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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

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ACI CODE-440.11-22

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

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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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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.

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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 $\mathbf{0.85}f_c'$ 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', psi	β_1	
$3000 \leq f_c' \leq 4000$	0.85	(a)
$4000 < f_c' \leq 8000$	$0.85 - \dfrac{0.05\,(f_c' - 4000)}{1000}$	(b)
$f_c' \geq 8000$	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 8000 psi (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 $\boldsymbol{c}$. 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 ($\boldsymbol{\varepsilon_c < \varepsilon_{cu}}$). In this case, equivalent rectangular stress block parameters (the ratio of the average concrete stress to the concrete strength $\boldsymbol{\alpha_1}$ and the ratio of the depth of the equivalent rectangular stress block to the depth of the neutral axis $\boldsymbol{\beta_1}$) 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 $\boldsymbol{\beta_1 c}$ 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 $\boldsymbol{\beta_1 c_{bal}}$ and is achieved if the maximum concrete strain ($\boldsymbol{\varepsilon_{cu} = 0.003}$) 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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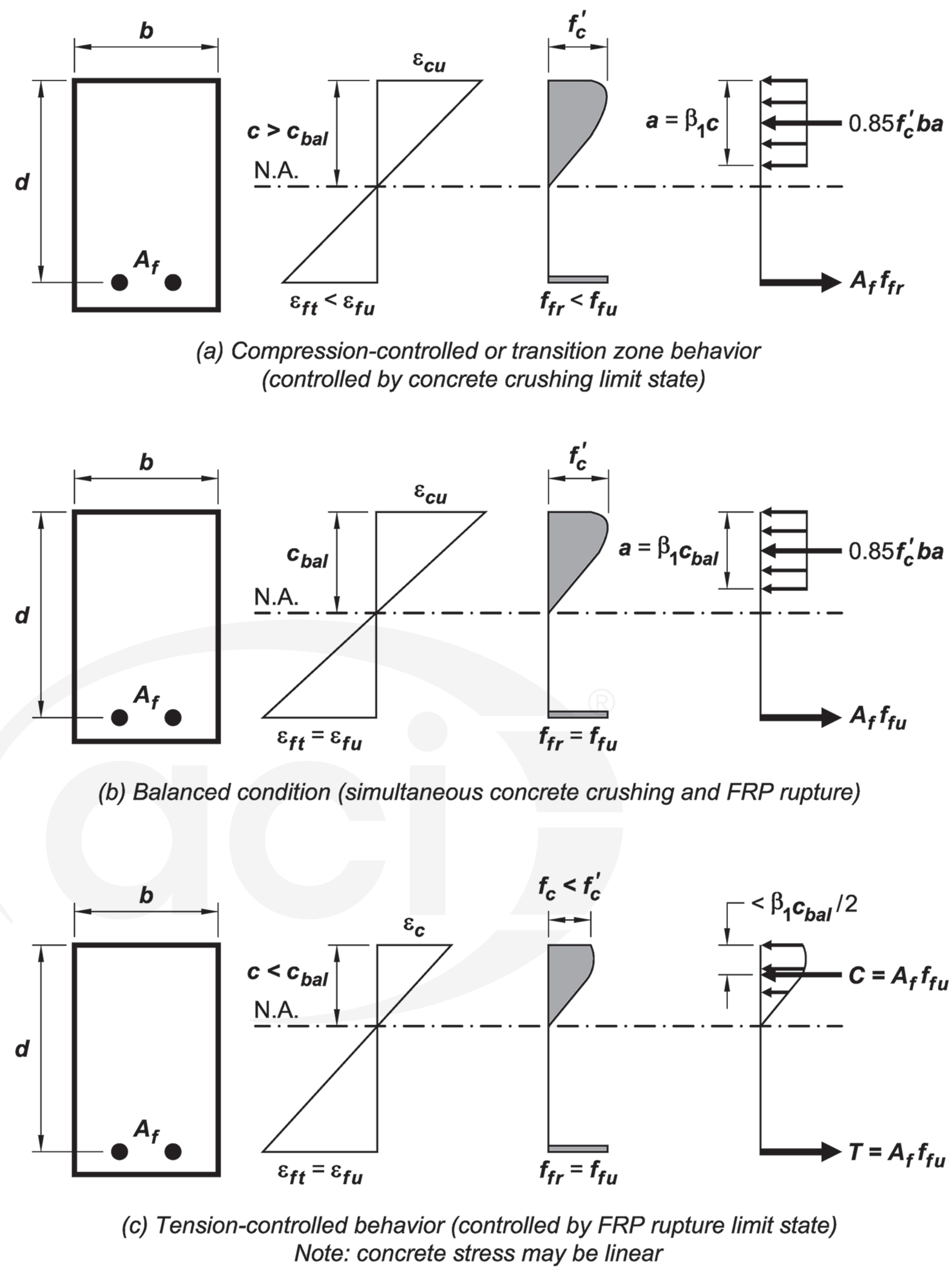

Fig. R22.3.1.1—Strain and stress distribution at ultimate conditions.

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

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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

$$P_o = 0.85 f_c' A_g \qquad (22.4.2.2)$$

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 $E_f A_f$. 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

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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 $\boldsymbol{b_w}$ shall be permitted to be taken as the diameter and $\boldsymbol{d}$ shall be permitted to be taken as 0.8 times the diameter.

$$V_u \le \phi 0.2 f_c' b_w d \tag{22.5.1.2}$$

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 $\boldsymbol{V_c}$ 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 $\boldsymbol{V_f}$ shall be calculated in accordance with 22.5.8.

22.5.1.7 Effect of any openings in members shall be considered in calculating $\boldsymbol{V_n}$.

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 $\boldsymbol{V_c}$.

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 $\boldsymbol{V_c}$.

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 100 psi.

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.

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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 10,000 psi, the Code imposes a maximum value of 100 psi 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 $8\sqrt{f_c'}\,b_w d$ 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 0.042 in. inclined crack width at ultimate; for the extreme

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case of dead to live load ratio of 5, the resulting maximum crack width at service load would be approximately 0.011 in.

22.5.4 *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 *Composite concrete members*

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 $\mathbf{V_c}$ *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 $\mathbf{V_c}$ *for nonprestressed members*

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	$5\lambda_s k_{cr}\sqrt{f'_c}b_w d$	(a)
		$0.8\lambda_s\sqrt{f'_c}\,b_w d$	(b)
Tensile axial load	$5\lambda_s k_{cr}\sqrt{f'_c}\,b_w d$		(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_fA_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 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 25.5.5.1.3—Size effect modification factor λ_s

Criteria	λ_s
$A_{fv} < A_{fv,min}$	$\sqrt{\dfrac{2}{1+(d/10)}} \leq 1.0$
$A_{fv} \geq Af_{v,min}$	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

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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_ws. 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

$$v_n = v_c \qquad (22.6.1.2)$$

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.

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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.

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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 100 psi.

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.

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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 10,000 psi, it is prudent to limit $\sqrt{f_c'}$ to 100 psi 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.

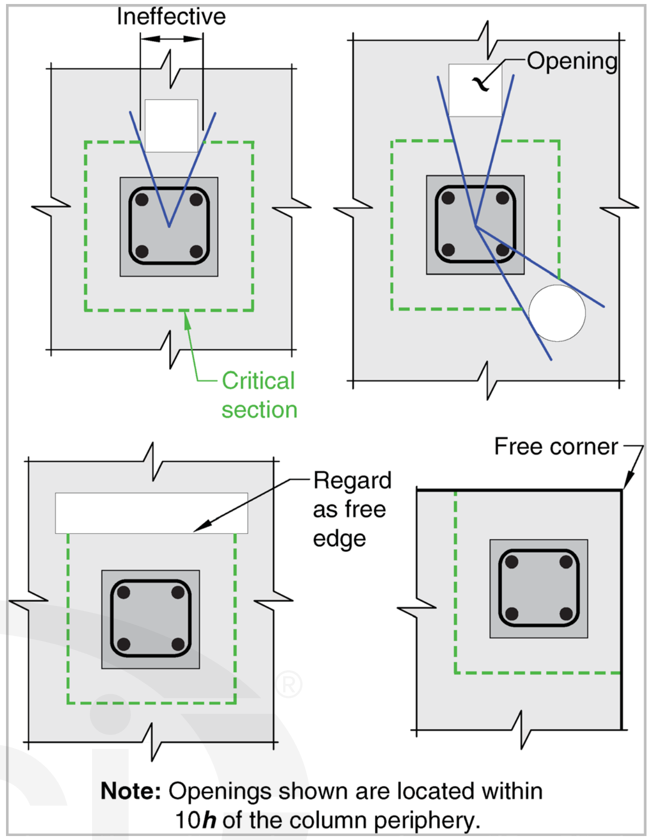

Fig. R22.6.4.3—Effect of openings and free edges (effective perimeter shown with dashed lines).

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).

$$v_c = 10\lambda_s k_{cr}\sqrt{f_c'} \qquad (22.6.5.2a)$$

but v_c need not be less than

$$v_c = 1.6\lambda_s\sqrt{f_c'} \qquad (22.6.5.2b)$$

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.

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 $d > 10$ **in.**, the size effect defined in Table 22.5.5.1.3 reduces the shear strength of two-way slabs below the traditional value of $4\sqrt{f_c'}b_od$ (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 $10k_{cr}\sqrt{f_c'}$ 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).

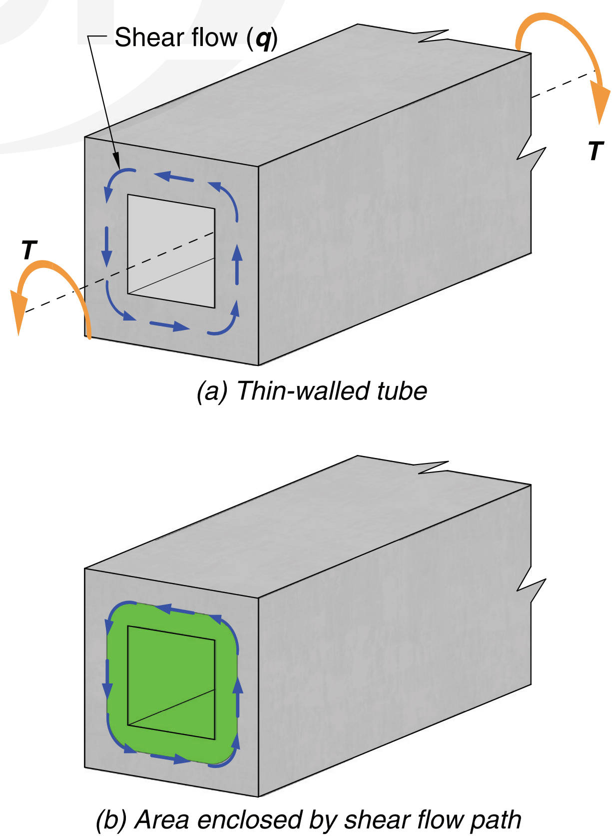

Fig. R22.7—(a) Thin-walled tube; and (b) area enclosed by shear flow path.

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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 100 psi.

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.

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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 10,000 psi, the Code imposes a maximum value of 100 psi 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 $4\sqrt{f_c'}$ 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.

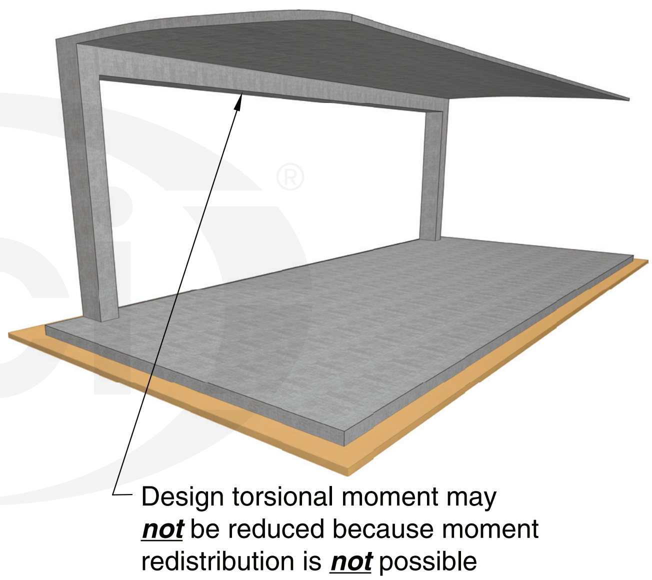

Fig. R22.7.3a—Equilibrium torsion, the design torsional moment may not be reduced (22.7.3.1).

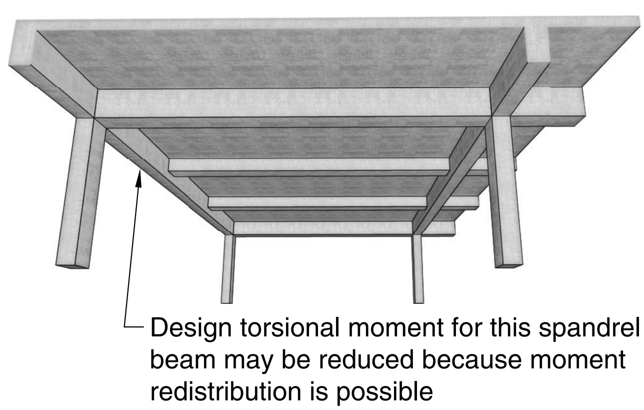

Fig. R22.7.3b—Compatibility torsion, the design torsional moment may be reduced (22.7.3.2).

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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	$\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$	(a)
Member subjected to axial force	$\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{4A_g\sqrt{f_c'}}}$	(b)

Table 22.7.4.1b—Threshold torsion for hollow cross sections

Type of member	T_{th}	
Member not subjected to axial force	$\sqrt{f_c'}\left(\frac{A_g^2}{p_{cp}}\right)$	(a)
Member subjected to axial force	$\sqrt{f_c'}\left(\frac{A_g^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{4A_g\sqrt{f_c'}}}$	(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 $4\sqrt{f_c'}$. The stress at cracking, $4\sqrt{f_c'}$, 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, $\tau = T/(2A_ot)$. Thus, cracking occurs when τ reaches $4\sqrt{f_c'}$, 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	$4\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$	(a)
Member subjected to axial force	$4\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{4A_g\sqrt{f_c'}}}$	(c)

22.7.6 *Torsional strength*

22.7.6.1 T_n shall be the lesser of (a) and (b):

(a) $$T_n = \frac{2A_oA_{ft}f_{ft}}{s} \qquad (22.7.6.1a)$$

(b) $$T_n = \frac{2A_oA_{fl}f_{fu}}{p_h} \qquad (22.7.6.1b)$$

where A_o shall be determined by analysis, A_{ft} is the area of one leg of a closed stirrup resisting torsion; A_{fl} is the area of longitudinal torsional reinforcement; and p_h is the perimeter of the centerline of the outermost closed stirrup.

COMMENTARY

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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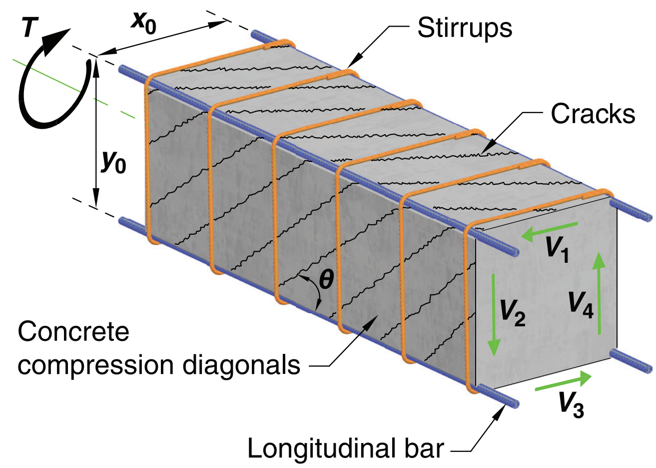

Fig. R22.7.6.1a—Space truss analogy.

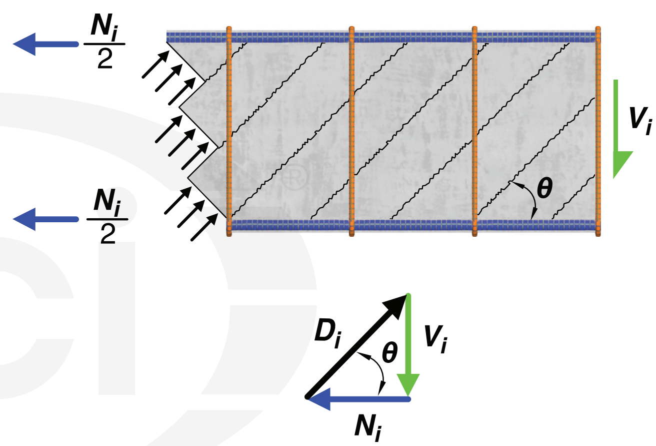

Fig. R22.7.6.1b—Resolution of shear force $\mathbf{V_i}$ *into diagonal compression force* $\mathbf{D_i}$ *and axial tension force* $\mathbf{N_i}$ *in one wall of tube.*

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 **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.

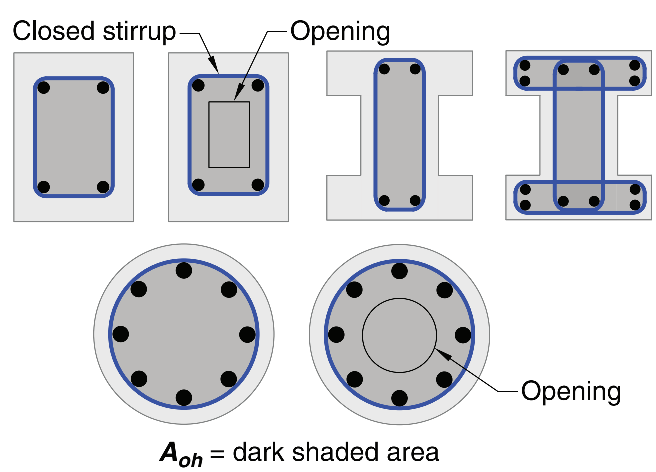

Fig. R22.7.6.1.1—Definition of $\mathbf{A_{oh}}$.

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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:

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u p_h}{1.7A_{oh}^2}\right)^2} \leq \phi(0.2f_c') \qquad (22.7.7.1)$$

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.

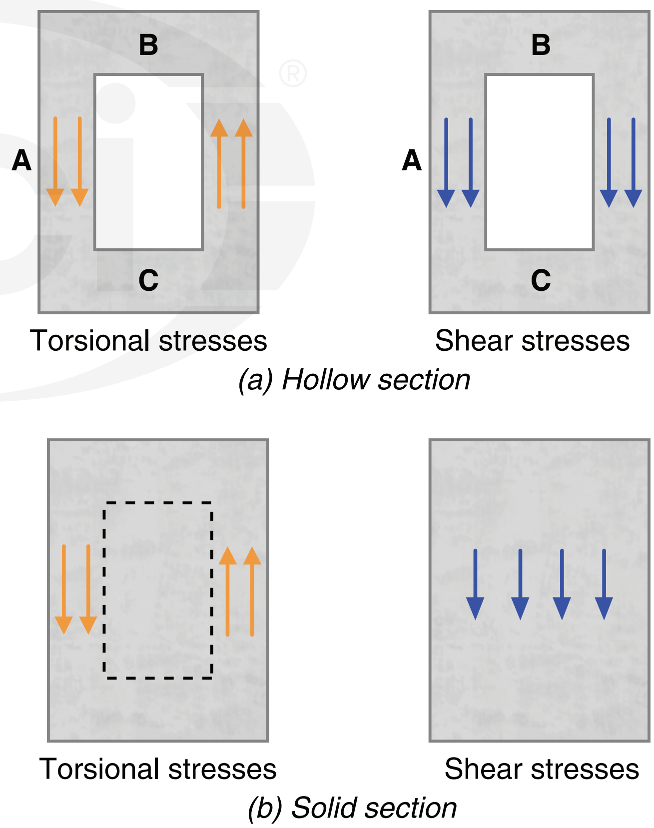

Fig. R22.7.7.1—Addition of torsional and shear stresses.

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, $\boldsymbol{B_n}$, shall be calculated in accordance with Table 22.8.3.2, where $\boldsymbol{A_1}$ is the loaded area and $\boldsymbol{A_2}$ 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)	$\sqrt{\frac{A_2}{A_1}}(0.85f_c'(A_1))$	(a)
		$2(0.85f_c'(A_1))$	(b)
Other cases	$0.85f_c'(A_1)$		(c)

COMMENTARY

R22.8.3 *Design strength*

R22.8.3.2 The permissible bearing stress of $\boldsymbol{0.85f_c'}$ 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.

$\boldsymbol{A_1}$ 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 $\boldsymbol{A_2}$ 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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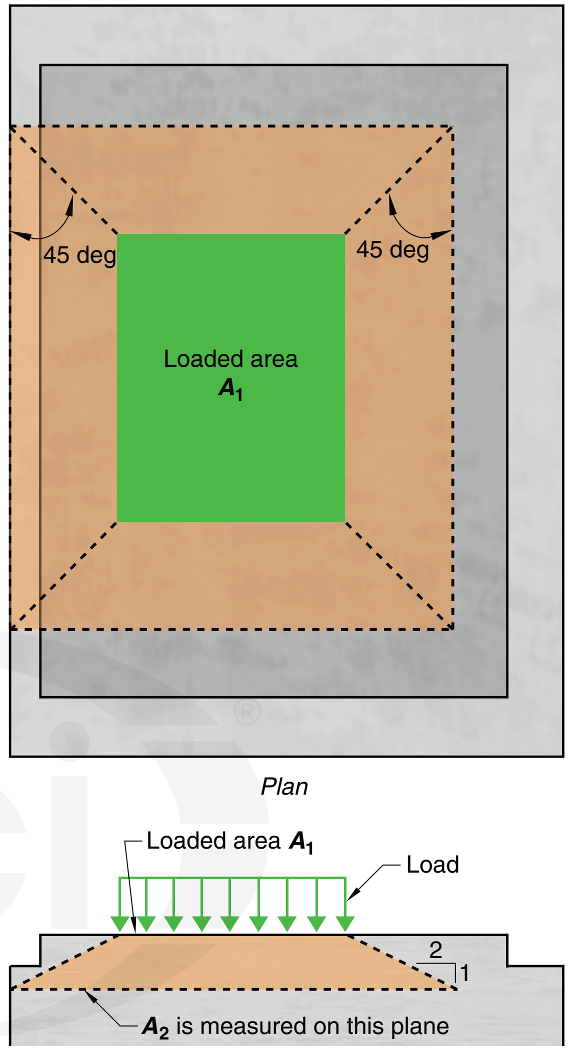

Fig. R22.8.3.2—Application of frustum to find $\mathbf{A_2}$ *in stepped or sloped supports.*

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).