CHAPTER 8—TWO-WAY SLABS

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8.1—Scope

8.1.1 This chapter shall apply to the design of nonprestressed and prestressed slabs reinforced for flexure in two directions, with or without beams between supports, including (a) through (d):

(a) Solid slabs
(b) Slabs cast on stay-in-place, noncomposite steel deck
(c) Composite slabs of concrete elements constructed in separate placements but connected so that all elements resist loads as a unit
(d) Two-way joist systems in accordance with 8.8

8.2—General

8.2.1 A slab system shall be permitted to be designed by any procedure satisfying equilibrium and geometric compatibility, provided that design strength at every section is at least equal to required strength, and all serviceability requirements are satisfied. The direct design method or the equivalent frame method is permitted.

8.2.2 The effects of concentrated loads, slab openings, and slab voids shall be considered in design.

8.2.3 Slabs prestressed with an average effective compressive stress less than 125 psi shall be designed as nonprestressed slabs.

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R8.1—Scope

R8.1.1 The design methods given in this chapter are based on analysis of results of an extensive series of tests (Burns and Hemakom 1977; Gamble et al. 1969; Gerber and Burns 1971; Guralnick and LaFraugh 1963; Hatcher et al. 1965, 1969; Hawkins 1981; Jirsa et al. 1966; PTI DC20.8; Smith and Burns 1974; Scordelis et al. 1959; Vanderbilt et al. 1969; Xanthakis and Sozen 1963) and well-established performance records of various slab systems. The fundamental design principles are applicable to all planar structural systems subjected to transverse loads. Several specific design rules, as well as historical precedents, limit the types of structures to which this chapter applies. General slab systems that may be designed according to this chapter include flat slabs, flat plates, slabs with beams in both directions, and waffle slabs. Slabs with paneled ceilings are two-way, wide-band, beam systems.

Slabs-on-ground that do not transmit vertical loads from other parts of the structure to the soil are excluded.

For slabs with beams, the design procedures of this chapter apply only where beams are located at the panel edges and where beams are supported by columns or other essentially nondeflecting supports at the corners of the panel. Two-way slabs with beams in one direction, with both slab and beams supported by girders in the other direction, may be designed under the general requirements of this chapter. Such designs should be based upon analysis compatible with the deflected position of the supporting beams and girders.

For slabs supported on walls, the design procedures in this chapter treat the wall as a beam of infinite stiffness; therefore, each wall should support the entire length of an edge of the panel (refer to 8.4.1.7). Each wall with a width less than a full panel length can be treated as a column support.

R8.2—General

R8.2.1 This section permits a design to be based directly on fundamental principles of structural mechanics, provided it can be demonstrated all strength and serviceability criteria are satisfied. Design of the slab may be achieved through combined use of classic solutions based on a linearly elastic continuum, numerical solutions based on discrete elements, or yield-line analyses, including, in all cases, evaluation of the stress conditions around supports in relation to shear, torsion, and flexure, as well as the effects of reduced stiffness of elements due to cracking and support geometry.

The direct design method and equivalent frame method are only suitable for orthogonal frames subject to gravity loads.

R8.2.2 Refer to R7.2.1.

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8.2.4 A drop panel in a nonprestressed slab, where used to reduce the minimum required thickness in accordance with 8.3.1.1 or the quantity of deformed negative moment reinforcement at a support in accordance with 8.5.2.2, shall satisfy (a) and (b):

(a) The drop panel shall project below the slab at least one-fourth of the adjacent slab thickness.
(b) The drop panel shall extend in each direction from the centerline of support a distance not less than one-sixth the span length measured from center-to-center of supports in that direction.

8.2.5 A shear cap, where used to increase the critical section for shear at a slab-column joint, shall project below the slab soffit and extend horizontally from the face of the column a distance at least equal to the thickness of the projection below the slab soffit.

8.2.6 *Materials*

8.2.6.1 Design properties for concrete shall be selected to be in accordance with Chapter 19.

8.2.6.2 Design properties for steel reinforcement shall be selected to be in accordance with Chapter 20.

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

8.2.7 *Connections to other members*

8.2.7.1 For cast-in-place construction, joints shall satisfy Chapter 15.

8.3—Design limits

8.3.1 *Minimum slab thickness*

8.3.1.1 For nonprestressed slabs without interior beams spanning between supports on all sides, having a maximum ratio of long-to-short span of 2, overall slab thickness h shall not be less than the limits in Table 8.3.1.1, and shall be at least the value in (a) or (b), unless the calculated deflection limits of 8.3.2 are satisfied:

(a) Slabs without drop panels as given in 8.2.4..........5 in.

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R8.2.4 Drop panel dimensions specified in 8.2.4 are necessary when reducing the amount of negative moment reinforcement following 8.5.2.2 or to satisfy minimum slab thicknesses permitted in 8.3.1.1. If the dimensions are less than specified in 8.2.4, the projection may be used as a shear cap to increase the shear strength of the slab. For slabs with changes in thickness, it is necessary to check the shear strength at multiple sections (refer to 22.6.4.1(b)).

R8.2.7 *Connections to other members*

R8.2.7.1 Calculation of the design strength of a slab system requires consideration of the transmission of load from the slab to the columns by moment, torsion, and shear.

R8.3—Design limits

R8.3.1 *Minimum slab thickness*

Minimum slab thicknesses in 8.3.1.1 and 8.3.1.2 are independent of loading and concrete modulus of elasticity. These minimum thicknesses are not applicable to slabs with unusually heavy superimposed sustained loads or for concrete with modulus of elasticity significantly lower than that of normalweight concrete. Deflections should be calculated for such situations.

R8.3.1.1 Minimum thicknesses in Table 8.3.1.1 are those that have been developed through the years. Use of longitudinal reinforcement with f_y > 80,000 psi may result in larger long-term deflections than in the case of $f_y \leq 80,000$ psi unless service stresses in reinforcement calculated for cracked sections are less than 40,000 psi. For $f_y > 80,000$ psi, deflections are required to be calculated.

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(b) Slabs with drop panels as given in 8.2.4...............4 in.

For f_y exceeding 80,000 psi, 8.3.2 shall be satisfied.

Table 8.3.1.1—Minimum thickness of nonprestressed two-way slabs without interior beams (in.)[1]

f_y, psi[2]	Without drop panels[3]			With drop panels[3]		
	Exterior panels			Exterior panels		
	Without edge beams	With edge beams[4]	Interior panels	Without edge beams	With edge beams[4]	Interior panels
40,000	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
60,000	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
80,000	$\ell_n/27$	$\ell_n/30$	$\ell_n/30$	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$

[1]ℓ_n is the clear span in the long direction, measured face-to-face of supports (in.).
[2]For f_y between the values given in the table, minimum thickness shall be calculated by linear interpolation. Minimum thickness values shall not be extrapolated for f_y exceeding 80,000 psi.
[3]Drop panels as given in 8.2.4.
[4]Slabs with beams between columns along exterior edges. Exterior panels shall be considered to be without edge beams if α_f is less than 0.8.

8.3.1.2 For nonprestressed slabs with beams spanning between supports on all sides, overall slab thickness h shall satisfy the limits in Table 8.3.1.2, unless the calculated deflection limits of 8.3.2 are satisfied.

Table 8.3.1.2—Minimum thickness of nonprestressed two-way slabs with beams spanning between supports on all sides

α_{fm}[1]	Minimum h, in.		
$\alpha_{fm} \le 0.2$	8.3.1.1 applies		(a)
$0.2 < \alpha_{fm} \le 2.0$	Greater of:	$\dfrac{\ell_n\left(0.8+\dfrac{f_y}{200{,}000}\right)}{36+5\beta\left(\alpha_{fm}-0.2\right)}$	(b)[1],[2]
		5.0	(c)
$\alpha_{fm} > 2.0$	Greater of:	$\dfrac{\ell_n\left(0.8+\dfrac{f_y}{200{,}000}\right)}{36+9\beta}$	(d)
		3.5	(e)

[1]α_{fm} is the average value of αf for all beams on edges of a panel.
[2]ℓ_n is the clear span in the long direction, measured face-to-face of beams (in.).
[3]β is the ratio of clear spans in long to short directions of slab.

8.3.1.2.1 At discontinuous edges of slabs conforming to 8.3.1.2, an edge beam with $\alpha_f \ge 0.80$ shall be provided, or the minimum thickness required by (b) or (d) of Table 8.3.1.2 shall be increased by at least 10% in the panel with a discontinuous edge.

8.3.1.3 The thickness of a concrete floor finish shall be permitted to be included in h if it is placed monolithically with the floor slab, or if the floor finish is designed to be composite with the floor slab in accordance with 16.4.

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R8.3.1.2 For panels having a ratio of long-to-short span greater than 2, use of expressions (b) and (d) of Table 8.3.1.2 may give unrealistic results. For such panels, the rules applying to one-way construction in 7.3.1 should be used.

R8.3.1.3 A concrete floor finish may be considered for strength purposes only if it is cast monolithically with the slab. A bonded concrete overlay or other separate concrete finish is permitted to be included in h if composite action is provided in accordance with 16.4.

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8.3.1.4 If single- or multiple-leg stirrups are used as shear reinforcement, the slab thickness shall be sufficient to satisfy the requirements for d in 22.6.7.1.

8.3.2 *Calculated deflection limits*

8.3.2.1 Immediate and time-dependent deflections shall be calculated in accordance with 24.2 and shall not exceed the limits in 24.2.2 for two-way slabs given in (a) through (c):

(a) Nonprestressed slabs not satisfying 8.3.1
(b) Nonprestressed slabs without interior beams spanning between the supports on all sides and having a ratio of long-to-short span exceeding 2.0
(c) Prestressed slabs

8.3.2.2 For nonprestressed composite concrete slabs satisfying 8.3.1.1 or 8.3.1.2, deflections occurring after the member becomes composite need not be calculated. Deflections occurring before the member becomes composite shall be investigated, unless the precomposite thickness also satisfies 8.3.1.1 or 8.3.1.2.

8.3.3 *Reinforcement strain limit in nonprestressed slabs*

8.3.3.1 Nonprestressed slabs shall be tension-controlled in accordance with Table 21.2.2.

8.3.4 *Stress limits in prestressed slabs*

8.3.4.1 Prestressed slabs shall be designed as Class U with $f_t \leq = 6\sqrt{f_c'}$. Other stresses in prestressed slabs immediately after transfer and at service loads shall not exceed the permissible stresses in 24.5.3 and 24.5.4.

8.4—Required strength

8.4.1 *General*

8.4.1.1 Required strength shall be calculated in accordance with the factored load combinations in Chapter 5.

8.4.1.2 Required strength shall be calculated in accordance with the analysis procedures given in Chapter 6.

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R8.3.2 *Calculated deflection limits*

R8.3.2.1 For prestressed flat slabs continuous over two or more spans in each direction, the span-thickness ratio generally should not exceed 42 for floors and 48 for roofs; these limits may be increased to 48 and 52, respectively, if calculations verify that both short- and long-term deflection, camber, and vibration frequency and amplitude are not objectionable.

R8.3.2.2 If any portion of a composite concrete member is prestressed, or if the member is prestressed after the components have been cast, the provisions of 8.3.2.1 apply and deflections are to be calculated. For nonprestressed concrete composite members, deflections need to be calculated and compared with the limiting values in Table 24.2.2, only when the thickness of the member or the precast part of the member is less than the minimum thickness given in Table 8.3.1.1. In unshored construction, the thickness of concern depends on whether the deflection before or after the attainment of effective composite action is being considered.

R8.3.3 *Reinforcement strain limit in nonprestressed slabs*

R8.3.3.1 The basis for a reinforcement strain limit for two-way slabs is the same as that for beams. Refer to R9.3.3 for additional information.

R8.4—Required strength

R8.4.1 *General*

R8.4.1.2 To determine service and factored moments as well as shears in prestressed slab systems, numerical analysis is required rather than simplified approaches such as the direct design method. The equivalent frame method of analysis as contained in the 2014 edition of the Code is a numerical method that has been shown by tests of large structural models to satisfactorily predict factored moments and shears in prestressed slab systems (Smith and Burns 1974; Burns and Hemakom

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1977; Hawkins 1981; PTI DC20.8; Gerber and Burns 1971; Scordelis et al. 1959). The referenced research also shows that analysis using prismatic sections or other approximations of stiffness may provide erroneous and unsafe results. Moment redistribution for prestressed slabs is permitted in accordance with 6.6.5. PTI DC20.8 provides guidance for prestressed concrete slab systems.

8.4.1.3 For prestressed slabs, effects of reactions induced by prestressing shall be considered in accordance with 5.3.14.

8.4.1.4 For a slab system supported by columns or walls, dimensions c_1, c_2, and ℓ_n shall be based on an effective support area. The effective support area is the intersection of the bottom surface of the slab, or drop panel or shear cap if present, with the largest right circular cone, right pyramid, or tapered wedge whose surfaces are located within the column and the capital or bracket and are oriented no greater than 45 degrees to the axis of the column.

8.4.1.5 A column strip is a design strip with a width on each side of a column centerline equal to the lesser of $0.25\ell_2$ and $0.25\ell_1$. A column strip shall include beams within the strip, if present.

8.4.1.6 A middle strip is a design strip bounded by two column strips.

8.4.1.7 A panel is bounded by column, beam, or wall centerlines on all sides.

R8.4.1.7 A panel includes all flexural elements between column centerlines. Thus, the column strip includes the beam, if any.

8.4.1.8 For monolithic or composite concrete construction supporting two-way slabs, a beam includes that portion of slab, on each side of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness.

R8.4.1.8 Two examples of monolithic or composite concrete beams, including the slab as a flange, are provided in Fig. R8.4.1.8.

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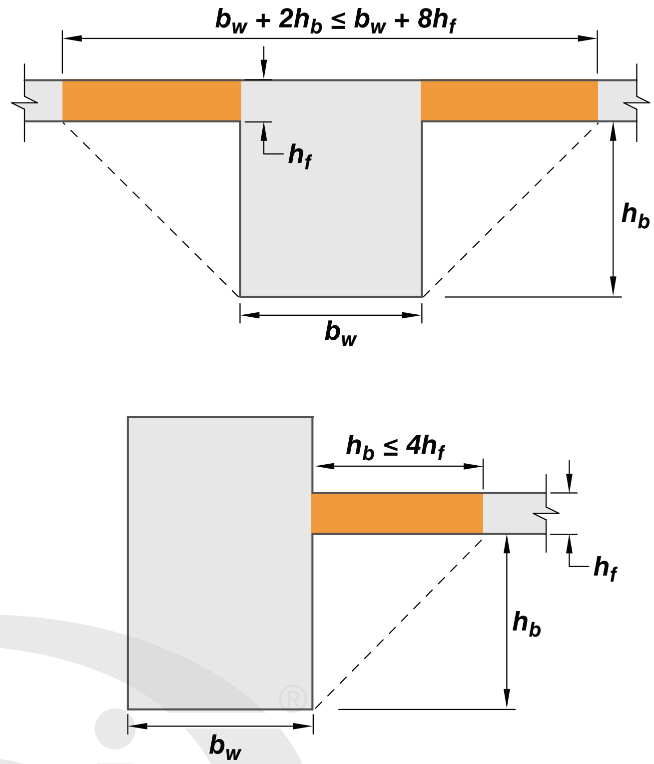

Fig. R8.4.1.8—Examples of the portion of slab to be included with the beam under 8.4.1.8.

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

8.4.2 *Factored moment*

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, capital, or shear cap plus $1.5h$

R8.4.2.2.3 All reinforcement resisting that part of the moment to be transferred to the column by flexure should be placed within b_{slab}.

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of slab or 1.5h of slab plus drop panel if present, on each side of the column, capital, or shear cap.

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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R8.4.2.2.4 Some modification in the 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 the limits given in Table 8.4.2.2.4. Tests (Moehle 1988; ACI PRC-352.1) 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 modification in the distributionof M_{sc} transferred by shear and flexure is possible, but with more severe limitations than for exterior columns with the slab span perpendicular to the edge. For interior columns and for exterior columns with the slab span parallel to the edge, M_{sc} transferred by flexure is permitted to be increased following the limits given in Table 8.4.2.2.4.

If the factored shear for a slab-column connection is larger than the shear limits in Table 8.4.2.2.4, the permitted increase in $\gamma_f M_{sc}$ may lead to brittle behavior of the slab-column connection because all of the reinforcement provided cannot be developed. Modifications for interior slab-column connections in this provision are permitted only where the slab has flexural ductility. So, the reinforcement required to develop $\gamma_f M_{sc}$ within the effective slab width should have 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 connection. If reversal of moments occurs at opposite faces of an interior column, both top and bottom reinforcement should be concentrated within b_{slab}. A ratio of top-to-bottom reinforcement of approximately 2 is appropriate.

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 modified 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	$\leq 0.4\phi v_c$	$\geq \varepsilon_{ty} + 0.008$	$\dfrac{1.25}{1+\left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \leq 1.0$
Interior column	Either direction	$\leq 0.4\phi v_c$	$\geq \varepsilon_{ty} + 0.008$	$\dfrac{1.25}{1+\left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \leq 1.0$

8.4.2.2.5 Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used

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to resist moment on the effective slab width defined in 8.4.2.2.2 and 8.4.2.2.3.

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

8.4.3 *Factored one-way shear*

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

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

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

8.4.4 *Factored two-way shear*

8.4.4.1 *Critical section*

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:

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

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R8.4.4 *Factored two-way shear*

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% of the moment should be considered transferred by flexure across the perimeter of the critical section defined in 22.6.4.1, and 40% by eccentricity of the shear about the centroid of the critical section. For rectangular columns, the portion of moment transferred by flexure increases as the width of the face of the critical section resisting moment increases.

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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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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 shear 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

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

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

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

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 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 resist M_{sc}. Available test data (Hanson and Hanson 1968) indicate that this practice does not increase shear strength but may increase stiffness of the slab-column junction.

Test data (Hawkins 1981) indicate prestressed slab-to-column connections can possess adequate strength to transfer the fractions of slab moment resisted by the column as required by 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 PRC-421.1.

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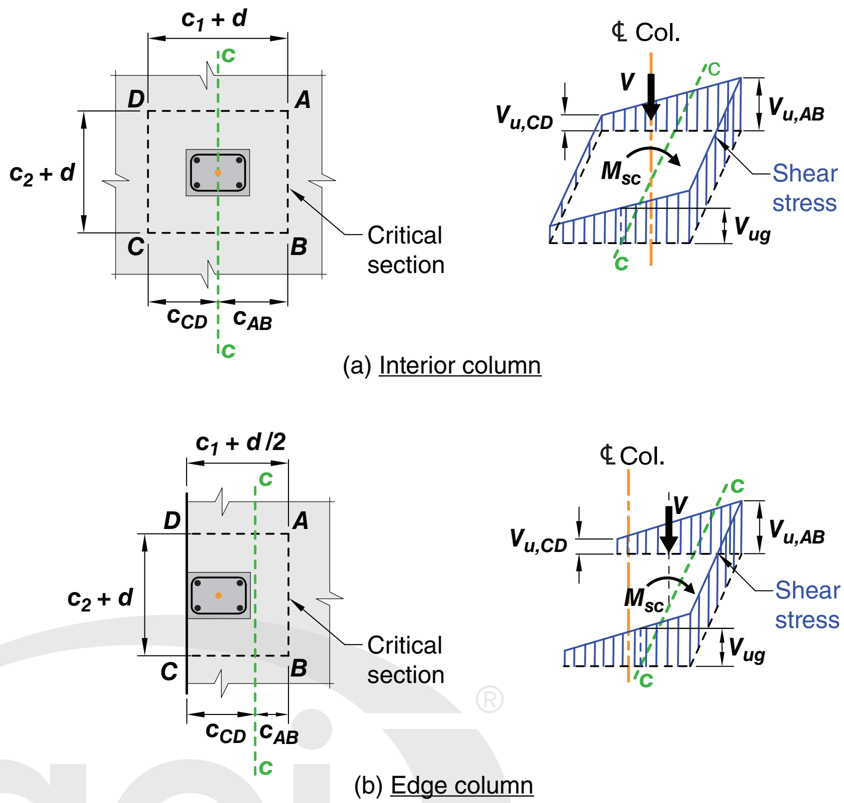

Fig. R8.4.4.2.3—Assumed distribution of shear stress.

8.5—Design strength

8.5.1 *General*

8.5.1.1 For each applicable factored load combination, design strength shall satisfy $\phi S_n \geq U$, including (a) through (d). Interaction between load effects shall be considered.

(a) $\phi M_n \geq M_u$ at all sections along the span in each direction
(b) $\phi M_n \geq \gamma_f M_{sc}$ within b_{slab} as defined in 8.4.2.2.3
(c) $\phi V_n \geq V_u$ at all sections along the span in each direction for one-way shear
(d) $\phi v_n \geq v_u$ at the critical sections defined in 8.4.4.1 for two-way shear

8.5.1.2 ϕ shall be in accordance with 21.2.

8.5.2 *Moment*

8.5.2.1 M_n shall be calculated in accordance with 22.3.

8.5.2.2 In calculating M_n for nonprestressed slabs with a drop panel, the thickness of the drop panel below the slab shall not be assumed to be greater than one-fourth the distance from the edge of drop panel to the face of column or column capital.

R8.5—Design strength

R8.5.1 *General*

R8.5.1.1 Refer to R9.5.1.1.

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8.5.2.3 In calculating M_n for prestressed slabs, external tendons shall be considered as unbonded unless the external tendons are effectively bonded to the slab along its entire length.

8.5.3 *Shear*

8.5.3.1 Design shear strength of slabs in the vicinity of columns, concentrated loads, or reaction areas shall be the more severe of 8.5.3.1.1 and 8.5.3.1.2.

8.5.3.1.1 For one-way shear, where each critical section to be investigated extends in a plane across the entire slab width, V_n shall be calculated in accordance with 22.5.

8.5.3.1.2 For two-way shear, v_n shall be calculated in accordance with 22.6.

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

8.5.4 *Openings in slab systems*

8.5.4.1 Openings of any size shall be permitted in slab systems if shown by analysis that all strength and serviceability requirements, including the limits on deflections, are satisfied.

8.5.4.2 As an alternative to 8.5.4.1, openings shall be permitted in slab systems without beams in accordance with (a) through (d).

(a) Openings of any size shall be permitted in the area common to intersecting middle strips, but the total quantity of reinforcement in the panel shall be at least that required for the panel without the opening.
(b) At two intersecting column strips, not more than one-eighth the width of column strip in either span shall be interrupted by openings. A quantity of reinforcement at least equal to that interrupted by an opening shall be added on the sides of the opening.
(c) At the intersection of one column strip and one middle strip, not more than one-fourth of the reinforcement in either strip shall be interrupted by openings. A quantity of reinforcement at least equal to that interrupted by an opening shall be added on the sides of the opening.
(d) If an opening is located closer than 4*h* from the periphery of a column, concentrated load or reaction area, 22.6.4.3 shall be satisfied.

8.6—Reinforcement limits

8.6.1 *Minimum flexural reinforcement in nonprestressed slabs*

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R8.5.3 *Shear*

R8.5.3.1 Differentiation should be made between a narrow slab acting as a beam, and a slab subject to two-way action where failure may occur by punching along a truncated cone or pyramid around a concentrated load or reaction area.

R8.6—Reinforcement limits

R8.6.1 *Minimum flexural reinforcement in nonprestressed slabs*

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8.6.1.1 A minimum area of flexural reinforcement, $A_{s,min}$ of $0.0018A_g$, or as defined in 8.6.1.2, shall be provided near the tension face of the slab in the direction of the span under consideration.

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R8.6.1.1 The required area of deformed or welded wire reinforcement used as minimum flexural reinforcement is the same as that required for shrinkage and temperature in 24.4.3.2. However, whereas shrinkage and temperature reinforcement is permitted to be distributed between the two faces of the slab as deemed appropriate for specific conditions, minimum flexural reinforcement should be placed as close as practicable to the face of the concrete in tension due to applied loads.

Figure R8.6.1.1 illustrates the arrangement of minimum reinforcement required near the top of a two-way slab supporting uniform gravity load. The bar cutoff points are based on the requirements shown in Fig. 8.7.4.1.3.

To improve crack control and to intercept potential punching shear cracks with tension reinforcement, the licensed design professional should consider specifying continuous reinforcement in each direction near both faces of thick two-way slabs, such as transfer slabs, podium slabs, and mat foundations. Also refer to R8.7.4.1.3.

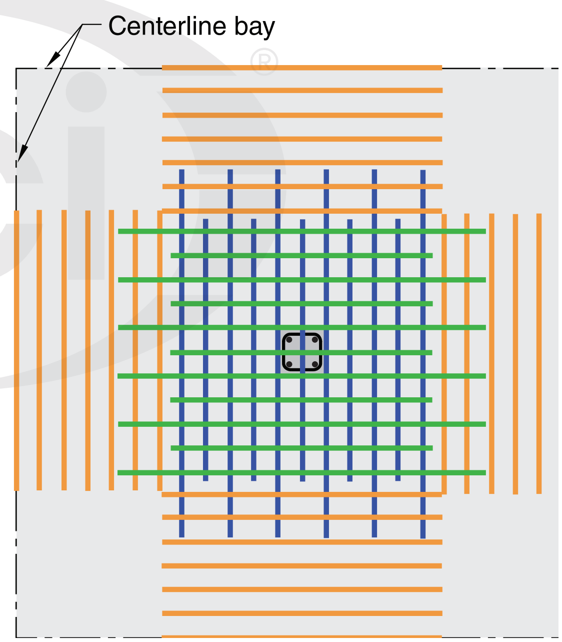

Fig. R8.6.1.1—Arrangement of minimum reinforcement near the top of a two-way slab.

8.6.1.2 If $v_{uv} > \phi 2\sqrt{f_c'}\lambda_s\lambda$ on the critical section for two-way shear surrounding a column, concentrated load, or reaction area, $A_{s,min}$, provided over the width b_{slab}, shall satisfy Eq. (8.6.1.2)

$$A_{s,min} = \frac{5v_{uv}b_{slab}b_o}{\phi\alpha_s f_y} \quad (8.6.1.2)$$

where b_{slab} is the width specified in 8.4.2.2.3, α_s is given in 22.6.5.3, ϕ is the value for shear, and λ_s is given in 22.5.5.1.3.

R8.6.1.2 Tests on interior column-to-slab connections with and without shear reinforcement, where the slabs are lightly reinforced for flexure (Peiris and Ghali 2012; Hawkins and Ospina 2017; Widianto et al. 2009; Muttoni 2008; Dam et al. 2017), have shown that yielding of the slab flexural tension reinforcement in the vicinity of the column or loaded area leads to increased local rotations and opening of any inclined crack existing within the slab. In such cases, sliding along the inclined crack can cause a flexure-driven punching failure. $A_{s,min}$ is intended to prevent a brittle punching failure if the maximum factored two-way shear stress is less than the stress corresponding to the design strength calculated

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by the two-way shear equations of Table 22.6.5.2 for slabs without shear reinforcement and less than $v_c + v_s$ with v_c calculated in accordance with Table 22.6.6.1 for slabs with shear reinforcement.

To derive Eq. (8.6.1.2), the shear force associated with local yielding around the column, taken as $8A_{s,min}f_yd/b_{slab}$ for an interior column connection (Hawkins and Ospina 2017) and generalized as $(\alpha_s/5)A_{s,min}f_yd/b_{slab}$ to account for edge and corner conditions, was set equal to the factored shear on the slab critical section. $A_{s,min}$ also needs to be provided at the periphery of drop panels and shear caps.

Commentary on size effect factor is provided in R22.5.5.1 and R22.6.5.2.

8.6.2 *Minimum flexural reinforcement in prestressed slabs*

8.6.2.1 For prestressed slabs, the effective prestress force $A_{ps}f_{se}$ shall provide a minimum average compressive stress of 125 psi on the slab section tributary to the tendon or tendon group. For slabs with varying cross section along the slab span, either parallel or perpendicular to the tendon or tendon group, the minimum average effective prestress of 125 psi is required at every cross section tributary to the tendon or tendon group along the span.

8.6.2.2 For slabs with bonded prestressed reinforcement, total quantity of A_s and A_{ps} shall be adequate to develop a factored load at least 1.2 times the cracking load calculated on the basis of f_r defined in 19.2.3.

8.6.2.2.1 For slabs with both flexural and shear design strength at least twice the required strength, 8.6.2.2 need not be satisfied.

8.6.2.3 For prestressed slabs, a minimum area of bonded deformed longitudinal reinforcement, $A_{s,min}$, shall be provided in the precompressed tension zone in the direction of the span under consideration in accordance with Table 8.6.2.3.

R8.6.2 *Minimum flexural reinforcement in prestressed slabs*

R8.6.2.1 The minimum average effective prestress of 125 psi was used in two-way test panels in the early 1970s to address punching shear concerns of lightly reinforced slabs. For this reason, the minimum effective prestress is required to be provided at every cross section.

If the slab thickness varies along the span of a slab or perpendicular to the span of a slab, resulting in a varying slab cross section, the 125 psi minimum effective prestress and maximum tendon spacing is required at every cross section tributary to the tendon or group of tendons along the span, considering both the thinner and thicker slab sections. This may result in higher than the minimum f_{pc} in thinner cross sections, and tendons spaced at less than the maximum in thicker cross sections along a span with varying thickness, due to the practical aspects of tendon placement in the field.

R8.6.2.2 Minimum flexural reinforcement is required for reasons similar to those discussed in R9.6.1.1 for nonprestressed beams.

R8.6.2.3 Some bonded reinforcement is required by the Code in prestressed slabs to limit crack width and spacing at service-level load when concrete tensile stresses exceed the modulus of rupture and, for slabs with unbonded tendons, to ensure flexural behavior at nominal strength, rather than tied-arch behavior. Providing the minimum bonded reinforcement as stipulated in this provision helps to ensure the desired behavior.

The minimum amount of bonded reinforcement in two-way flat slab systems is based on reports by Joint ACI-ASCE Committee 423 (1958) and ACI PRC-423.3. Limited research available for two-way flat slabs with drop panels (Odello and Mehta 1967) indicates that behavior of these particular systems is similar to the behavior of flat plates.

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Table 8.6.2.3—Minimum bonded deformed longitudinal reinforcement $A_{s,min}$ in two-way slabs with bonded or unbonded tendons

Region	Calculated ft afterall losses, psi	$A_{s,min}$, in.^{2}	
Positive moment	$f_t \leq 2\sqrt{f_c'}$	Not required	(a)
	$2\sqrt{f_c'} < f_t \leq 6\sqrt{f_c'}$	$\dfrac{N_c}{0.5f_y}$	(b)[1],[2]
Negative moment at columns	$f_t \leq 6\sqrt{f_c'}$	$0.00075A_{cf}$	(c)[2]

[1]The value of f_y shall not exceed 60,000 psi.
[2]For slabs with bonded tendons, it shall be permitted to reduce $A_{s,min}$ by the area of the bonded prestressed reinforcement located within the area used to determine N_c for positive moment, or within the width of slab defined in 8.7.5.3(a) for negative moment.

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For usual loads and span lengths, flat plate tests (Joint ACI-ASCE Committee 423 (1958)) indicate satisfactory performance without bonded reinforcement in positive moment regions where $f_t \leq 2\sqrt{f_c'}$. In positive moment regions where $2\sqrt{f_c'} \leq f_t \leq 6\sqrt{f_c'}$, a minimum bonded reinforcement area proportioned to resist N_c according to Eq. (8.6.2.3(b)) is required. The tensile force N_c is calculated at service load on the basis of an uncracked, homogeneous section.

Research on unbonded post-tensioned two-way flat slab systems (Joint ACI-ASCE Committee 423 [1958], [1974]; ACI PRC-423.3; Odello and Mehta [1967]) shows that bonded reinforcement in negative moment regions, proportioned on the basis of 0.075 percent of the cross-sectional area of the slab-beam strip reduces crack width and spacing. The same area of bonded reinforcement is required in slabs with either bonded or unbonded tendons. The minimum bonded reinforcement area required by Eq. (8.6.2.3(c)) is a minimum area independent of grade of reinforcement or design yield strength. To account for different adjacent tributary spans, this equation is given on the basis of slab-beam strips as defined in 2.3. For rectangular slab panels, this equation is conservatively based on the greater of the cross-sectional areas of the two intersecting slab-beam strips at the column. This ensures that the minimum percentage of reinforcement recommended by research is provided in both directions. Concentration of this reinforcement in the top of the slab directly over and immediately adjacent to the column is important. Research also shows that where low tensile stresses occur at service loads, satisfactory behavior has been achieved at factored loads without bonded reinforcement. However, the Code requires minimum bonded reinforcement regardless of service load stress levels to help ensure flexural continuity and ductility, and to limit crack widths and spacing due to overload, temperature, or shrinkage. Research on post-tensioned flat plate-to-column connections is reported in Smith and Burns (1974), Burns and Hemakom (1977), Hawkins (1981), PTI TAB.1, and Foutch et al. (1990).

Unbonded post-tensioned members do not inherently provide large capacity for energy dissipation under severe earthquake loadings because the member response is primarily elastic. For this reason, unbonded post-tensioned structural members reinforced in accordance with the provisions of this section should be assumed to resist only vertical loads and to act as horizontal diaphragms between energy-dissipating elements under earthquake loadings of the magnitude defined in 18.2.1.

8.7—Reinforcement detailing

8.7.1 *General*

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

8.7.1.2 Development lengths of deformed and prestressed reinforcement shall be in accordance with 25.4.

R8.7—Reinforcement detailing

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8.7.1.3 Splice lengths of deformed reinforcement shall be in accordance with 25.5.

8.7.1.4 Bundled bars shall be detailed in accordance with 25.6.

8.7.2 *Flexural reinforcement spacing*

8.7.2.1 Minimum spacing s shall be in accordance with 25.2.

8.7.2.2 For nonprestressed solid slabs, maximum spacing s of deformed longitudinal reinforcement shall be the lesser of $2h$ and 18 in. at critical sections, and the lesser of $3h$ and 18 in. at other sections.

8.7.2.3 For prestressed slabs with uniformly distributed loads, maximum spacing s of tendons or groups of tendons in at least one direction shall be the lesser of $8h$ and 5 ft.

8.7.2.4 Concentrated loads and openings shall be considered in determining tendon spacing.

8.7.3 *Corner restraint in slabs*

8.7.3.1 At exterior corners of slabs supported by edge walls or where one or more edge beams have a value of α_f greater than 1.0, reinforcement at top and bottom of slab shall be designed to resist M_u per unit width due to corner effects equal to the maximum positive M_u per unit width in the slab panel.

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R8.7.2 *Flexural reinforcement spacing*

R8.7.2.2 The requirement that center-to-center spacing of longitudinal reinforcement be not more than twice the slab thickness at critical sections applies only to reinforcement in solid slabs, and not to reinforcement in joists or waffle slabs. This limit is to ensure uniform flexural action, control cracking, and provide strength for loads concentrated on small areas of the slab. Refer also to R24.3.

R8.7.2.3 This section provides spacing of tendonsin one direction that will permit use of banded tendon distributions in the perpendicular direction, based on results reported by Burns and Hemakom (1977).

R8.7.3 *Corner restraint in slabs*

R8.7.3.1 Unrestrained corners of two-way slabs tend to lift when loaded. If this lifting tendency is restrained by edge walls, columns, or stiff beams, bending moments will develop in the corner of the slab. This section requires reinforcement, equivalent to that required for positive moment in the primary direction, to resist these moments and control cracking. Refer to Fig. R8.7.3.1.

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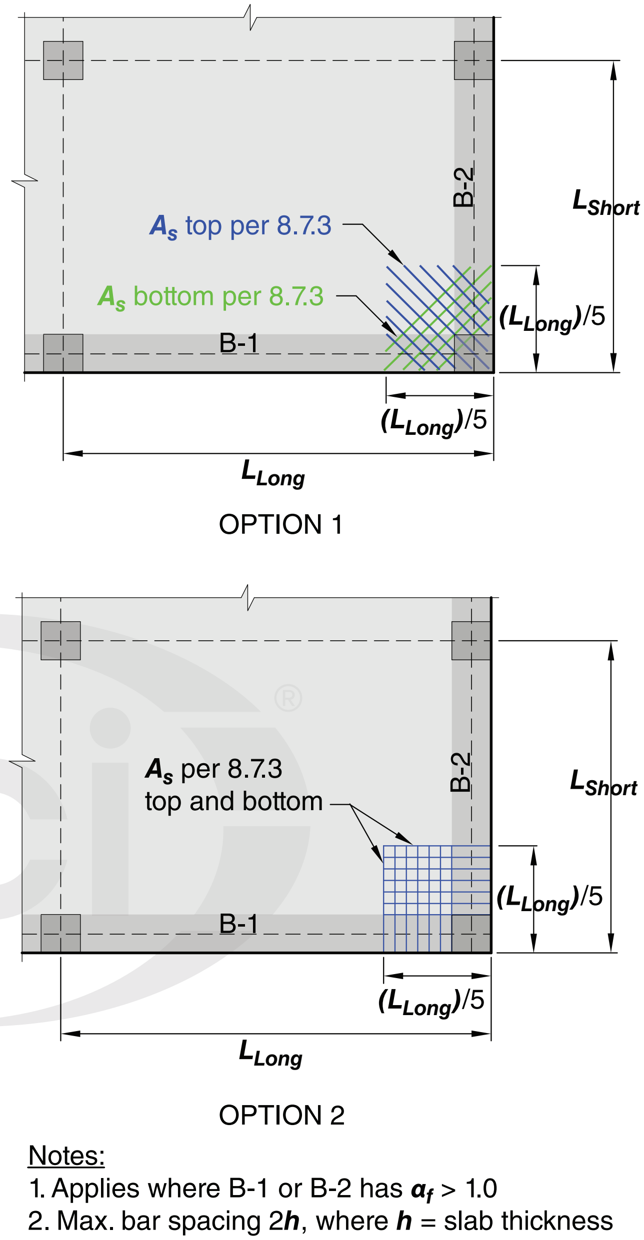

Fig. R8.7.3.1—Slab corner reinforcement.

8.7.3.1.1 Factored moment due to corner effects, M_u, shall be assumed to be about an axis perpendicular to the diagonal from the corner in the top of the slab and about an axis parallel to the diagonal from the corner in the bottom of the slab.

8.7.3.1.2 Reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth the longer span.

8.7.3.1.3 Reinforcement shall be placed parallel to the diagonal in the top of the slab and perpendicular to the diagonal in the bottom of the slab. Alternatively, reinforcement

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shall be placed in two layers parallel to the sides of the slab in both the top and bottom of the slab.

8.7.4 *Flexural reinforcement in nonprestressed slabs*

8.7.4.1 *Termination of reinforcement*

8.7.4.1.1 Where a slab is supported on spandrel beams, columns, or walls, reinforcement perpendicular to a discontinuous edge shall satisfy (a) and (b):

(a) Positive moment reinforcement shall extend to the edge of slab and have embedment, straight or hooked, at least 6 in. into spandrel beams, columns, or walls
(b) Negative moment reinforcement shall be bent, hooked, or otherwise embedded into spandrel beams, columns, or walls, to develop f_y in tension at the face of support

8.7.4.1.2 Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, it shall be permitted to develop the reinforcement within the slab.

8.7.4.1.3 For slabs without beams, reinforcement extensions shall be in accordance with (a) through (c):

(a) Reinforcement lengths shall be at least in accordance with Fig. 8.7.4.1.3, and if slabs act as primary members resisting lateral loads, reinforcement lengths shall be at least those required by analysis.
(b) If adjacent spans are unequal, extensions of negative moment reinforcement beyond the face of support in accordance with Fig. 8.7.4.1.3 shall be based on the longer span.
(c) Bent bars shall be permitted only where the depth-to-span ratio permits use of bends of 45 degrees or less.

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R8.7.4 *Flexural reinforcement in nonprestressed slabs*

R8.7.4.1 *Termination of reinforcement*

R8.7.4.1.1 If spandrel beams are built solidly into walls, the slab edge support approaches complete fixity. Without an integral wall, the slab edge support could approach being a free edge or simply supported, depending on the flexural and torsional rigidity of the spandrel beam. These requirements provide for unknown edge support conditions that might occur in a structure.

R8.7.4.1.2 If spandrel beams are built solidly into walls, the slab edge support approaches complete fixity. Without an integral wall, the slab edge support could approach being a free edge or simply supported, depending on the flexural and torsional rigidity of the spandrel beam. These requirements provide for unknown edge support conditions that might occur in a structure.

R8.7.4.1.3 The minimum lengths and extensions of reinforcement expressed as a fraction of the clear span in Fig. 8.7.4.1.3 were developed for slabs of ordinary proportions supporting gravity loads. These minimum lengths and extensions of bars may not be sufficient to intercept potential punching shear cracks in thick two-way slabs such as transfer slabs, podium slabs, and mat foundations. Therefore, the Code requires extensions for at least half of the column strip top bars to be at least $5d$. For slabs with drop panels, d is the effective depth within the drop panel. In these thick two-way slabs, continuous reinforcement in each direction near both faces is desirable to improve structural integrity, control cracking, and reduce creep deflections. As illustrated in Fig. R8.7.4.1.3, punching shear cracks, which can develop at angles as low as approximately 20 degrees, may not be intercepted by the tension reinforcement in thick slabs if this reinforcement does not extend to at least $5d$ beyond the face of the support. For moments resulting from combined lateral and gravity loadings, these minimum lengths and extensions may not be sufficient.

Bent bars are seldom used and are difficult to place properly. Bent bars, however, are permitted provided they comply with 8.7.4.1.3(c). Guidance on the use of bent bar systems can be found in 13.4.8 of the 1983 Code.

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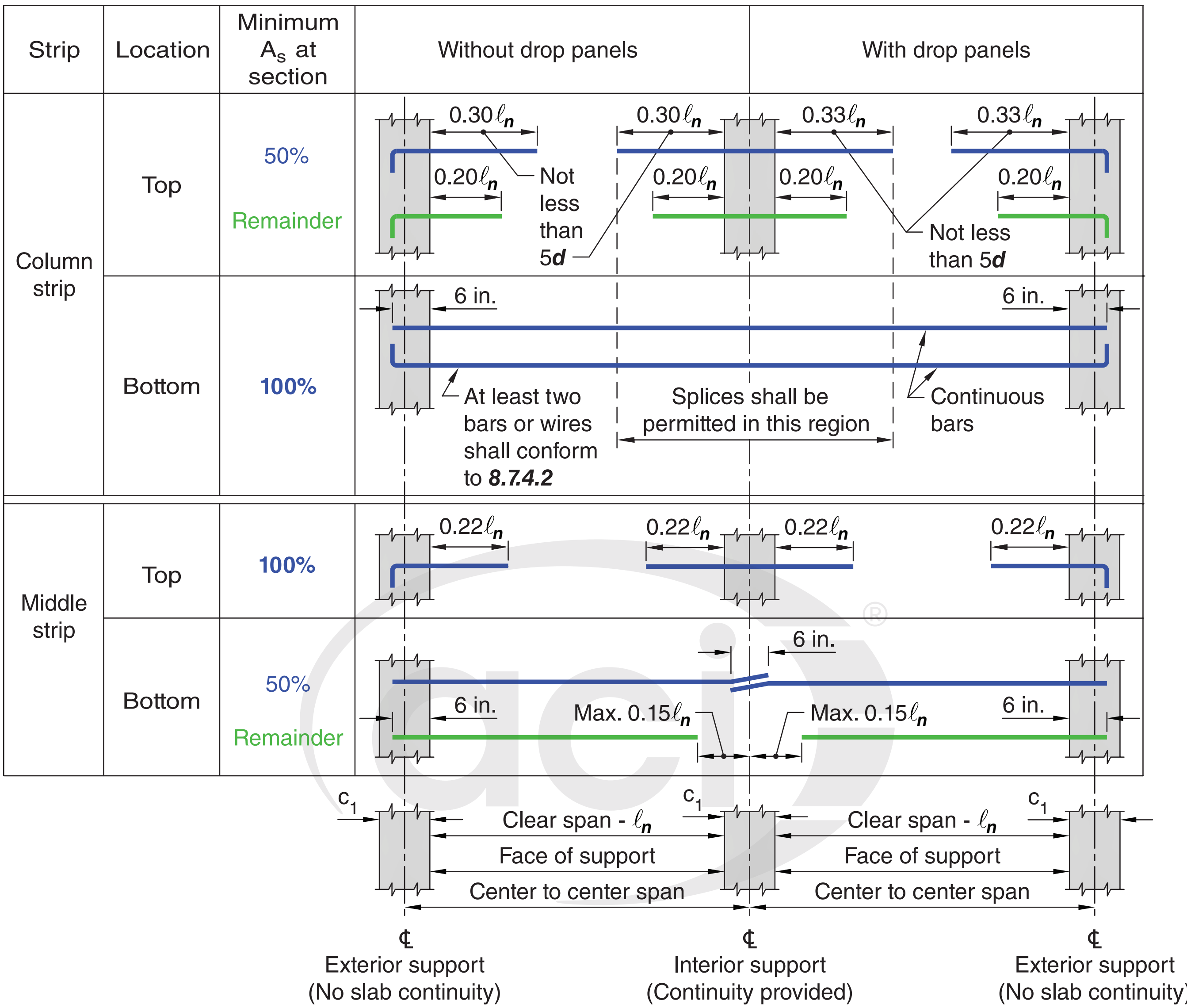

Fig. 8.7.4.1.3—Minimum extensions for deformed reinforcement in two-way slabs without beams.

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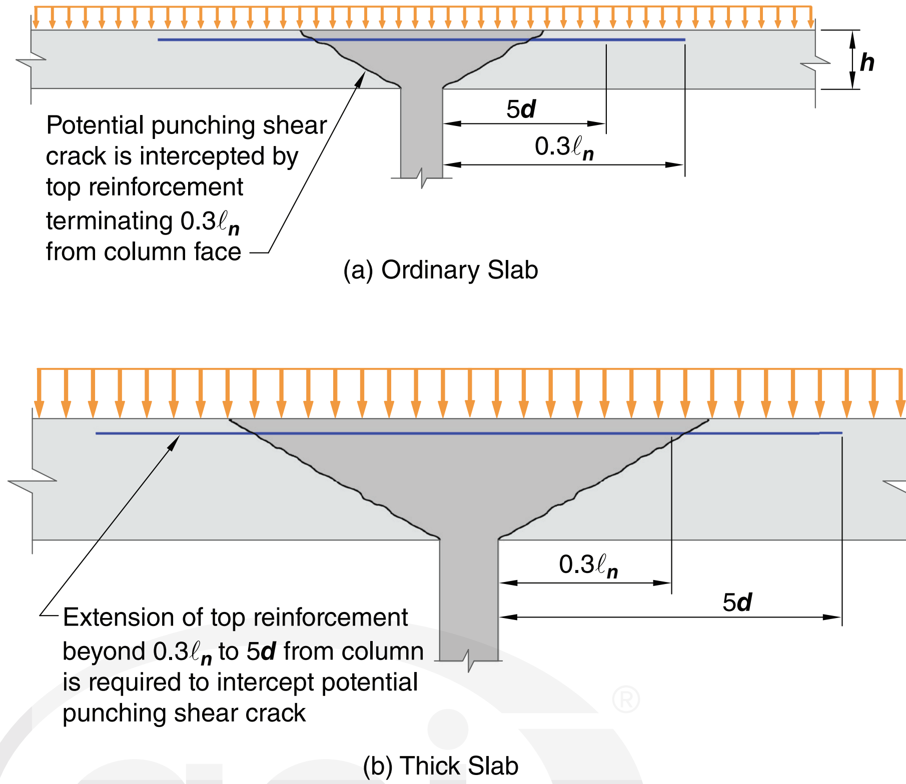

Fig. R8.7.4.1.3—Punching shear cracks in ordinary and thick slabs.

8.7.4.2 *Structural integrity*

8.7.4.2.1 All bottom deformed bars or deformed wires within the column strip, in each direction, shall be continuous or spliced using mechanical or welded splices in accordance with 25.5.7 or Class B tension lap splices in accordance with 25.5.2. Splices shall be located in accordance with Fig. 8.7.4.1.3.

8.7.4.2.2 At least two of the column strip bottom bars or wires in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be developed in tension in accordance with 25.4 by substituting a bar stress of $1.25f_y$ for f_y at exterior supports.

R8.7.4.2 *Structural integrity*

R8.7.4.2.1 The continuous column strip bottom reinforcement provides the slab some residual ability to span to adjacent supports should a single support be damaged. The two continuous column strip bottom bars or wires through the column are termed "integrity reinforcement," and give the slab some residual strength following a punching shear failure at a single support (Mitchell and Cook 1984). Joint ACI-ASCE Committee 352 (ACI PRC-352.1) provides further guidance on design of integrity reinforcement at slab-column connections. Similar provisions for slabs with unbonded tendons are provided in 8.7.5.6.

8.7.5 *Flexural reinforcement in prestressed slabs*

8.7.5.1 External tendons shall be attached to the slab in a manner that maintains the specified eccentricity between the tendons and the concrete centroid through the full range of anticipated member deflections.

R8.7.5 *Flexural reinforcement in prestressed slabs*

8.7.5.2 If bonded deformed longitudinal reinforcement is required to satisfy flexural strength or for tensile stress conditions in accordance with Eq. (8.6.2.3(b)), the detailing requirements of 7.7.3 shall be satisfied.

R8.7.5.2 Bonded reinforcement should be developed for the required strength to resist factored loads. The requirements of 7.7.3 are intended to develop bonded reinforcement for tensile or compressive forces resulting from flexure under factored loads in accordance with 22.3.2, or by tensile stresses at service load in accordance with Eq. (8.6.2.3(b)).

8.7.5.3 Bonded longitudinal reinforcement required by Eq. (8.6.2.3(c)) shall be placed in the top of the slab, and shall be in accordance with (a) through (c):

(a) Reinforcement shall be distributed between lines that are 1.5h outside opposite faces of the column support.
(b) At least four deformed bars, deformed wires, or bonded strands shall be provided in each direction.
(c) Maximum spacing s between bonded longitudinal reinforcement shall not exceed 12 in.

8.7.5.4 *Termination of prestressed reinforcement*

8.7.5.4.1 Post-tensioned anchorage zones shall be designed and detailed in accordance with 25.9.

8.7.5.4.2 Post-tensioning anchorages and couplers shall be designed and detailed in accordance with 25.8.

8.7.5.5 *Termination of deformed reinforcement in slabs with unbonded tendons*

R8.7.5.5 *Termination of deformed reinforcement in slabs with unbonded tendons*

8.7.5.5.1 Length of deformed reinforcement required by 8.6.2.3 shall be in accordance with (a) and (b):

(a) In positive moment areas, length of reinforcement shall be at least $\ell_n/3$ and be centered in those areas
(b) In negative moment areas, reinforcement shall extend at least $\ell_n/6$ on each side of the face of support

R8.7.5.5.1 The minimum lengths apply to bonded reinforcement required by 8.6.2.3, but not required for flexural strength in accordance with 22.3.2. Research (Odello and Mehta 1967) on continuous spans shows that these minimum lengths provide adequate behavior under service load and factored load conditions.

8.7.5.6 *Structural integrity*

R8.7.5.6 *Structural integrity*

8.7.5.6.1 Except as permitted in 8.7.5.6.3, at least two tendons with 1/2 in. diameter or larger strand shall be placed in each direction at columns in accordance with (a) or (b):

(a) Tendons shall pass through the region bounded by the longitudinal reinforcement of the column.
(b) Tendons shall be anchored within the region bounded by the longitudinal reinforcement of the column, and the anchorage shall be located beyond the column centroid and away from the anchored span.

R8.7.5.6.1 Prestressing tendons that pass through the slab-column joint at any location over the depth of the slab are assumed to suspend the slab following a punching shear failure, provided the tendons are continuous through or anchored within the region bounded by the longitudinal reinforcement of the column and are prevented from bursting through the top surface of the slab (ACI PRC-352.1).

8.7.5.6.2 Outside of the column and shear cap faces, the two structural integrity tendons required by 8.7.5.6.1 shall pass under any orthogonal tendons in adjacent spans.

R8.7.5.6.2 Outside of column and shear cap faces, structural integrity tendons should pass below orthogonal tendons from adjacent spans so that vertical movements of the integrity tendons are restrained by the orthogonal tendons. Where tendons are distributed in one direction and banded in the orthogonal direction, this requirement can be satisfied by first

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8.7.5.6.3 Slabs with tendons not satisfying 8.7.5.6.1 shall be permitted if bonded bottom deformed reinforcement is provided in each direction in accordance with 8.7.5.6.3.1 through 8.7.5.6.3.3.

8.7.5.6.3.1 Minimum bottom deformed reinforcement A_s in each direction shall be the larger of (a) and (b). The value of f_y shall be limited to a maximum of 80,000 psi:

(a) $$A_s = \frac{4.5\sqrt{f_c'}c_2 d}{f_y} \quad (8.7.5.6.3.1a)$$

(b) $$A_s = \frac{300 c_2 d}{f_y} \quad (8.7.5.6.3.1b)$$

where c_2 is measured at the column faces through which the reinforcement passes.

8.7.5.6.3.2 Bottom deformed reinforcement calculated in 8.7.5.6.3.1 shall pass within the region bounded by the longitudinal reinforcement of the column and shall be developed in tension in accordance with 25.4 by substituting a bar stress of $1.25f_y$ for f_y at exterior supports.

8.7.5.6.3.3 Bottom deformed reinforcement shall be developed in tension in accordance with 25.4 by substituting a bar stress of $1.25f_y$ for f_y beyond the column or shear cap face.

8.7.6 *Shear reinforcement – stirrups*

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placing the integrity tendons for the distributed tendon direction and then placing the banded tendons. Where tendons are distributed in both directions, weaving of tendons is necessary and use of 8.7.5.6.3 may be an easier approach.

R8.7.5.6.3 In some prestressed slabs, tendon layout constraints make it difficult to provide the structural integrity tendons required by 8.7.5.6.1. In such situations, the structural integrity tendons can be replaced by deformed bar bottom reinforcement. The equations given in 8.7.5.6.3.1 are the equivalent of 1.5 times the minimum flexural reinforcement area required for nonprestressed beams (9.6.1.2).

R8.7.6 *Shear reinforcement – stirrups*

Research (Hawkins 1974; Broms 1990; Yamada et al. 1991; Hawkins et al. 1975; ACI PRC-421.1) has shown that shear reinforcement consisting of properly anchored bars or wires and single- or multiple-leg stirrups, or closed stirrups, can increase punching shear resistance of slabs. Spacing limits given in 8.7.6.3 correspond to slab shear reinforcement details that have been shown to be effective. Anchorage requirements for stirrup-type shear reinforcement, which should also be applied for bars or wires used as slab shear reinforcement, are given in 25.7.1. It is essential that this shear reinforcement engage longitudinal reinforcement at both the top and bottom of the slab, as shown for typical details in Fig. R8.7.6(a) to (d). Anchorage of shear reinforcement with hooks according to 25.7.1 is difficult in slabs thinner than 10 in. Shear reinforcement consisting of headed studs in accordance with 8.7.7 has been used successfully (ACI PRC-421.1). Heads conforming to 20.2.1.6 are

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permitted in 25.7.1 as an alternative to hooks at one or both ends for anchorage of shear reinforcement in slabs.

In a slab-column connection for which moment transfer is negligible, the shear reinforcement should be symmetrical about the centroid of the critical section (Fig. R8.7.6(e)). Spacing limits defined in 8.7.6.3 are also shown in Fig. R8.7.6(e) and (f).

At exterior connections or for interior connections where moment transfer is significant, closed stirrups are recommended in a pattern as symmetrical as possible. Although the average shear stresses on faces *AD* and *BC* of the exterior connection in Fig. R8.7.6(f) are lower than on face *AB*, the closed stirrups extending from faces *AD* and *BC* provide some torsional strength along the edge of the slab.

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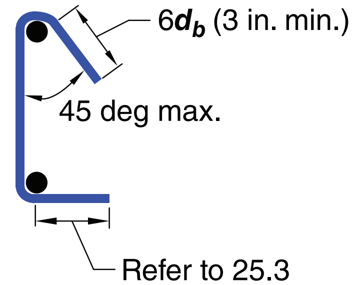
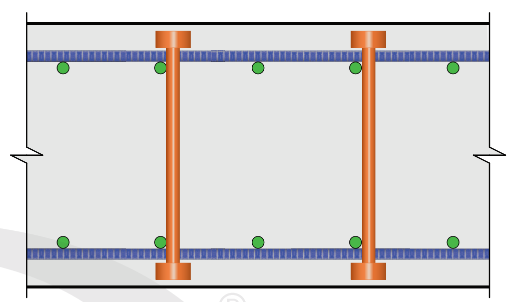
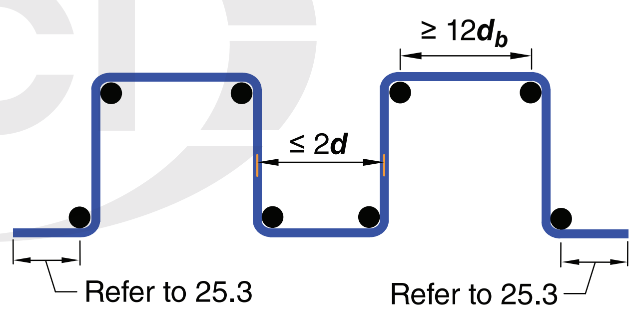


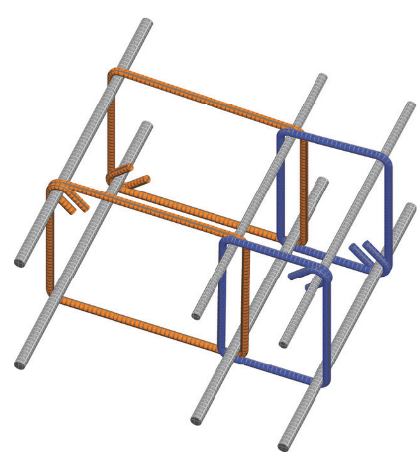
(a) single-leg stirrup or bar



(b) single-leg stirrup or bar with heads



(c) multiple-leg stirrup or bar



(d) closed stirrup

Fig. R8.7.6(a)-(d)—Single- or multiple-leg stirrup-type slab shear reinforcement.

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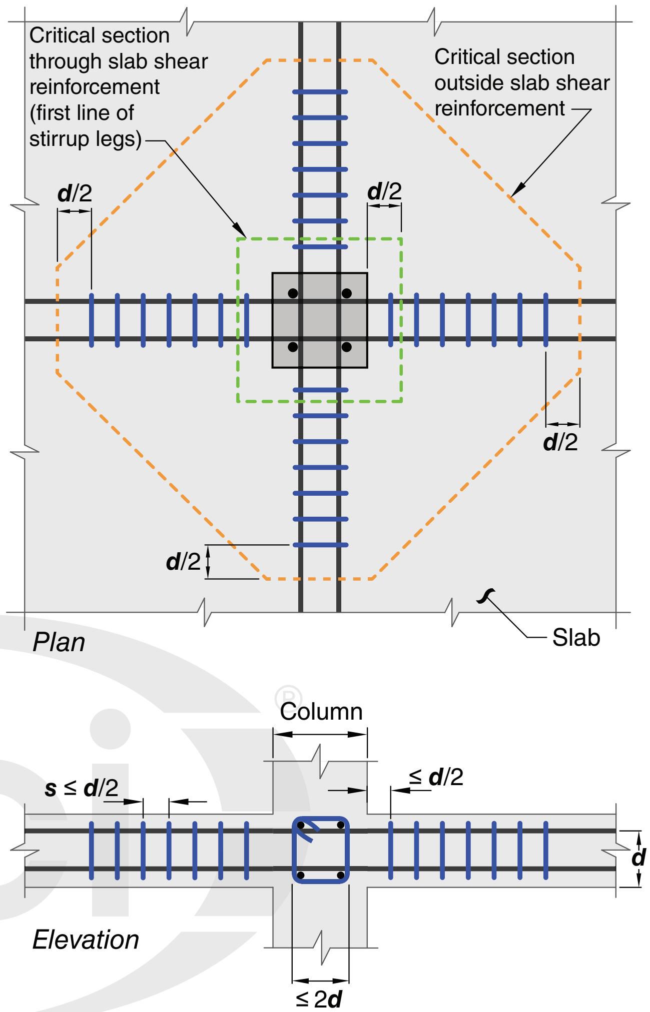

Fig. R8.7.6(e)—Arrangement of stirrup shear reinforcement, interior connection.

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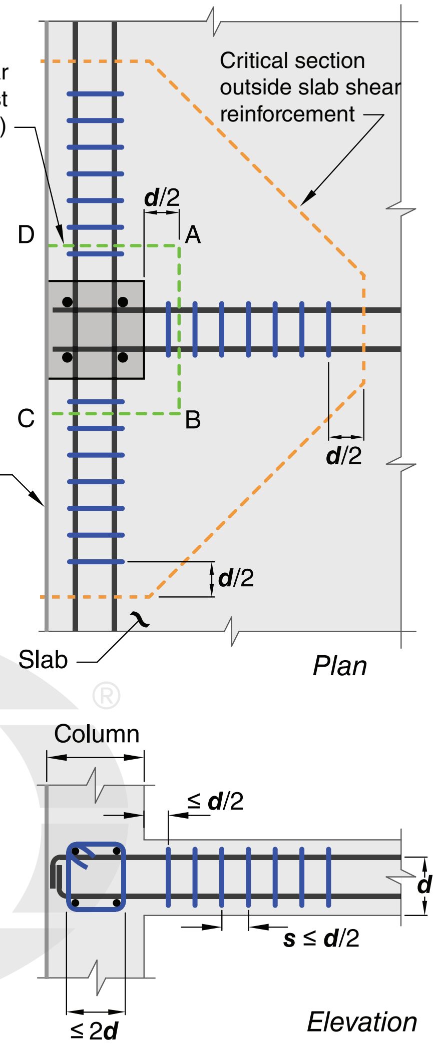

Fig. R8.7.6(f)—Arrangement of stirrup shear reinforcement, edge connection.

8.7.6.1 Single-leg, simple-U, multiple-U, and closed stirrups shall be permitted as shear reinforcement.

8.7.6.2 Stirrup anchorage and geometry shall be in accordance with 25.7.1.

8.7.6.3 If stirrups are provided, location and spacing shall be in accordance with Table 8.7.6.3.

Table 8.7.6.3—First stirrup location and spacing limits

Direction of measurement	Description of measurement	Maximum distance or spacing, in.
Perpendicular to column face	Distance from column face to first stirrup	$d/2$
	Spacing between stirrups	$d/2$
Parallel to column face	Spacing between vertical legs of stirrups	$2d$

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8.7.7 *Shear reinforcement – headed studs*

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R8.7.7 *Shear reinforcement – headed studs*

Using headed stud assemblies as shear reinforcement in slabs requires specifying the stud shank diameter, spacing of the studs, and height of the assemblies for the particular applications.

Tests (ACI PRC-421.1) have shown that vertical studs mechanically anchored close to the top and bottom of slabs are effective in resisting punching shear. The specified height to achieve this objective while providing a reasonable tolerance in specifying the height, as shown in Fig. R20.5.1.3.6.

Compared with a leg of a stirrup having bends at the ends, in a thin slab a stud head exhibits smaller slip and results in smaller shear crack widths. The improved performance results in increased limits for shear strength and spacing between peripheral lines of headed shear stud reinforcement. Typical arrangements of headed shear stud reinforcement are shown in Fig. R8.7.7. The critical section beyond the shear reinforcement generally has a polygonal shape. Equations for calculating shear stresses on such sections are given in ACI PRC-421.1.

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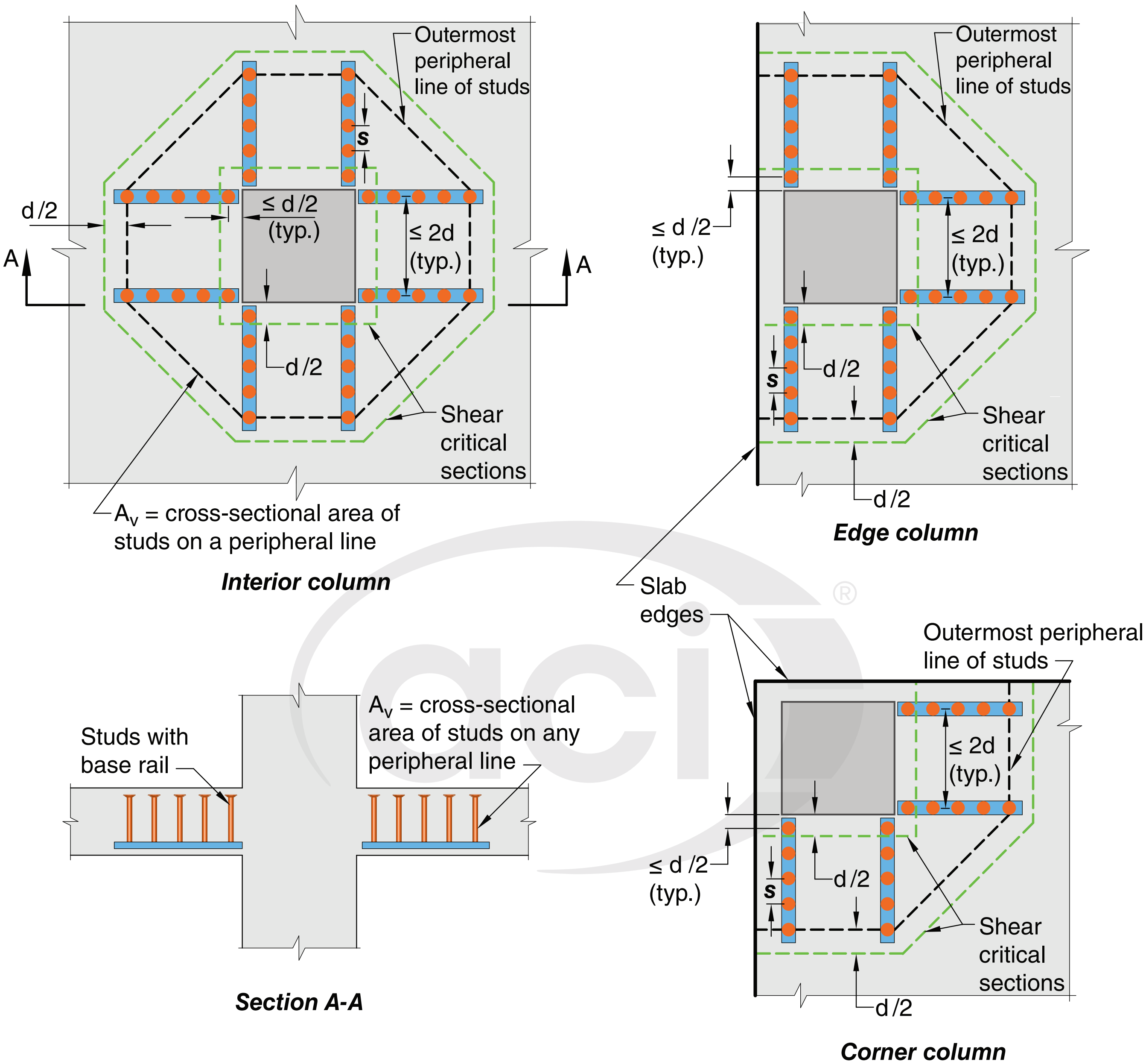

Fig. R8.7.7—Typical arrangements of headed shear stud reinforcement and critical sections.

8.7.7.1 Headed shear stud reinforcement shall be permitted if placed perpendicular to the plane of the slab.

8.7.7.1.1 The overall height of the shear stud assembly shall be at least the thickness of the slab minus the sum of (a) through (c):

(a) Concrete cover on the top flexural reinforcement
(b) Concrete cover on the base rail
(c) One-half the bar diameter of the flexural tension reinforcement

8.7.7.1.2 Headed shear stud reinforcement location and spacing shall be in accordance with Table 8.7.7.1.2.

R8.7.7.1.2 The specified spacings between peripheral lines of shear reinforcement are justified by experiments

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Table 8.7.7.1.2—Shear stud location and spacing limits

Direction of measurement	Description of measurement	Condition		Maximum distance or spacing, in.
Perpendicular to column face	Distance from column face to first peripheral line of shear studs	All		$d/2$
	Constant spacing between peripheral lines of shear studs	Nonprestressed slab with	$v_u \leq \phi 6\sqrt{f_c'}$	$3d/4$
		Nonprestressed slab with	$v_u > \phi 6\sqrt{f_c'}$	$d/2$
		Prestressed slabs conforming to 22.6.5.4		$3d/4$
Parallel to column face	Spacing between adjacent shear studs on peripheral line nearest to column face	All		$2d$

8.8—Nonprestressed two-way joist systems

8.8.1 *General*

8.8.1.1 Nonprestressed two-way joist construction consists of a monolithic combination of regularly spaced ribs and a top slab designed to span in two orthogonal directions.

8.8.1.2 Width of ribs shall be at least 4 in. at any location along the depth.

8.8.1.3 Overall depth of ribs, excluding slab thickness, shall not exceed 3.5 times the minimum width.

8.8.1.4 Clear spacing between ribs shall not exceed 30 in.

8.8.1.5 V_c shall be permitted to be taken as 1.1 times the values calculated in 22.5.

8.8.1.6 For structural integrity, at least one bottom bar in each joist shall be continuous and shall be developed in tension in accordance with 25.4 by substituting a bar stress of $1.25f_y$ for f_y at the face of supports.

8.8.1.7 Reinforcement area perpendicular to the ribs shall satisfy slab moment strength requirements, considering load concentrations, and shall be at least the shrinkage and temperature reinforcement area in accordance with 24.4.

COMMENTARY

(ACI PRC-421.1). Clear spacing between the heads of studs should be adequate to permit placing flexural reinforcement.

R8.8—Nonprestressed two-way joist systems

R8.8.1 *General*

The empirical limits established for nonprestressed reinforced concrete joist floors are based on successful past performance of joist construction using common joist forming systems. For prestressed joist construction, these limits may be used as a guide.

R8.8.1.4 A limit on maximum spacing of ribs is required because of the provisions permitting higher shear strengths and less concrete cover for the reinforcement for these relatively small, repetitive members.

R8.8.1.5 The increase in shear strength is justified on the basis of: 1) satisfactory performance of joist construction designed with higher calculated shear strengths; and 2) potential for redistribution of local overloads to adjacent joists.

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COMMENTARY

8.8.1.8 Two-way joist construction not satisfying the limitations of 8.8.1.1 through 8.8.1.4 shall be designed as slabs and beams.

8.8.2 *Joist systems with structural fillers*

8.8.2.1 If permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to f_c' in the joists are used, 8.8.2.1.1 and 8.8.2.1.2 shall apply.

8.8.2.1.1 Slab thickness over fillers shall be at least the greater of one-twelfth the clear distance between ribs and 1.5 in.

8.8.2.1.2 For calculation of shear and negative moment strength, it shall be permitted to include the vertical shells of fillers in contact with the ribs. Other portions of fillers shall not be included in strength calculations.

8.8.3 *Joist systems with other fillers*

8.8.3.1 If fillers not complying with 8.8.2.1 or removable forms are used, slab thickness shall be at least the greater of one-twelfth the clear distance between ribs and 2 in.

8.9—Lift-slab construction

8.9.1 In slabs constructed with lift-slab methods where it is impractical to pass the tendons required by 8.7.5.6.1 or the bottom bars required by 8.7.4.2 or 8.7.5.6.3 through the column, at least two post-tensioned tendons or two bonded bottom bars or wires in each direction shall pass through the lifting collar as close to the column as practicable, and be continuous or spliced using mechanical or welded splices in accordance with 25.5.7 or Class B tension lap splices in accordance with 25.5.2. At exterior columns, the reinforcement shall be anchored at the lifting collar.

Notes