k_{cr}\sqrt{f_c'}b_w d$	$V_c = 1.3\lambda_s k_{cr}\sqrt{f_c'}b_w d$	$V_c = 5\lambda_s k_{cr}\sqrt{f_c'}b_w d$
22.5.5.1(b)	$V_c = 0.066\lambda_s\sqrt{f_c'}b_w d$	$V_c = 0.21\lambda_s\sqrt{f_c'}b_w d$	$V_c = 0.8\lambda_s\sqrt{f_c'}b_w d$
22.5.5.1.3	$\sqrt{\dfrac{2}{1+0.004d}}\le 1.0$	$\sqrt{\dfrac{2}{1+0.04d}}\le 1.0$	$\sqrt{\dfrac{2}{1+\left(\dfrac{d}{10}\right)}}\le 1.0$
22.6.3.1	$\sqrt{f_c'}\le 8.3$ MPa	$\sqrt{f_c'}\le 27$ kgf/cm^2	$\sqrt{f_c'}\le 100$ psi
22.6.5.2(a)	$v_c = 0.83\lambda_s k_{cr}\sqrt{f_c'}$	$v_c = 2.7\lambda_s k_{cr}\sqrt{f_c'}$	$v_c = 10\lambda_s k_{cr}\sqrt{f_c'}$
22.6.5.2(b)	$v_c = 0.13\lambda_s\sqrt{f_c'}$	$v_c = 0.42\lambda_s\sqrt{f_c'}$	$v_c = 1.6\lambda_s\sqrt{f_c'}$
22.7.2.1	$\sqrt{f_c'}\le 8.3$ MPa	$\sqrt{f_c'}\le 27$ kgf/cm^2	$\sqrt{f_c'}\le 100$ psi
22.7.4.1(a)(a)	$0.083\sqrt{f_c'}\left(\dfrac{A_{cp}^2}{p_{cp}}\right)$	$0.27\sqrt{f_c'}\left(\dfrac{A_{cp}^2}{p_{cp}}\right)$	$\sqrt{f_c'}\left(\dfrac{A_{cp}^2}{p_{cp}}\right)$
22.7.4.1(a)(b)	$0.083\sqrt{f_c'}\left(\dfrac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\dfrac{N_u}{0.33A_g\sqrt{f_c'}}}$	$0.27\sqrt{f_c'}\left(\dfrac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\dfrac{N_u}{1.1A_g\sqrt{f_c'}}}$	$\sqrt{f_c'}\left(\dfrac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\dfrac{N_u}{4A_g\sqrt{f_c'}}}$
22.7.4.1(b)(a)	$0.083\sqrt{f_c'}\left(\dfrac{A_g^2}{p_{cp}}\right)$	$0.27\sqrt{f_c'}\left(\dfrac{A_g^2}{p_{cp}}\right)$	$\sqrt{f_c'}\left(\dfrac{A_g^2}{p_{cp}}\right)$
22.7.4.1(b)(b)	$0.083\sqrt{f_c'}\left(\dfrac{A_g^2}{p_{cp}}\right)\sqrt{1+\dfrac{N_u}{0.33A_g\sqrt{f_c'}}}$	$0.27\sqrt{f_c'}\left(\dfrac{A_g^2}{p_{cp}}\right)\sqrt{1+\dfrac{N_u}{1.1A_g\sqrt{f_c'}}}$	$\sqrt{f_c'}\left(\dfrac{A_g^2}{p_{cp}}\right)\sqrt{1+\dfrac{N_u}{4A_g\sqrt{f_c'}}}$

22.7.5.1(a)	$0.33\sqrt{f'_c}\left(\frac{A_{cp}^2}{p_{cp}}\right)$	$1.1\sqrt{f'_c}\left(\frac{A_{cp}^2}{p_{cp}}\right)$	$4\sqrt{f'_c}\left(\frac{A_{cp}^2}{p_{cp}}\right)$
22.7.5.1(c)	$0.33\sqrt{f'_c}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{0.33A_g\sqrt{f'_c}}}$	$1.1\sqrt{f'_c}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{1.1A_g\sqrt{f'_c}}}$	$4\sqrt{f'_c}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{4A_g\sqrt{f'_c}}}$
24.3.2(a)	$s \le \frac{0.81E_f}{f_{fs}k_b} - 2.5c_c$	$s \le \frac{0.081E_f}{f_{fs}k_b} - 2.5c_c$	$s \le \frac{0.032E_f}{f_{fs}k_b} - 2.5c_c$
24.3.2(b)	$s \le \frac{0.66E_f}{f_{fs}k_b}$	$s \le \frac{0.066E_f}{f_{fs}k_b}$	$s \le \frac{0.026E_f}{f_{fs}k_b}$
24.3.2.2	$f_{fs} \le \frac{0.36E_f}{d_c\beta_c k_b}$	$f_{fs} \le \frac{0.036E_f}{d_c\beta_c k_b}$	$f_{fs} \le \frac{0.014E_f}{d_c\beta_c k_b}$
24.4.3.2	$140/E_f$	$1400/E_f$	$20{,}000/E_f$
25.4.1.4	$\sqrt{f'_c} \le 8.3$ MPa	$\sqrt{f'_c} \le 27$ kgf/cm^2	$\sqrt{f'_c} \le 100$ psi
25.4.2.4	$\ell_d = \frac{d_b\left(\frac{f_{fr}}{0.083\sqrt{f'_c}} - 340\right)\psi_t}{13.6+\frac{c_b}{d_b}}$	$\ell_d = \frac{d_b\left(\frac{f_{fr}}{0.27\sqrt{f'_c}} - 340\right)\psi_t}{13.6+\frac{c_b}{d_b}}$	$\ell_d = \frac{d_b\left(\frac{f_{fr}}{\sqrt{f'_c}} - 340\right)\psi_t}{13.6+\frac{c_b}{d_b}}$
25.4.3.1	$165\frac{d_b}{\sqrt{f'_c}}$ $\frac{f_{fu}}{3.1}\frac{d_b}{\sqrt{f'_c}}$ $330\frac{d_b}{\sqrt{f'_c}}$	$530\frac{d_b}{\sqrt{f'_c}}$ $\frac{f_{fu}}{10}\frac{d_b}{\sqrt{f'_c}}$ $1060\frac{d_b}{\sqrt{f'_c}}$	$2000\frac{d_b}{\sqrt{f'_c}}$ $\frac{f_{fu}}{37.5}\frac{d_b}{\sqrt{f'_c}}$ $4000\frac{d_b}{\sqrt{f'_c}}$

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