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8.4.1.9 Combining the results of a gravity load analysis with the results of a lateral load analysis shall be permitted.

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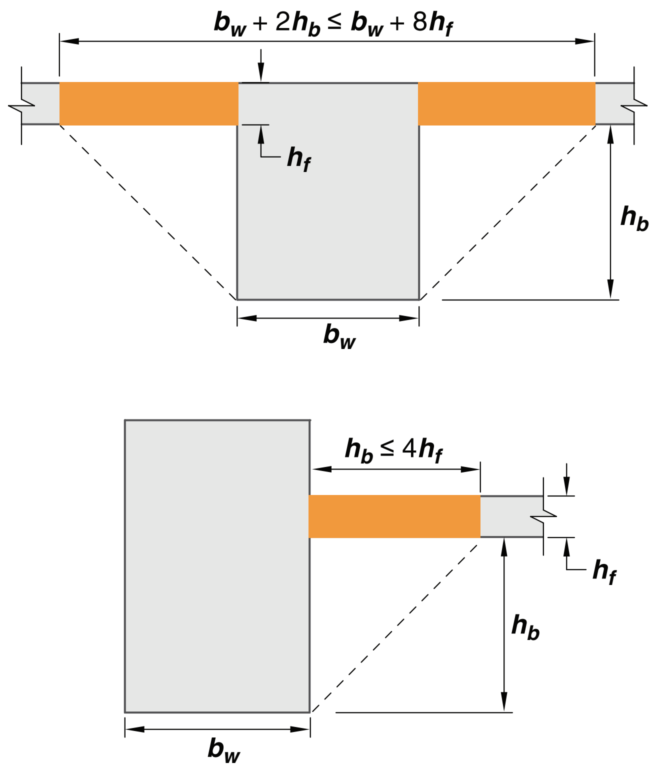

Fig. R8.4.1.8*—Examples of the portion of slab to be included with the beam under 8.4.1.8.*

8.4.2 *Factored moment*

R8.4.2 *Factored moment*

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

8.4.2.2 *Factored slab moment resisted by the column*

R8.4.2.2 *Factored slab moment resisted by the column*

8.4.2.2.1 If gravity, wind, earthquake, or other loads cause a transfer of moment between the slab and column, a fraction of M_{sc}, the factored slab moment resisted by the column at a joint, shall be transferred by flexure in accordance with 8.4.2.2.2 through 8.4.2.2.5.

R8.4.2.2.1 This section is concerned primarily with slab systems without beams.

8.4.2.2.2 The fraction of factored slab moment resisted by the column, $\gamma_f M_{sc}$, shall be assumed to be transferred by flexure, where γ_f shall be calculated by:

$$\gamma_f = \frac{1}{1+\left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \qquad (8.4.2.2.2)$$

8.4.2.2.3 The effective slab width b_{slab} for resisting $\gamma_f M_{sc}$ shall be the width of column or capital plus a distance on each side in accordance with Table 8.4.2.2.3.

R8.4.2.2.3 Unless measures are taken to resist the torsional and shear stresses, all reinforcement resisting that part of the moment to be transferred to the column by flexure should

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Table 8.4.2.2.3—Dimensional limits for effective slab width

	Distance on each side of column or capital	
Without drop panel or shear cap	Lesser	1.5*h* of slab
		Distance to edge of slab
With drop panel or shear cap	Lesser	1.5*h* of drop or cap
		Distance to edge of the drop or cap plus 1.5*h* of slab

8.4.2.2.4 For nonprestressed slabs, where the limitations on v_{uv} and ε_t in Table 8.4.2.2.4 are satisfied, γ_f shall be permitted to be increased to the maximum modified values provided in Table 8.4.2.2.4, where v_c is calculated in accordance with 22.6.5.

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be placed between lines that are one and one-half the slab or drop panel thickness, **1.5*h***, on each side of the column.

R8.4.2.2.4 Some flexibility in distribution of M_{sc} transferred by shear and flexure at both exterior and interior columns is possible. Interior, exterior, and corner columns refer to slab-column connections for which the critical perimeter for rectangular columns has four, three, and two sides, respectively.

At exterior columns, for M_{sc} resisted about an axis parallel to the edge, the portion of moment transferred by eccentricity of shear $\gamma_v M_{sc}$ may be reduced, provided that the factored shear at the column (excluding the shear produced by moment transfer) does not exceed 75 percent of the shear strength ϕv_c as defined in 22.6.5.1 for edge columns, or 50 percent for corner columns. Tests (Moehle 1988; ACI 352.1R) indicate that there is no significant interaction between shear and M_{sc} at the exterior column in such cases. Note that as $\gamma_v M_{sc}$ is decreased, $\gamma_f M_{sc}$ is increased.

At interior columns, some flexibility in distributing M_{sc} transferred by shear and flexure is possible, but with more severe limitations than for exterior columns. For interior columns, M_{sc} transferred by flexure is permitted to be increased up to 25 percent, provided that the factored shear (excluding the shear caused by the moment transfer) at the interior columns does not exceed 40 percent of the shear strength ϕv_c as defined in 22.6.5.1.

If the factored shear for a slab-column connection is large, the slab-column joint cannot always develop all of the reinforcement provided in the effective width. The modifications for interior slab-column connections in this provision are permitted only where the reinforcement required to develop $\gamma_f M_{sc}$ within the effective width has a net tensile strain ε_t not less than ε_{ty} **+ 0.008**, where the value of ε_{ty} is determined in 21.2.2. The use of Eq. (8.4.2.2.2) without the modification permitted in this provision will generally indicate overstress conditions on the joint. This provision is intended to improve ductile behavior of the slab-column joint. If reversal of moments occurs at opposite faces of an interior column, both top and bottom reinforcement should be concentrated within the effective width. A ratio of top-to-bottom reinforcement of approximately 2 has been observed to be appropriate.

Before the 2019 Code, the strain limits on ε_t in Table 8.4.2.2.4 were constants of 0.004 and 0.010. Beginning with the 2019 Code, to accommodate nonprestressed reinforcement

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of higher grades, these limits are replaced by the expressions $\boldsymbol{\varepsilon_{ty}}$ **+ 0.003** and $\boldsymbol{\varepsilon_{ty}}$ **+ 0.008**, respectively. The first expression is the same expression as used for the limit on $\boldsymbol{\varepsilon_t}$ for classification of tension-controlled members in Table 21.2.2; this expression is further described in Commentary R21.2.2. The second expression provides a limit on $\boldsymbol{\varepsilon_t}$ with Grade 420 reinforcement that is approximately the same value as the former constant of 0.010.

Table 8.4.2.2.4—Maximum modified values of γ_f for nonprestressed two-way slabs

Column location	Span direction	v_{uv}	ε_t (within b_{slab})	Maximum modifie γ_f
Corner column	Either direction	$\leq 0.5\phi v_c$	$\geq \varepsilon_{ty} + 0.003$	1.0
Edge column	Perpendicular to the edge	$\leq 0.75\phi v_c$	$\geq \varepsilon_{ty} + 0.003$	1.0
	Parallel to the edge	$\leq 0.4\phi v_c$	$\geq \varepsilon_{ty} + 0.008$	$\dfrac{1.25}{1+\left(\dfrac{2}{3}\right)\sqrt{\dfrac{b_1}{b_2}}} \leq 1.0$
Interior column	Either direction	$\leq 0.4\phi v_c$	$\geq \varepsilon_{ty} + 0.008$	$\dfrac{1.25}{1+\left(\dfrac{2}{3}\right)\sqrt{\dfrac{b_1}{b_2}}} \leq 1.0$

8.4.2.2.5 Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist moment on the effective slab width defined in 8.4.2.2.2 and 8.4.2.2.3.

8.4.2.2.6 The fraction of $\boldsymbol{M_{sc}}$ not calculated to be resisted by flexure shall be assumed to be resisted by eccentricity of shear in accordance with 8.4.4.2.

8.4.3 *Factored one-way shear*

8.4.3.1 For slabs built integrally with supports, $\boldsymbol{V_u}$ at the support shall be permitted to be calculated at the face of support.

8.4.3.2 Sections between the face of support and a critical section located $\boldsymbol{d}$ from the face of support for nonprestressed slabs and $\boldsymbol{h/2}$ from the face of support for prestressed slabs shall be permitted to be designed for $\boldsymbol{V_u}$ at that critical section if (a) through (c) are satisfied:

(a) Support reaction, in direction of applied shear, introduces compression into the end regions of the slab.
(b) Loads are applied at or near the top surface of the slab.
(c) No concentrated load occurs between the face of support and critical section.

8.4.4 *Factored two-way shear*

R8.4.4 *Factored two-way shear*

The calculated shear stresses in the slab around the column are required to conform to the requirements of 22.6.

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8.4.4.1 *Critical section*

8.4.4.1.1 Slabs shall be evaluated for two-way shear in the vicinity of columns, concentrated loads, and reaction areas at critical sections in accordance with 22.6.4.

8.4.4.1.2 Slabs reinforced with stirrups or headed shear stud reinforcement shall be evaluated for two-way shear at critical sections in accordance with 22.6.4.2.

8.4.4.2 *Factored two-way shear stress due to shear and factored slab moment resisted by the column*

8.4.4.2.1 For two-way shear with factored slab moment resisted by the column, factored shear stress v_u shall be calculated at critical sections in accordance with 8.4.4.1. Factored shear stress v_u corresponds to a combination of v_{uv} and the shear stress produced by $\gamma_v M_{sc}$, where γ_v is given in 8.4.4.2.2 and M_{sc} is given in 8.4.2.2.1.

8.4.4.2.2 The fraction of M_{sc} transferred by eccentricity of shear, $\gamma_v M_{sc}$, shall be applied at the centroid of the critical section in accordance with 8.4.4.1, where:

$$\gamma_v = 1 - \gamma_f \qquad (8.4.4.2.2)$$

8.4.4.2.3 The factored shear stress resulting from $\gamma_v M_{sc}$ shall be assumed to vary linearly about the centroid of the critical section in accordance with 8.4.4.1.

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R8.4.4.2 *Factored two-way shear stress due to shear and factored slab moment resisted by the column*

R8.4.4.2.2 Hanson and Hanson (1968) found that where moment is transferred between a column and a slab, 60 percent of the moment should be considered transferred by flexure across the perimeter of the critical section defined in 22.6.4.1, and 40 percent by eccentricity of the shear about the centroid of the critical section. For rectangular columns, the portion of the moment transferred by flexure increases as the width of the face of the critical section resisting the moment increases, as given by Eq. (8.4.2.2.2).

Most of the data in Hanson and Hanson (1968) were obtained from tests of square columns. Limited information is available for round columns; however, these can be approximated as square columns having the same cross-sectional area.

R8.4.4.2.3 The stress distribution is assumed as illustrated in Fig. R8.4.4.2.3 for an interior or exterior column. The perimeter of the critical section, ***ABCD***, is determined in accordance with 22.6.4.1. The factored shear stress v_{uv} and factored slab moment resisted by the column M_{sc} are determined at the centroidal axis ***c-c*** of the critical section. The maximum factored shear stress may be calculated from:

$$v_{u,AB} = v_{uv} + \frac{\gamma_v M_{sc} c_{AB}}{J_c}$$

or

$$v_{u,CD} = v_{uv} - \frac{\gamma_v M_{sc} cCD}{J_c}$$

where γ_v is given by Eq. (8.4.4.2.2).

For an interior column, J_c may be calculated by:

J_c = property of assumed critical section analogous to polar moment of inertia

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$$= \frac{d(c_1+d)^3}{6} + \frac{(c_1+d)d^3}{6} + \frac{d(c_2+d)(c_1+d)^2}{2}$$

Similar equations may be developed for J_c for columns located at the edge or corner of a slab.

The fraction of M_{sc} not transferred by eccentricity of the shear should be transferred by flexure in accordance with 8.4.2.2. A conservative method assigns the fraction transferred by flexure over an effective slab width defined in 8.4.2.2.3. Often, column strip reinforcement is concentrated near the column to accommodate M_{sc}. Available test data (Hanson and Hanson 1968) seem to indicate that this practice does not increase shear strength but may be desirable to increase the stiffness of the slab-column junction.

Test data (Hawkins 1981) indicate that the moment transfer strength of a prestressed slab-to-column connection can be calculated using the procedures of 8.4.2.2 and 8.4.4.2.

Where shear reinforcement has been used, the critical section beyond the shear reinforcement generally has a polygonal shape (Fig. R8.7.6(d) and (e)). Equations for calculating shear stresses on such sections are given in ACI 421.1R.

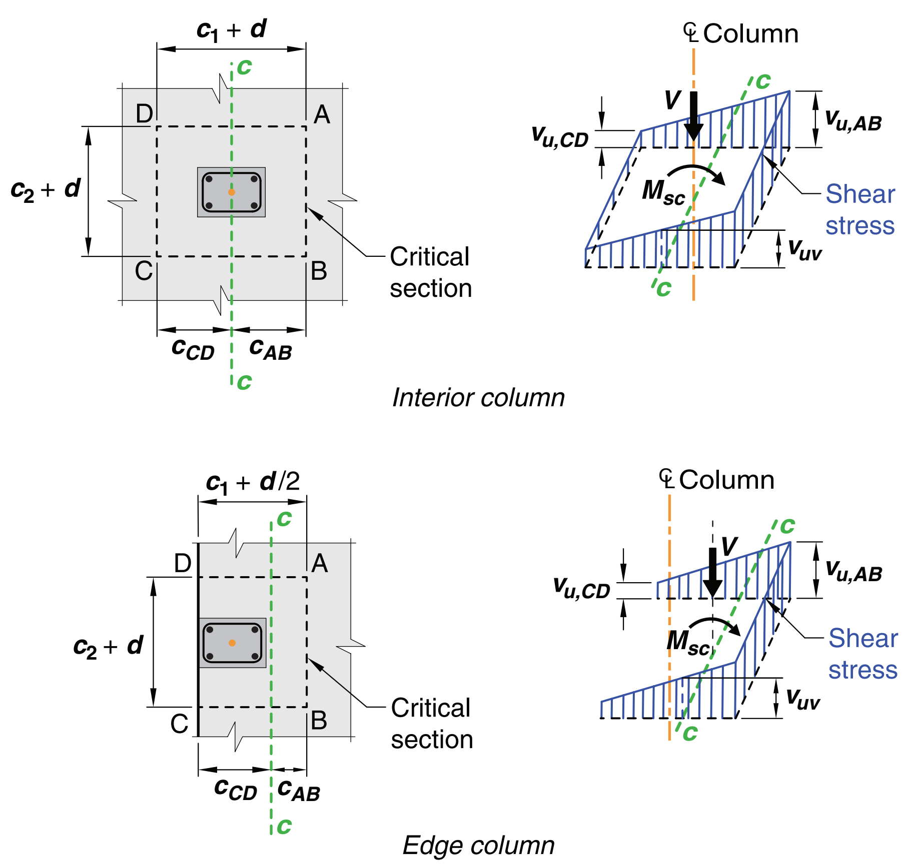

Fig. R8.4.4.2.3—Assumed distribution of shear stress.

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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 shear strength of two-way members with and without shear reinforcement.

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

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

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

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

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 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.

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

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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, 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 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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22.6.2 *Effective depth*

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

22.6.2.2 For prestressed, two-way members, $\boldsymbol{d}$ need not be taken less than $\mathbf{0.8}\boldsymbol{h}$.

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 8.3 MPa.

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

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 70 MPa, it is prudent to limit $\sqrt{f_c'}$ to 8.3 MPa for the calculation of shear strength.

R22.6.3.2 The upper limit of 420 MPa on the value of f_{yt} used in design is intended to control cracking.

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 For two-way members reinforced 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.

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 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.

R22.6.4.2 For two-way members 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 for the cases depicted in these figures. Additional

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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.

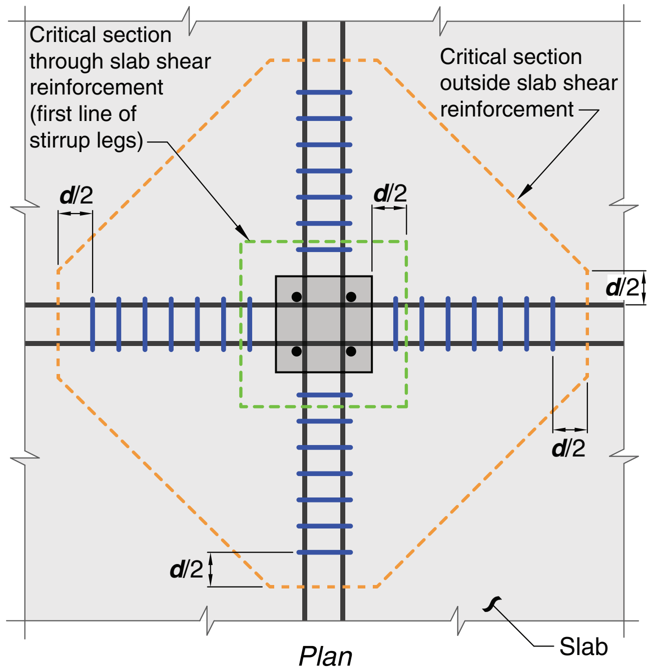

***Fig. R22.6.4.2a**—Critical sections for two-way shear in slab with shear reinforcement at interior column.*

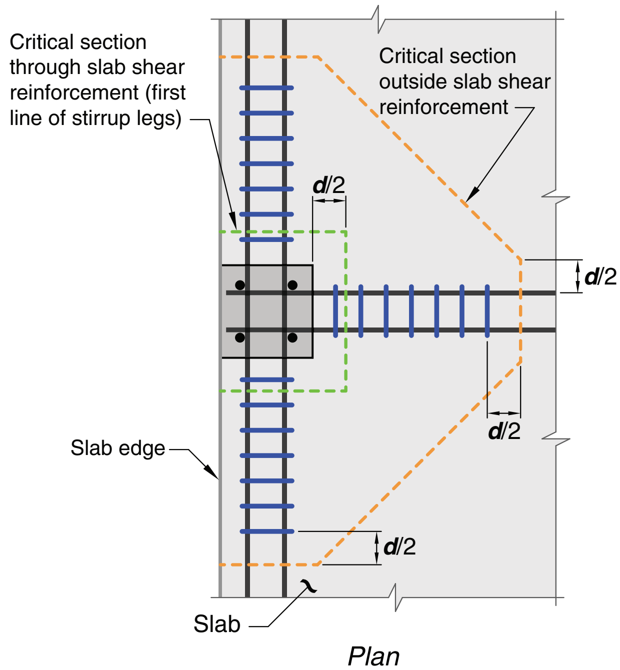

***Fig. R22.6.4.2b**—Critical sections for two-way shear in slab with shear reinforcement at edge column.*

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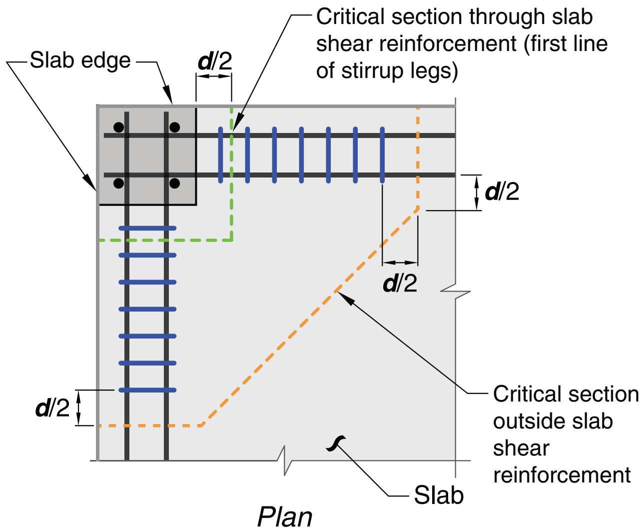

Fig. R22.6.4.2c—Critical sections for two-way shear in slab with shear reinforcement at corner column.

22.6.4.3 If an opening is located closer than **4*h*** 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 **4*d*** from the periphery of a column, the punching shear strength is the same as that for a slab without openings.

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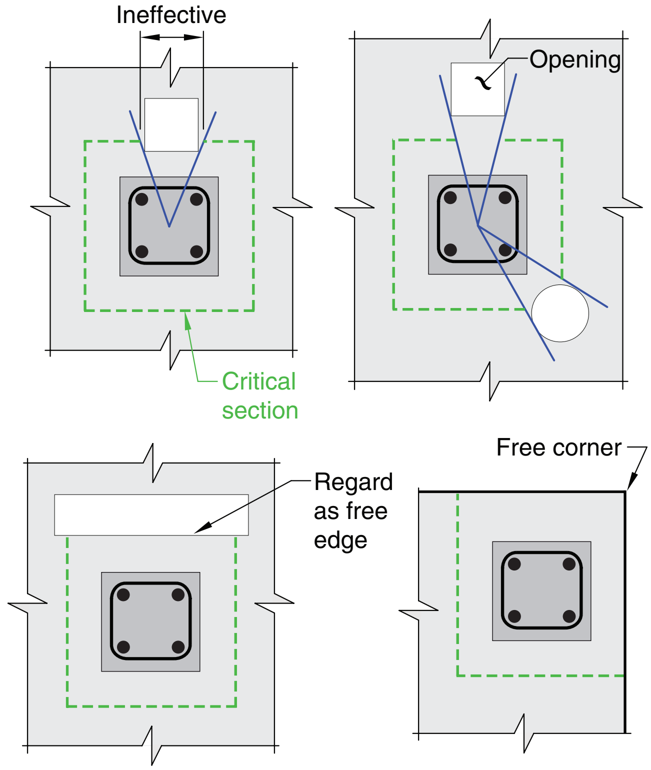

***Fig. R22.6.4.3**—Effect of openings and free edges (effective perimeter shown with dashed lines).*

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

22.6.5.1 For nonprestressed two-way members, v_c shall be calculated in accordance with 22.6.5.2. For prestressed two-way 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.

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 concrete shear strength of two-way 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 $d > 250$ mm, the size effect specified in 22.5.5.1.3 reduces the shear strength of two-way slabs below $0.33\sqrt{f_c'}b_od$ (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

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Table 22.6.5.2—v_c for two-way members without shear reinforcement

v_c		
Least of (a), (b), and (c):	$0.33\lambda_s\lambda\sqrt{f_c'}$	(a)
	$\left(0.17+\frac{0.33}{\beta}\right)\lambda_s\lambda\sqrt{f_c'}$	(b)
	$\left(0.17+\frac{0.083\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f_c'}$	(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.

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

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

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slabs subjected to bending in two directions is limited to $0.33\lambda_s\sqrt{f_c'}$. However, tests (Joint ACI-ASCE Committee 426 1974) have indicated that the value of $0.33\lambda_s\sqrt{f_c'}$ 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 $0.33\lambda_s\sqrt{f_c'}$ around the corners of the column or loaded area, down to $0.17\lambda_s\sqrt{f_c'}$ 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.

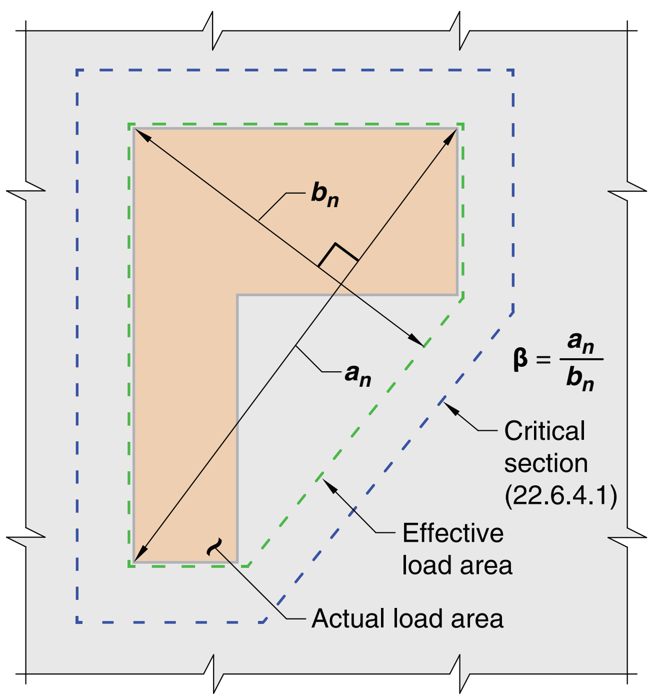

Fig. R22.6.5.2—Value of β for a nonrectangular loaded area.

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.

R22.6.5.4 For prestressed two-way members, modified forms of expressions (b) and (c) in Table 22.6.5.2 are specified. Research (ACI 423.3R) indicates that the shear strength of two-way prestressed slabs around interior columns is

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(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

(c) Effective prestress f_{pc} in each direction is not less than 0.9 MPa

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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 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.

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

(a) $$v_c = 0.29\lambda\sqrt{f_c'} + 0.3f_{pc} + \frac{V_p}{b_o d} \quad (22.6.5.5a)$$

(b) $$v_c = 0.083\left(1.5 + \frac{\alpha_s d}{b_o}\right)\lambda\sqrt{f_c'} + 0.3f_{pc} + \frac{V_p}{b_o d} \quad (22.6.5.5b)$$

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 3.5 MPa; 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 5.8 MPa.

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

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

Critical sections for two-way 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.

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

R22.6.6.1 For two-way slabs with stirrups, the maximum value of v_c is taken as $0.17\lambda_s\lambda\sqrt{f_c'}$ 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, $0.5 \times 0.33\lambda_s\lambda\sqrt{f_c'} = 0.17\lambda_s\lambda\sqrt{f_c'}$) (Hawkins 1974). The higher value of v_c for two-way slabs with headed shear stud reinforcement is based on research (Elgabry and Ghali 1987).

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Table 22.6.6.1—v_c for two-way members with shear reinforcement

Type of shear reinforcement	Critical sections	v_c		
Stirrups	All	$0.17\lambda_s\lambda\sqrt{f_c'}$		(a)
Headed shear stud reinforcement	According to 22.6.4.1	Least of (b), (c), and (d):	$0.25\lambda_s\lambda\sqrt{f_c'}$	(b)
			$0.17\left(1+\frac{2}{\beta}\right)\lambda_s\lambda\sqrt{f_c'}$	(c)
			$0.083\left(2+\frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f_c'}$	(d)
	According to 22.6.4.2		$0.17\lambda_s\lambda\sqrt{f_c'}$	(e)

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.

22.6.6.2 It shall be permitted to take $\boldsymbol{\lambda_s}$ as 1.0 if (a) or (b) is satisfied:

(a) Stirrups are designed and detailed in accordance with 8.7.6 and $\boldsymbol{A_v/s \geq 0.17\sqrt{f_c'}\,b_o/f_{yt}}$.

(b) Smooth headed shear stud reinforcement with stud shaft length not exceeding 250 mm is designed and detailed in accordance with 8.7.7 and $\boldsymbol{A_v/s \geq 0.17\sqrt{f_c'}\,b_o/f_{yt}}$.

COMMENTARY

R22.6.6.2 The size effect in slabs with $\boldsymbol{d > 250}$ **mm** can be mitigated if a minimum amount of shear reinforcement is provided. The ability of ordinary (smooth) headed shear stud reinforcement to effectively mitigate the size effect on the two-way shear strength of slabs may be compromised if studs longer than 250 mm are used. Until experimental evidence becomes available, it is not permitted to use $\boldsymbol{\lambda_s}$ equal to 1.0 for slabs with $\boldsymbol{d > 250}$ **mm** without headed shear stud reinforcement with stud shaft length not exceeding 250 mm. Stacking or "piggybacking" of headed shear studs, as shown in Fig. R22.6.6.2, introduces an intermediate head that contributes to further anchor the stacked stud.

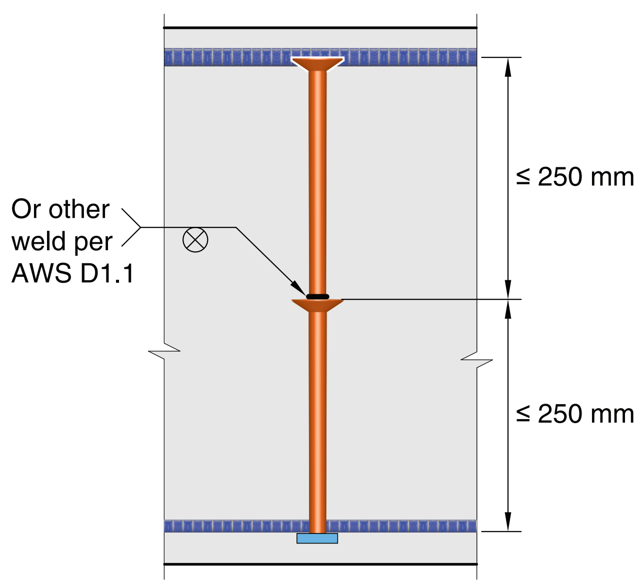

Fig. R22.6.6.2—Stacking (piggybacking) of headed shear stud reinforcement.

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22.6.6.3 For two-way members with shear reinforcement, effective depth shall be selected such that v_u calculated at critical sections does not exceed the values in Table 22.6.6.3.

Table 22.6.6.3—Maximum v_u for two-way members with shear reinforcement

Type of shear reinforcement	Maximum v_u at critical sections defined in 22.6.4.	
Stirrups	$0.5\phi\sqrt{f_c'}$	(a)
Headed shear stud reinforcement	$0.66\phi\sqrt{f_c'}$	(b)

22.6.7 *Two-way shear strength provided by single- or multiple-leg stirrups*

22.6.7.1 Single- or multiple-leg stirrups fabricated from bars or wires shall be permitted to be used as shear reinforcement in slabs and footings satisfying (a) and (b):

(a) d is at least 150 mm

(b) d is at least $16d_b$, where d_b is the diameter of the stirrups

22.6.7.2 For two-way members with stirrups, v_s shall be calculated by:

$$v_s = \frac{A_v f_{yt}}{b_o s} \qquad (22.6.7.2)$$

where A_v is the sum of the area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the column section, and s is the spacing of the peripheral lines of shear reinforcement in the direction perpendicular to the column face.

22.6.8 *Two-way shear strength provided by headed shear stud reinforcement*

22.6.8.1 Headed shear stud reinforcement shall be permitted to be used as shear reinforcement in slabs and footings if the placement and geometry of the headed shear stud reinforcement satisfies 8.7.7.

22.6.8.2 For two-way members with headed shear stud reinforcement, v_s shall be calculated by:

$$v_s = \frac{A_v f_{yt}}{b_o s} \qquad (22.6.8.2)$$

COMMENTARY

R22.6.7 *Two-way shear strength provided by single- or multiple-leg stirrups*

R22.6.7.2 Because shear stresses are used for two-way shear in this chapter, shear strength provided by transverse reinforcement is averaged over the cross-sectional area of the critical section.

R22.6.8 *Two-way shear strength provided by headed shear stud reinforcement*

Tests (ACI 421.1R) show that headed shear stud reinforcement mechanically anchored as close as practicable to the top and bottom of slabs is effective in resisting punching shear. The critical section beyond the shear reinforcement is generally assumed to have a polygonal shape (refer to Fig. R22.6.4.2a, R22.6.4.2b, and R22.6.4.2c). Equations for calculating shear stresses on such sections are given in ACI 421.1R.

R22.6.8.2 Because shear stresses are used for two-way shear in this chapter, shear strength provided by transverse reinforcement is averaged over the cross-sectional area of the critical section.

CODE

where A_v is the sum of the area of all shear studs on one peripheral line that is geometrically similar to the perimeter of the column section, and s is the spacing of the peripheral lines of headed shear stud reinforcement in the direction perpendicular to the column face.

22.6.8.3 If headed shear stud reinforcement is provided, A_v/s shall satisfy:

$$\frac{A_v}{s} \geq 0.17\sqrt{f_c'}\frac{b_o}{f_{yt}} \tag{22.6.8.3}$$

22.7—Torsional strength

COMMENTARY

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 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. Both hollow and solid sections are idealized as thin-walled tubes both before and after cracking.

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. For a hollow member with continuous walls, A_o includes the area of the hole.

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 test results in MacGregor and Ghoneim (1995) and Hsu (1997).

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