CHAPTER 22—SECTIONAL STRENGTH

CODE

22.1—Scope

22.1.1 This chapter shall apply to calculating nominal strength at sections of members, including (a) through (g):

(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
(g) Shear friction

22.1.2 Sectional strength requirements of this chapter shall be satisfied unless the member or region of the member is designed in accordance with Chapter 23.

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 nonprestressed reinforcement shall be assumed proportional to the distance from neutral axis.

22.2.1.3 Strain in prestressed concrete and in bonded and unbonded prestressed reinforcement shall include the strain due to effective prestress.

22.2.1.4 Changes in strain for bonded prestressed reinforcement shall be assumed proportional to the distance from neutral axis.

COMMENTARY

R22.1—Scope

R22.1.1 The provisions in this chapter apply where the strength of the member is evaluated at critical sections.

R22.1.2 Chapter 23 provides methods for designing discontinuity regions where section-based methods do not apply.

R22.2—Design assumptions for moment and axial strength

R22.2.1 *Equilibrium and strain compatibility*

R22.2.1.1 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 It is reasonable to assume a linear distribution of strain over the depth of a reinforced concrete cross section for bending moment up to nominal strength, except in deep beams described in 9.9 and discontinuity regions addressed in Chapter 23. This assumption allows the determination of strain and corresponding stress in the reinforcement.

R22.2.1.4 The change in strain for bonded prestressed reinforcement is influenced by the change in strain at the section under consideration. For unbonded prestressed reinforcement, the change in strain is influenced by external load, reinforcement location, and boundary conditions along the length of the reinforcement. Current Code equations for calculating f_{ps} for unbonded tendons, as provided in 20.3.2.4, have been correlated with test results

CODE

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.

22.2.2.2 Tensile strength of concrete shall be neglected in flexural and axial strength calculations.

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.

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.

22.2.2.4.1 Concrete stress of $0.85f_c'$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a line parallel to the neutral axis located a distance a from the fiber of maximum compressive strain, as calculated by:

$$a = \beta_1 c \qquad (22.2.2.4.1)$$

22.2.2.4.2 Distance from the fiber of maximum compressive strain to the neutral axis, c, shall be measured perpendicular to the neutral axis.

22.2.2.4.3 Values of β_1 shall be in accordance with Table 22.2.2.4.3.

COMMENTARY

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 kinds to vary from 0.003 to higher than 0.008 under special conditions. However, the strain at which strength of the member is developed is usually 0.003 to 0.004 for members of normal proportions, materials, and strength.

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 approximately 10 to 15 percent 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.

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. 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.

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.

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 (Mattock et al. 1961).

R22.2.2.4.3 The values for β_1 were determined experimentally. The lower limit of β_1 is based on experimental data from beams constructed with concrete strengths greater than 8000 psi (Leslie et al. 1976; Karr et al. 1978).

CODE

Table 22.2.2.4.3—Values of β_1 for equivalent rectangular concrete stress distribution

f_c', psi	β_1	
$2500 \leq f_c' \leq 4000$	0.85	(a)
$4000 < f_c' < 8000$	$0.85 - \frac{0.05(f_c' - 4000)}{1000}$	(b)
$f_c' \geq 8000$	0.65	(c)

22.2.3 *Design assumptions for nonprestressed reinforcement*

22.2.3.1 Deformed reinforcement used to resist tensile or compressive forces shall conform to 20.2.1.

22.2.3.2 Stress-strain relationship and modulus of elasticity for deformed reinforcement shall be idealized in accordance with 20.2.2.1 and 20.2.2.2.

22.2.4 *Design assumptions for prestressed reinforcement*

22.2.4.1 For members with bonded prestressed reinforcement conforming to 20.3.1, stress at nominal flexural strength, f_{ps}, shall be calculated in accordance with 20.3.2.3.

22.2.4.2 For members with unbonded prestressed reinforcement conforming to 20.3.1, f_{ps} shall be calculated in accordance with 20.3.2.4.

22.2.4.3 If the embedded length of the prestressed strand is less than ℓ_d, the design stress of the prestressed strand shall not exceed the value given in 25.4.8.3, as modified by 25.4.8.1(b).

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.

22.3.2 *Prestressed concrete members*

22.3.2.1 Deformed reinforcement conforming to 20.2.1, provided in conjunction with prestressed reinforcement, shall be permitted to be considered to contribute to the tensile force and be included in flexural strength calculations at a stress equal to f_y.

22.3.2.2 Other nonprestressed reinforcement shall be permitted to be considered to contribute to the flexural strength if a strain compatibility analysis is performed to calculate stresses in such reinforcement.

COMMENTARY

R22.3—Flexural strength

R22.3.2 *Prestressed concrete members*

R22.3.2.2 Bond length for nontensioned prestressing strand (Salmons and McCrate 1977; PCA 1980) should be sufficient to develop the stress consistent with strain compatibility analysis at the critical section.

CODE

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.

22.3.3.2 For calculation of M_n for composite concrete slabs and beams, use of the entire composite section shall be permitted.

22.3.3.3 For calculation of M_n for composite concrete slabs and beams, no distinction shall be made between shored and unshored members.

22.3.3.4 For calculation of M_n for composite concrete members where the specified concrete compressive strength of different elements varies, 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 most critical value of M_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.

22.4.2 *Maximum axial compressive 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) for nonprestressed members and by Eq. (22.4.2.3) for prestressed members. The value of f_y shall be limited to a maximum of 80,000 psi.

Table 22.4.2.1—Maximum axial strength

Member	Transverse reinforcement	$P_{n,max}$	
Nonprestressed	Ties conforming to 22.4.2.4	$0.80P_o$	(a)
	Spirals conforming to 22.4.2.5	$0.85P_o$	(b)
Prestressed	Ties	$0.80P_o$	(c)
	Spirals	$0.85P_o$	(d)
Deep foundation member	Ties conforming to Ch. 13	$0.80P_o$	(e)

22.4.2.2 For nonprestressed members, P_o shall be calculated by:

COMMENTARY

R22.3.3 *Composite concrete members*

R22.3.3.1 The scope of Chapter 22 is intended to include composite concrete flexural members. Where separate placements of concrete are designed to act as a unit, the interface is designed for the forces that will be transferred across the interface. Composite structural steel-concrete beams are not covered in the Code. Design provisions for these types of composite members are covered in ANSI/AISC 360.

R22.4—Axial strength or combined flexural and axial strength

R22.4.2 *Maximum axial compressive 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 0.05 for tied and spirally reinforced members conforming to 22.4.2.4 and 22.4.2.5, respectively. The same axial load limitation applies to both cast-in-place and precast compression members. The value of f_y is limited to 80,000 psi because the compression capacity of the concrete is likely to be reached before this stress is exceeded. The transverse reinforcement requirements for columns do not apply to deep foundation members. Chapter 13 provides the detailing requirements for these members.

CODE

$$P_o = 0.85f_c'(A_g - A_{st}) + f_yA_{st} \qquad (22.4.2.2)$$

where A_{st} is the total area of nonprestressed longitudinal reinforcement.

22.4.2.3 For prestressed members, P_o shall be calculated by:

$$P_o = 0.85f_c'(A_g - A_{st} - A_{pd}) + f_yA_{st} - (f_{se} - 0.003E_p)A_{pt} \qquad (22.4.2.3)$$

where A_{pt} is the total area of prestressing reinforcement, and A_{pd} is the total area occupied by duct, sheathing, and prestressing reinforcement; the value of f_{se} shall be at least $0.003E_p$. For grouted, post-tensioned tendons, it shall be permitted to assume A_{pd} equals A_{pt}.

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 of a nonprestressed or prestressed member, P_{nt}, shall not be taken greater than $P_{nt,max}$, calculated by:

$$P_{nt,max} = f_yA_{st} + (f_{se} + \Delta f_p)A_{pt} \qquad (22.4.3.1)$$

where $(f_{se} + \Delta f_p)$ shall not exceed f_{py}, and A_{pt} is zero for nonprestressed members.

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:

$$V_n = V_c + V_s \qquad (22.5.1.1)$$

COMMENTARY

R22.4.2.3 The effects of prestressing on the axial strength of compression members are taken into account in Eq. (22.4.2.3). Equation (22.4.2.3) is similar to Eq. (22.4.2.2) for nonprestressed compression members. The effective area of concrete subjected to the limiting stress of $0.85f_c'$ is reduced by the term A_{pd} to account for the area of ducts, sheathing, and prestressing reinforcement. A third term is added to account for the reduction of column capacity due to the prestress force. At nominal strength, the stress in the prestressed reinforcement, f_{se}, is decreased by $0.003E_p$, where 0.003 is the assumed compressive strain at the axial capacity of the member.

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.

The one-way shear equations for nonprestressed concrete were changed in the 2019 Code with the primary objectives of including effect of member depth, commonly referred to as the “size effect,” and the effects of the longitudinal reinforcement ratio on shear strength.

The shear strength is based on an average shear stress over the effective cross section, b_wd.

Chapter 23 allows the use of the strut-and-tie method in the shear design of any structural concrete member, or discontinuity region in a member.

CODE

22.5.1.2 Cross-sectional dimensions shall be selected to satisfy Eq. (22.5.1.2).

$$V_u \leq \phi\left(V_c + 8\sqrt{f_c'}\, b_w d\right) \qquad (22.5.1.2)$$

22.5.1.3 For nonprestressed members, V_c shall be calculated in accordance with 22.5.5.

22.5.1.4 For prestressed members, V_c shall be calculated in accordance with 22.5.6 or 22.5.7.

22.5.1.5 For calculation of V_c, V_{ci}, and V_{cw}, λ shall be in accordance with 19.2.4.

22.5.1.6 V_s 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.

22.5.1.8 Effect of axial tension due to creep and shrinkage in members shall be considered in calculating V_c.

22.5.1.9 Effect of inclined flexural compression in variable depth members shall be permitted to be considered in calculating V_c.

22.5.1.10 The interaction of shear forces acting along orthogonal axes shall be permitted to be neglected if (a) or (b) is satisfied.

(a) $\dfrac{V_{u,x}}{\phi V_{n,x}} \leq 0.5$ (22.5.1.10a)

(b) $\dfrac{V_{u,y}}{\phi V_{n,y}} \leq 0.5$ (22.5.1.10b)

COMMENTARY

R22.5.1.2 The limit on cross-sectional dimensions in 22.5.1.2 is intended to control cracking under service load and to minimize the likelihood of diagonal compression failure.

R22.5.1.7 Openings in the web of a member can reduce its shear strength. The effects of openings are discussed in Section 4.7 of Joint ACI-ASCE Committee 426 (1973), Barney et al. (1977), and Schlaich et al. (1987). The strut-and-tie method as addressed in Chapter 23 can be used to design members with openings.

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.

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.

R22.5.1.10 and **R.22.5.1.11** Reinforced concrete members, such as columns and beams, may be subjected to biaxial shear. For symmetrically reinforced circular sections, nominal one-way shear strength about any axis is the same. Therefore, when a circular section is subjected to shear along two centroidal axes, shear strength can be evaluated using the resultant shear. However, for rectangular and other cross sections, calculating nominal one-way shear strength along the axis of the resultant shear is not practical. Tests and analytical results for columns have indicated that for biaxial shear loading, the shear strength follows an elliptical interaction diagram that requires calculating nominal one-way shear strength along two orthogonal directions (Umehara and Jirsa 1984). Considering shear along each centroidal axis independently can be unconservative. Thus, linear interaction accounts for biaxial shear.

CODE

22.5.1.11 If $\frac{V_{u,x}}{\phi V_{n,x}} > 0.5$ and $\frac{V_{n,y}}{\phi V_{n,y}} > 0.5$ then Eq. (22.5.1.11) shall be satisfied.

$$\frac{V_{u,x}}{\phi V_{n,x}} + \frac{V_{u,y}}{\phi V_{n,y}} \leq 1.5 \qquad (22.5.1.11)$$

22.5.2 *Geometric assumptions*

22.5.2.1 For calculation of V_c and V_s, it shall be permitted to assume (a) through (d) :

(a) d equal to $0.8h$ for rectangular columns
(b) d equal to 0.8 times the diameter for circular sections
(c) b_w equal to the diameter for solid circular sections
(d) b_w equal to twice the wall thickness for hollow circular sections

22.5.2.2 For calculation of V_c and V_s in prestressed members, d shall be taken as the distance from the extreme compression fiber to the centroid of prestressed and any nonprestressed longitudinal reinforcement but need not be taken less than $0.8h$.

22.5.3 *Limiting material strengths*

22.5.3.1 The value of $\sqrt{f_c'}$ used to calculate V_c, V_{ci}, and V_{cw} for one-way shear shall not exceed 100 psi, unless allowed in 22.5.3.2.

22.5.3.2 Values of $\sqrt{f_c'}$ greater than 100 psi shall be permitted in calculating V_c, V_{ci}, and V_{cw} for reinforced or prestressed concrete beams and concrete joist construction having minimum web reinforcement in accordance with 9.6.3.4 or 9.6.4.2.

22.5.3.3 The values of f_y and f_{yt} used to calculate V_s shall not exceed the limits in 20.2.2.4.

COMMENTARY

R22.5.2 *Geometric assumptions*

R22.5.2.1 The computation of d for rectangular columns and circular sections can be complicated considering variations in axial load as well as multiple layers of reinforcement. While the actual d can be used, simplified definitions of d are provided. Experimental results indicate that d computed as 80% of the overall column dimension is appropriate and results in good accuracy (Sezen et al. 2021).

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 defined in 22.5.2.1 (Faradji and Diaz de Cossio 1965; Khalifa and Collins 1981).

R22.5.2.2 Although the value of d may vary along the span of a prestressed beam, studies (MacGregor and Hanson 1969) have shown that, for prestressed concrete members, d need not be taken less than $0.8h$. The beams considered had some straight prestressed reinforcement or reinforcing bars at the bottom of the section and had stirrups that enclosed the longitudinal reinforcement.

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. Exceptions to this limit are permitted in beams and joists if the transverse reinforcement satisfies the requirements in 22.5.3.2.

R22.5.3.2 Based on the beam test results in Mphonde and Frantz (1984), Elzanaty et al. (1986), Roller and Russell (1990), Johnson and Ramirez (1989), and Ozcebe et al. (1999), an increase in the minimum amount of transverse reinforcement is required for high-strength concrete. These tests indicate a reduction in reserve shear strength occurs as f_c' increases in beams reinforced with transverse reinforcement providing an effective shear stress of 50 psi. By providing minimum transverse reinforcement, which increases as f_c' increases, the reduction in shear strength is offset.

R22.5.3.3 The upper limit of 60,000 psi on the value of f_y and f_{yt} used in design is intended to control diagonal crack widths.

CODE

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.

22.5.4.2 For calculation of V_n for composite concrete members, no distinction shall be made between shored and unshored members.

22.5.4.3 For calculation of V_n for composite concrete 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 concrete 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 concrete member is assumed to resist vertical shear, it shall be permitted to calculate V_s 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*

22.5.5.1 For nonprestressed members, 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.

Table 22.5.5.1—V_c for nonprestressed members

Criteria		V_c	
$A_v \geq A_{v,min}$	Either of:	$\left[2\lambda\sqrt{f'_c} + \dfrac{N_u}{6A_g}\right]b_w d$	(a)
		$\left[8\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \dfrac{N_u}{6A_g}\right]b_w d$	(b)
$A_v < A_{v,min}$		$\left[8\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \dfrac{N_u}{6A_g}\right]b_w d$	(c)

Notes:
1. Axial load N_u is positive for compression and negative for tension.
2. V_c shall not be taken less than zero.

COMMENTARY

R22.5.4 *Composite concrete members*

R22.5.4.1 The scope of Chapter 22 includes composite concrete members. Composite structural steel-concrete beams are not covered in the Code. Design provisions for such composite members are covered in ANSI/AISC 360.

R22.5.5 V_c *for nonprestressed members*

R22.5.5.1 Test results for nonprestressed members without shear reinforcement indicate that measured shear strength, attributed to concrete, does not increase in direct proportion with member depth. This phenomenon is often referred to as "size effect." For example, if member depth doubles, shear at failure for the deeper beam may be less than twice the shear at failure for the shallower beam (Sneed and Ramirez 2010). Research (Angelakos et al. 2001; Lubell et al. 2004; Brown et al. 2006; Becker and Buettner 1985; Anderson 1978; Bažant et al. 2007) has shown that shear stress at failure is lower for beams and slabs with increased depth and a reduced area of longitudinal reinforcement. Changes were made in the ACI 318-19 code (Kuchma et al. 2019) to account for size effect and the effect of longitudinal reinforcement ratio on shear strength of members.

In Table 22.5.5.1, for $A_v > A_{v,min}$, either equation for V_c may be used. Equation (a) is provided as a simpler option.

When calculating V_c by Table 22.5.5.1, an axial tension force can cause V_c to have a negative value. In those cases, the Code specifies that V_c should be taken equal to zero.

The criteria column in Table 22.5.5.1 references $A_{v,min}$, which is defined in 7.6.3.3 for one-way slabs, 9.6.3.4 for beams, and 10.6.2.2 for columns and referenced throughout the Code.

CODE

22.5.5.1.1 V_c shall not be taken greater than $5\lambda\sqrt{f_c'}b_wd$. V_c need not be taken less than $\lambda\sqrt{f_c'}b_wd$ except in cases (a) or (b):

(a) elements subjected to net axial tension
(b) if 18.6.5.2 or 18.7.6.2.1 apply.

22.5.5.1.2 In Table 22.5.5.1, the value of $N_u/6A_g$ shall not be taken greater than $0.05f_c'$.

22.5.5.1.3 The size effect modification factor, λ_s, shall be determined by

$$\lambda_s = \frac{2}{1+\frac{d}{10}} \leq 1 \qquad (22.5.5.1.3)$$

22.5.5.1.4 For nonprestressed beams and one-way slabs constructed with steel fiber-reinforced concrete, conforming to 26.4.1.6.1(a), 26.4.2.2(h), and 26.12.8.1(a), V_c shall be the greater of Eq. (a) and 1.3 times Equation (b) of Table 22.5.5.1.

22.5.6 V$_c$ *for prestressed members*

22.5.6.1 This section shall apply to the calculation of V_c for post-tensioned and pretensioned members in regions where the effective force in the prestressed reinforcement is fully transferred to the concrete. For regions of pretensioned members where the effective force in the prestressed reinforcement is not fully transferred to the concrete, 22.5.7 shall govern the calculation of V_c.

22.5.6.2 For prestressed members, V_c shall be permitted to be the lesser of V_{ci} calculated in accordance with 22.5.6.2.1 and V_{cw} calculated in accordance with 22.5.6.2.2 or 22.5.6.2.3.

COMMENTARY

R22.5.5.1.1 Because of the compounded effect of λ_s and ρ_w in Eq. 22.5.5.1c, V_c may tend to zero for large, lightly reinforced concrete members. The lower bound $\lambda\sqrt{f_c'}b_wd$ is intended to result in a concrete shear strength consistent with earlier, successful practice.

R22.5.5.1.3 The parameters within the size effect modification factor, λ_s, are consistent with fracture mechanics theory for reinforced concrete (Bažant et al. 2007; Frosch et al. 2017).

R22.5.5.1.4 The use of Eq. (a) and (b) in Table 22.5.5.1 for steel fiber-reinforced concrete and associated dosages in Chapter 26 are supported by experimental results (Dinh et al. 2010; Shoaib et al. 2014; Zarrinpour and Chao 2017). The presence of steel fibers is not considered in determining the value of $A_{v,min}$. Requirements for steel fibers are included in Chapter 26.

R22.5.6 V$_c$ *for prestressed members*

R22.5.6.1 Editions of the Code prior to 2025 included an approximate method for the calculation of V_c for prestressed members. This method was removed because the method in Section 22.5.6.2 is a better predictor of shear strength.

R22.5.6.2 Two types of inclined cracking occur in concrete beams: web-shear cracking and flexure-shear cracking. These two types of inclined cracking are illustrated in Fig. R22.5.6.2.

Web-shear cracking begins from an interior point in a member when the principal tensile stresses exceed the tensile strength of the concrete. Flexure-shear cracking is initiated by flexural cracking. When flexural cracking occurs, the shear stresses in the concrete above the crack are increased. The flexure-shear crack develops when the combined shear and flexural-tensile stress exceeds the tensile strength of the concrete.

The nominal shear strength provided by the concrete, V_c, is assumed equal to the lesser of V_{ci} and V_{cw}. The derivations of Eq. (22.5.6.2.1a) and Eq. (22.5.6.2.2) are summarized in SP-10 (1965).

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22.5.6.2.1 The flexure-shear strength V_{ci} shall be calculated by (a) but need not be taken less than (b) or (c):

(a) $$V_{ci} = 0.6\lambda\sqrt{f_c'}\,b_w d_p + V_d + \frac{V_i M_{cre}}{M_{max}} \quad (22.5.6.2.1a)$$

(b) For members with $A_{ps}f_{se} < 0.4(A_{ps}f_{pu} + A_s f_y)$,

$$V_{ci} = 1.7\lambda\sqrt{f_c'}\,b_w d \quad (22.5.6.2.1b)$$

(c) For members with $A_{ps}f_{se} \geq 0.4(A_{ps}f_{pu} + A_s f_y)$,

$$V_{ci} = 2\lambda\sqrt{f_c'}\,b_w d \quad (22.5.6.2.1c)$$

where d_p need not be taken less than $0.80h$, the values of M_{max} and V_i shall be calculated from the load combinations causing maximum factored moment to occur at section considered, and M_{cre} shall be calculated by:

$$M_{cre} = \left(\frac{I}{y_t}\right)\left(6\lambda\sqrt{f_c'} + f_{pe} - f_d\right) \quad (22.5.6.2.1d)$$

COMMENTARY

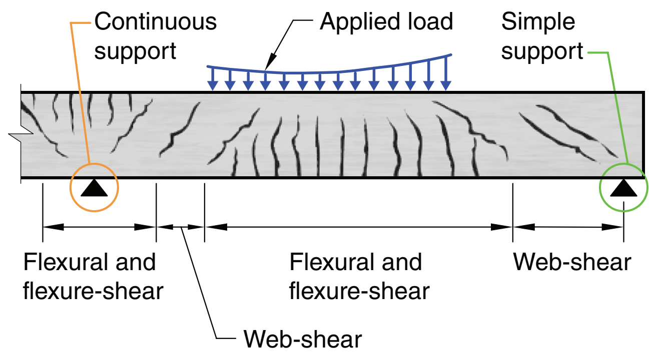

Fig. R22.5.6.3—Types of cracking in concrete beams.

R22.5.6.2.1 In deriving Eq. (22.5.6.2.1a), it was assumed that V_{ci} is the sum of the shear required to cause a flexural crack at the section in question given by:

$$V = \frac{V_i M_{cre}}{M_{max}} \quad (R22.5.6.2.1a)$$

plus an additional increment of shear required to change the flexural crack to a flexure-shear crack. The externally applied factored loads, from which V_i and M_{max} are determined, include superimposed dead load and live load. In calculating M_{cre} for substitution into Eq. (22.5.6.2.1a), I and y_t are the properties of the section resisting the externally applied loads.

For a composite concrete member, where part of the dead load is resisted by only a part of the section, appropriate section properties should be used to calculate f_d. The shear due to dead loads, V_d, and that due to other loads, V_i, are separated in this case. V_d is then the total shear force due to unfactored dead load acting on that part of the section resisting the dead loads acting prior to composite action plus the unfactored superimposed dead load acting on the composite member. The terms V_i and M_{max} may be taken as

$$V_i = V_u - V_d \quad (R22.5.6.2.1b)$$

$$M_{max} = M_u - M_d \quad (R22.5.6.2.1c)$$

where V_u and M_u are the factored shear and moment due to the total factored loads, and M_d is the moment due to unfactored dead load (the moment corresponding to f_d).

For noncomposite, uniformly loaded beams, the total cross section resists all the shear, and the live and dead load shear force diagrams are similar. In this case, Eq. (22.5.6.2.1a) and Eq. (22.5.6.2.1d) reduce to

$$V_{ci} = 0.6\lambda\sqrt{f_c'}\,b_w d + \frac{V_u M_{ct}}{M_u} \quad (R22.5.6.2.1d)$$

where

$$M_{ct} = (I/y_t)(6\lambda\sqrt{f_c'} + f_{pe}) \quad (R22.5.6.2.1e)$$

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The cracking moment M_{ct} in the two preceding equations represents the total moment, including dead load, required to cause cracking at the extreme fiber in tension. This is not the same as M_{cre} in Eq. (22.5.6.2.1a) where the cracking moment is that due to all loads except the dead load. In Eq. (22.5.6.2.1a), the dead load shear is added as a separate term.

M_u is the factored moment on the beam at the section under consideration, and V_u is the factored shear force occurring simultaneously with M_u. Because the same section properties apply to both dead and live load stresses, there is no need to calculate dead load stresses and shears separately. M_{ct} reflects the total stress change from effective prestress to a tension of $6\lambda\sqrt{f_c'}$, assumed to cause flexural cracking.

22.5.6.2.2 The web-shear strength V_{cw} shall be calculated by:

$$V_{cw} = \left(3.5\lambda\sqrt{f_c'} + 0.3f_{pc}\right)b_w d_p + V_p \qquad (22.5.6.2.2)$$

where d_p need not be taken less than $0.80h$, and V_p is the vertical component of the effective prestress.

R22.5.6.2.2 Equation (22.5.6.2.2) is based on the assumption that web-shear cracking occurs at a shear level causing a principal tensile stress of approximately $4\lambda\sqrt{f_c'}$ at the centroidal axis of the cross section. V_p is calculated from the effective prestress force without load factors.

22.5.6.2.3 As an alternative to 22.5.6.2.2, it shall be permitted to calculate V_{cw} as the shear force corresponding to dead load plus live load that results in a principal tensile stress of $4\lambda\sqrt{f_c'}$ at location (a) or (b):

(a) Where the centroidal axis of the prestressed cross section is in the web, the principal tensile stress shall be calculated at the centroidal axis.
(b) Where the centroidal axis of the prestressed cross section is in the flange, the principal tensile stress shall be calculated at the intersection of the flange and the web.

22.5.6.2.4 In composite concrete members, the principal tensile stress shall be calculated at the location specified in 22.5.6.2.3 for the composite section, considering superposition of stresses calculated cross sections that resist the corresponding loads.

R22.5.6.2.4 Generally, in unshored construction the principal tensile stresses due to dead load are caused before composite action and principal tensile stresses due to live load are caused after composite action is developed in a member. In shored construction the principal tensile stresses due to both the dead load and live load are caused after composite action is developed.

22.5.7 V_c *for pretensioned members in regions of reduced prestress force*

22.5.7.1 When calculating V_c, the transfer length of prestressed reinforcement, ℓ_{tr}, shall be assumed to be $50d_b$ for strand and $100d_b$ for wire.

R22.5.7 V_c *for pretensioned members in regions of reduced prestress force*

R22.5.7.1 The effect of the reduced prestress near the ends of pretensioned beams on the shear strength should be taken into account. Provision 22.5.7.2 relates to the reduced shear strength at sections within the transfer length of prestressed reinforcement when bonding of prestressed reinforcement extends to the end of the member. Provision 22.5.7.3 relates to the reduced shear strength at sections within the length over which some of the prestressed reinforcement is not bonded to the concrete, or within the transfer length of

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22.5.7.2 If bonding of strands extends to the end of the member, (a) and (b) shall apply:

(a) The effective prestress force shall be assumed to vary linearly from zero at the end of the prestressed reinforcement to a maximum at a distance ℓ_{tr} from the end of the prestressed reinforcement.
(b) The reduced effective prestress force shall be used to calculate V_{cw} in 22.5.6.2.

22.5.7.3 If bonding of strands does not extend to the end of the member, (a) and (b) shall apply:

(a) The effective prestress force shall be assumed to vary linearly from zero at the point where bonding commences to a maximum at a distance ℓ_{tr} from that point.
(b) The reduced effective prestress force shall be used to calculate V_c in accordance with 22.5.6.2.

22.5.8 *One-way 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.

$$V_s \geq \frac{V_u}{\phi} - V_c \qquad (22.5.8.1)$$

22.5.8.2 For one-way members reinforced with transverse reinforcement, V_s shall be calculated in accordance with 22.5.8.5.

22.5.8.3 For one-way members reinforced with bent-up longitudinal bars, V_s shall be calculated in accordance with 22.5.8.6.

22.5.8.4 If more than one type of shear reinforcement is provided to reinforce the same portion of a member, V_s shall be the sum of the V_s values for the various types of shear reinforcement.

22.5.8.5 *One-way shear strength provided by transverse reinforcement*

22.5.8.5.1 In nonprestressed and prestressed members, shear reinforcement satisfying (a), (b), or (c) shall be permitted:

(a) Stirrups, ties, or hoops perpendicular to longitudinal axis of member
(b) Welded wire reinforcement with wires located perpendicular to longitudinal axis of member

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the prestressed reinforcement for which bonding does not extend to the end of the beam.

R22.5.8 *One-way shear reinforcement*

R22.5.8.2 Provisions of 22.5.8.5 apply to all types of transverse reinforcement, including stirrups, ties, hoops, crossties, and spirals.

R22.5.8.5 *One-way shear strength provided by transverse reinforcement*

R22.5.8.5.1 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. 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. The concrete is assumed to contribute to the shear capacity through resistance across the concrete compressive

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(c) Spiral reinforcement

22.5.8.5.2 Inclined stirrups making an angle of at least 45 degrees with the longitudinal axis of the member and crossing the plane of the potential shear crack shall be permitted to be used as shear reinforcement in nonprestressed members.

22.5.8.5.3 V_s for shear reinforcement in 22.5.8.5.1 shall be calculated by:

$$V_s = \frac{A_v f_{yt} d}{s} \qquad (22.5.8.5.3)$$

where s is the spiral pitch or the longitudinal spacing of the shear reinforcement, and A_v is given in 22.5.8.5.5 or 22.5.8.5.6.

22.5.8.5.4 V_s for shear reinforcement in 22.5.8.5.2 shall be calculated by:

$$V_s = \frac{A_v f_{yt}(\sin\alpha + \cos\alpha)d}{s} \qquad (22.5.8.5.4)$$

where α is the angle between the inclined stirrups and the longitudinal axis of the member, s is measured parallel to the longitudinal reinforcement, and A_v is given in 22.5.8.5.5.

22.5.8.5.5 For each rectangular tie, stirrup, hoop, or crosstie, A_v shall be the effective area of all bar legs or wires within spacing s.

22.5.8.5.6 For each circular tie or spiral, A_v shall be two times the area of the bar or wire within spacing s.

22.5.8.6 *One-way shear strength provided by bent-up longitudinal bars*

22.5.8.6.1 The center three-fourths of the inclined portion of bent-up longitudinal bars shall be permitted to be used as shear reinforcement in nonprestressed members if the angle α between the bent-up bars and the longitudinal axis of the member is at least 30 degrees.

22.5.8.6.2 If shear reinforcement consists of a single bar or a single group of parallel bars having an area A_v, all bent

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zone, aggregate interlock, and dowel action in an amount equivalent to that which caused inclined cracking.

Equations (22.5.8.5.3), (22.5.8.5.4), and (22.5.8.6.2a) are presented in terms of nominal shear strength provided by shear reinforcement, V_s. Where shear reinforcement perpendicular to the axis of the member is used, the required area of shear reinforcement, A_v, and its spacing, s, are calculated by

$$\frac{A_v}{s} = \frac{V_u - \phi V_c}{\phi f_{yt} d} \qquad (R22.5.8.5)$$

R22.5.8.5.2 Although inclined stirrups crossing the plane of the potential shear cracks are permitted, their use is not appropriate where the direction of net shear reverses due to changes in transient load.

R22.5.8.5.4 To be effective, it is critical that inclined stirrups cross potential shear cracks. If the inclined stirrups are generally oriented parallel to the potential shear cracks, the stirrups provide no shear strength.

R22.5.8.5.6 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 defined in 22.5.2.2 (Faradji and Diaz de Cossio 1965; Khalifa and Collins 1981).

R22.5.8.6 *One-way shear strength provided by bent-up longitudinal bars*

R22.5.8.6.1 To be effective, it is critical that the inclined portion of the bent-up longitudinal bar cross potential shear cracks. If the inclined bars are generally oriented parallel to the potential shear cracks, the bars provide no shear strength.

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the same distance from the support, V_s shall be the lesser of (a) and (b):

(a) $V_s = A_v f_y \sin\alpha$ (22.5.8.6.2a)

(b) $V_s = 3\sqrt{f_c'} b_w d$ (22.5.8.6.2b)

where α is the angle between bent-up reinforcement and longitudinal axis of the member.

22.5.8.6.3 If shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, V_s shall be calculated by Eq. (22.5.8.5.4).

22.6—Two-way shear strength

22.6.1 *General*

22.6.1.1 Provisions 22.6.1 through 22.6.8 apply to the nominal two-way shear strength of members with and without shear reinforcement.

22.6.1.2 Nominal two-way shear strength for members without shear reinforcement shall be calculated by

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

22.6.1.3 Nominal two-way shear strength for members with shear reinforcement shall be calculated by

$$v_n = v_c + v_s \quad (22.6.1.3)$$

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. For two-way shear in members with shear reinforcement, v_c shall not exceed the limits in 22.6.6.1.

22.6.1.6 For calculation of v_c, λ shall be in accordance with 19.2.4.

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R22.6—Two-way shear strength

Factored two-way shear stress 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, either without shear reinforcement or with shear reinforcement in the form of stirrups or headed shear studs. Factored shear demand and strength are calculated in terms of stress, permitting superposition of effects from direct shear and moment transfer.

Design provisions for shearheads have been eliminated from the Code because this type of shear reinforcement is seldom used in current practice. Shearheads may be designed following the provisions of ACI CODE-318-14.

R22.6.1 *General*

R22.6.1.4 The critical section perimeter b_o is defined in 22.6.4.

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22.6.1.7 For two-way shear in members reinforced with single- or multiple-leg stirrups, v_s shall be calculated in accordance with 22.6.7.

22.6.1.8 For two-way shear in members reinforced with headed shear stud reinforcement, v_s shall be calculated in accordance with 22.6.8.

22.6.2 *Effective depth*

22.6.2.1 For calculation of v_c and v_s for two-way shear, d shall be the average of the effective depths in the two orthogonal directions.

22.6.2.2 For two-way shear in prestressed members, d need not be taken less than $0.8h$.

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 The value of f_{yt} used to calculate v_s shall not exceed the limits in 20.2.2.4.

22.6.4 *Critical sections for two-way shear*

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

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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 shear in 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.

R22.6.3.2 The upper limit of 60,000 psi on the value of f_{yt} used in design is intended to control cracking.

R22.6.4 *Critical sections for two-way shear*

R22.6.4.1 The critical section defined in 22.6.4.1(a) for two-way shear in slabs and footings follows the perimeter at the edge of the loaded area (Joint ACI-ASCE Committee 326 1962). Loaded area for two-way shear in 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 or with shear reinforcement, it is necessary to check shear at multiple sections as defined in 22.6.4.1(a) and (b) and 22.6.4.2.

For columns near an edge or corner, the critical perimeter may extend to the edge of the slab.

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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 For members reinforced for two-way shear with headed shear reinforcement or single- or multi-leg stirrups, a critical section with perimeter b_o located $d/2$ beyond the outermost peripheral line of shear reinforcement shall also be considered. The shape of this critical section shall be a polygon selected to minimize b_o.

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R22.6.4.2 For members reinforced for two-way shear with stirrup or headed stud shear reinforcement, it is required to check shear stress in concrete at a critical section located a distance $d/2$ beyond the point where shear reinforcement is discontinued. Calculated shear stress at this section must not exceed the limits given in expressions (a) and (e) in Table 22.6.6.1. The shape of this outermost critical section should correspond to the minimum value of b_o, as depicted in Fig. R22.6.4.2a, b, and c. Note that these figures depict slabs reinforced with stirrups. The shape of the outermost critical section is similar for slabs with headed shear reinforcement. The square or rectangular critical sections described in 22.6.4.1.1 will not result in the minimum value of b_o for the cases depicted in these figures. Additional critical section checks are required at a distance $d/2$ beyond any point where variations in shear reinforcement occur, such as changes in size, spacing, or configuration.

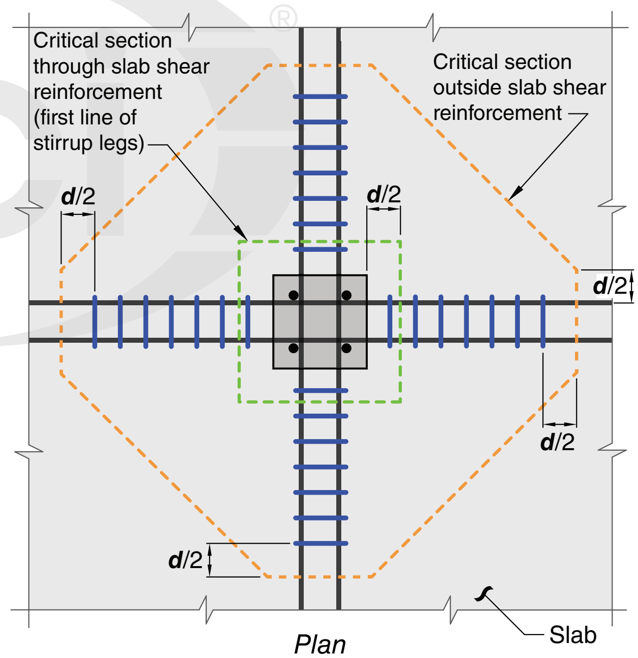

Fig. R22.6.4.2a—Critical sections for two-way shear in slab with shear reinforcement at interior column.

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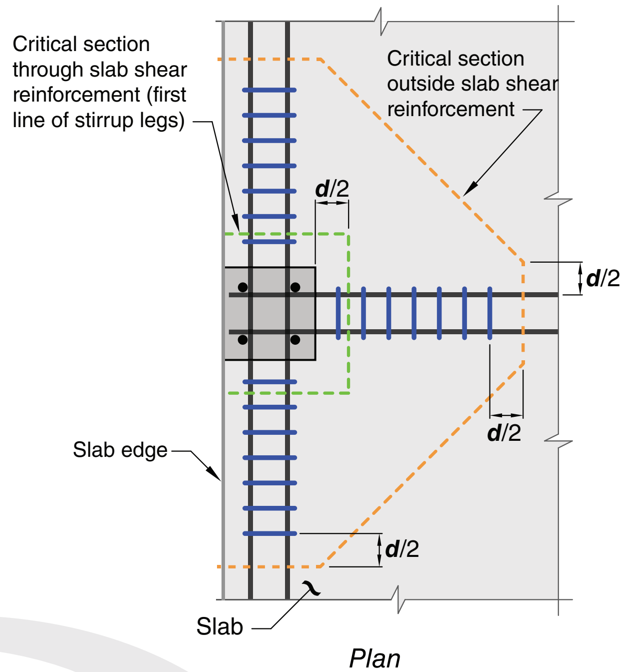

Fig. R22.6.4.2b—Critical sections for two-way shear in slab with shear reinforcement at edge column.

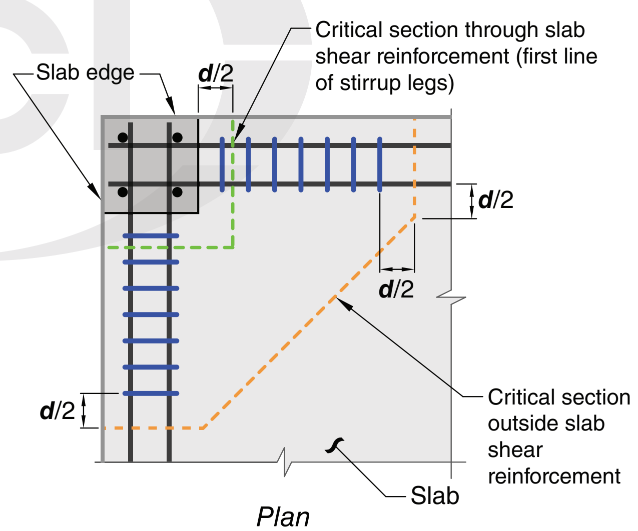

Fig. R22.6.4.2c—Critical sections for two-way shear in slab with shear reinforcement at corner column.

22.6.4.3 If an opening is located closer than $4h$ from the periphery of a column, concentrated load, or reaction area, the 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.

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.

Research (Genikomsou and Polak 2017) has shown that when openings are located at distances greater than $4d$ from the periphery of a column, the punching shear strength is the same as that for a slab without openings.

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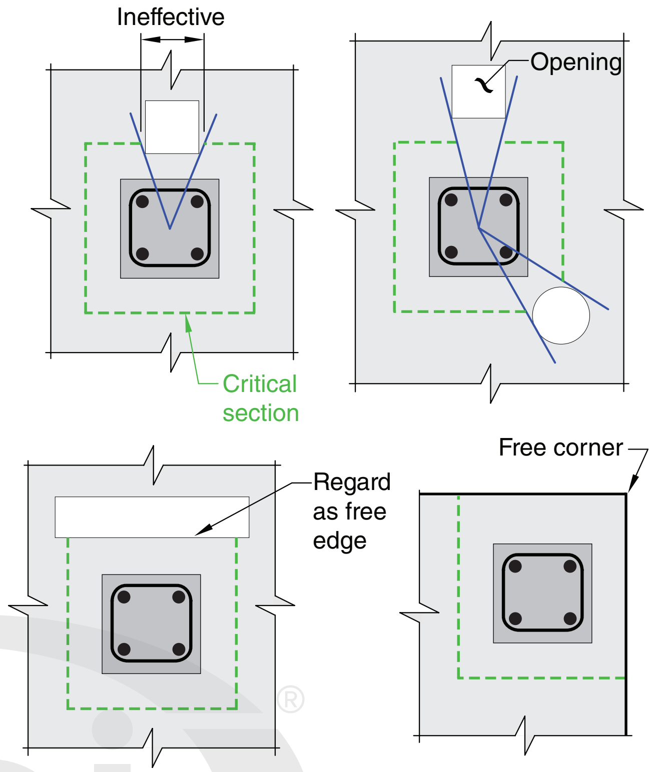

Fig. R22.6.4.3—Effect of openings and free edges (effective perimeter shown with dashed lines).

22.6.5 *Two-way shear strength provided by concrete in members without shear reinforcement*

22.6.5.1 For nonprestressed members, v_c shall be calculated in accordance with 22.6.5.2. For prestressed members, v_c shall be calculated in accordance with (a) or (b):

(a) 22.6.5.2
(b) 22.6.5.5, if the conditions of 22.6.5.4 are satisfied

22.6.5.2 v_c shall be calculated in accordance with Table 22.6.5.2.

Table 22.6.5.2—Two-way shear strength v_c for members without shear reinforcement

	v_c	
Least of (a), (b), and (c):	$4\lambda_s\lambda\sqrt{f_c'}$	(a)
	$\left(2+\frac{4}{\beta}\right)\lambda_s\lambda\sqrt{f_c'}$	(b)
	$\left(2+\frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f_c'}$	(c)

Notes: (i) λ_s is the size effect factor given in 22.5.5.1.3. (ii) β is the ratio of long to short sides of the column, concentrated load, or reaction area. (iii) α_s is given in 22.6.5.3.

R22.6.5 *Two-way shear strength provided by concrete in members without shear reinforcement*

R22.6.5.2 Experimental evidence indicates that the measured two-way concrete shear strength of members without shear reinforcement does not increase in direct proportion with member depth. This phenomenon is referred to as the "size effect." The modification factor λ_s accounts for the dependence of two-way shear strength of slabs on effective depth.

For nonprestressed two-way slabs without a minimum amount of shear reinforcement and with $d > 10$ in., the size effect specified in 22.5.5.1.3 reduces the shear strength of two-way slabs below $4\sqrt{f_c'}b_od$ (Hawkins and Ospina 2017; Dönmez and Bažant 2017).

For square columns, the stress corresponding to the nominal two-way shear strength provided by concrete in slabs subjected to bending in two directions is limited to $4\lambda_s\sqrt{f_c'}$. However, tests (Joint ACI-ASCE Committee 426 1974)

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have indicated that the value of $4\lambda_s\sqrt{f_c'}$ is unconservative when the ratio β of the lengths of the long and short sides of a rectangular column or loaded area is larger than 2.0. In such cases, the actual shear stress on the critical section at punching shear failure varies from a maximum of approximately $4\lambda_s\sqrt{f_c'}$ around the corners of the column or loaded area, down to $2\lambda_s\sqrt{f_c'}$ or less along the long sides between the two end sections. Other tests (Vanderbilt 1972) indicate that v_c decreases as the ratio b_o/d increases. Expressions (b) and (c) in Table 22.6.5.2 were developed to account for these two effects.

For shapes other than rectangular, β is taken to be the ratio of the longest overall dimension of the effective loaded area to the largest overall perpendicular dimension of the effective loaded area, as illustrated for an L-shaped reaction area in Fig. R22.6.5.2. The effective loaded area is that area totally enclosing the actual loaded area, for which the perimeter is a minimum.

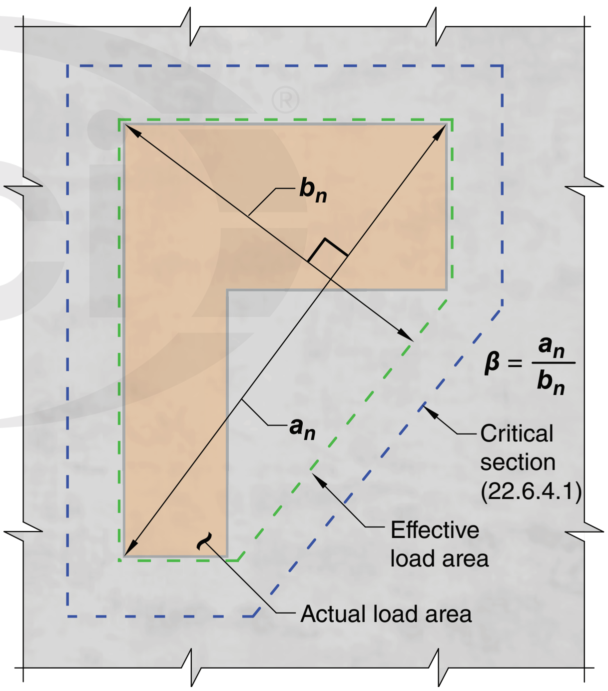

Fig. R22.6.5.2—Value of β for a nonrectangular loaded area.

22.6.5.3 The value of α_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns.

R22.6.5.3 The terms "interior columns," "edge columns," and "corner columns" in this provision refer to critical sections with a continuous slab on four, three, and two sides, respectively.

22.6.5.4 For two-way shear in prestressed members, it shall be permitted to calculate v_c using 22.6.5.5, provided that (a) through (c) are satisfied:

(a) Bonded reinforcement is provided in accordance with 8.6.2.3 and 8.7.5.3

(b) No portion of the column cross section is closer to a discontinuous edge than four times the slab thickness h

R22.6.5.4 For prestressed members, modified forms of expressions (b) and (c) in Table 22.6.5.2 are specified. Research (ACI PRC-423.3) indicates that the shear strength of two-way prestressed slabs around interior columns is conservatively calculated by the expressions in 22.6.5.5, where v_c corresponds to a diagonal tension failure of the concrete initiating at the critical section defined in 22.6.4.1. The mode of failure differs from a punching shear failure around the perimeter of the loaded

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(c) Effective prestress f_{pc} in each direction is not less than 125 psi

22.6.5.5 For two-way shear in prestressed members conforming to 22.6.5.4, v_c shall be permitted to be the lesser of (a) and (b)

$$\text{(a) } v_c = 3.5\lambda\sqrt{f_c'} + 0.3f_{pc} + \frac{V_p}{b_o d} \quad (22.6.5.5a)$$

$$\text{(b) } v_c = \left(1.5 + \frac{\alpha_s d}{b_o}\right)\lambda\sqrt{f_c'} + 0.3f_{pc} + \frac{V_p}{b_o d} \quad (22.6.5.5b)$$

where α_s is given in 22.6.5.3; the value of f_{pc} is the average of f_{pc} in the two directions and shall not exceed 500 psi; V_p is the vertical component of all effective prestress forces crossing the critical section; and the value of $\sqrt{f_c'}$ shall not exceed 70 psi.

22.6.6 *Two-way shear strength provided by concrete in members with shear reinforcement*

22.6.6.1 For members where shear reinforcement is required to resist two-way shear, v_c at critical sections shall be calculated in accordance with Table 22.6.6.1.

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area of a nonprestressed slab calculated using expression (b) in Table 22.6.5.2. Consequently, the expressions in 22.6.5.5 differ from those for nonprestressed slabs. Values for $\sqrt{f_c'}$ and f_{pc} are restricted in design due to limited test data available beyond the specified limits. When calculating f_{pc}, loss of prestress due to restraint of the slab by structural walls and other structural elements should be taken into account.

R22.6.6 *Two-way shear strength provided by concrete in members with shear reinforcement*

Critical sections for members with shear reinforcement are defined in 22.6.4.1 for the sections adjacent to the column, concentrated load, or reaction area, and 22.6.4.2 for the section located just beyond the outermost peripheral line of stirrup or headed shear stud reinforcement. Values of maximum v_c for these critical sections are given in Table 22.6.6.1. Limiting values of v_u for the critical sections defined in 22.6.4.1 are given in Table 22.6.6.3.

The maximum v_c and limiting value of v_u at the innermost critical section (defined in 22.6.4.1) are higher where headed shear stud reinforcement is provided than the case where stirrups are provided (refer to R8.7.7). Maximum v_c values at the critical sections defined in 22.6.4.2 beyond the outermost peripheral line of shear reinforcement are independent of the type of shear reinforcement provided.

R22.6.6.1 For slabs with stirrups, the maximum value of v_c is taken as $2\lambda_s\lambda\sqrt{f_c'}$ because the stirrups resist all the shear beyond that at inclined cracking (which occurs at approximately half the capacity of a slab without shear reinforcement (that is, $0.5 \times 4\lambda_s\lambda\sqrt{f_c'} = 2\lambda_s\lambda\sqrt{f_c'}$) (Hawkins 1974). The higher value of v_c for slabs with headed shear stud reinforcement is based on research (Elgabry and Ghali 1987).