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Guide for Shear Reinforcement for Slabs

Reported by Joint ACI-ASCE Committee 421

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Guide for Shear Reinforcement for Slabs

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Guide for Shear Reinforcement for Slabs

Reported by Joint ACI-ASCE Committee 421

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Tests have established that punching shear in slabs can be effectively resisted by reinforcement consisting of vertical rods mechanically anchored at top and bottom of slabs. ACI 318 sets out the principles of design for slab shear reinforcement and makes specific reference to stirrups and headed studs. This guide reviews available types and makes recommendations for their design. The application of these recommendations is illustrated through numerical examples.

Keywords: column-slab connection; concrete flat plate; headed shear studs; moment transfer; prestressed concrete; punching shear; shear heads; shear stresses; slabs; two-way slabs.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

In flat-plate floors, slab-column connections are subjected to high shear stresses produced by the transfer of the internal forces between the columns and the slabs. ACI 318-14 Section 8.7.6.1 allows the use of shear reinforcement for slabs and footings in the form of bars, as in the vertical legs of stirrups. ACI 318 emphasizes the importance of anchorage details and accurate placement of the shear reinforcement, especially in thin slabs. ACI 318-14 Section 8.7.7.1 permits headed shear stud reinforcement conforming to ASTM A1044/A1044M. A general procedure for evaluation of the punching shear strength of slab-column connections is given in ACI 318-14 Section 8.5.3.

Shear reinforcement consisting of vertical rods (studs) or the equivalent, mechanically anchored at each end, can be used. In this guide, all types of mechanically anchored shear reinforcement are referred to as shear stud or stud. To be fully effective, the anchorage should be capable of developing the specified yield strength of the studs. The mechanical anchorage can be obtained by heads or strips connected to the studs by welding. The heads can also be formed by forging the stud ends.

1.2—Scope

Recommendations in this guide are for the design of shear reinforcement in slabs. The design is in accordance with ACI 318. Numerical design examples are included.

1.3—Evolution of the practice

Extensive tests (Dilger and Ghali 1981; Andrä 1981a,b; Van der Voet et al. 1982; Mokhtar et al. 1985; Elgabry and Ghali 1987; Mortin and Ghali 1991; Dilger and Shatila

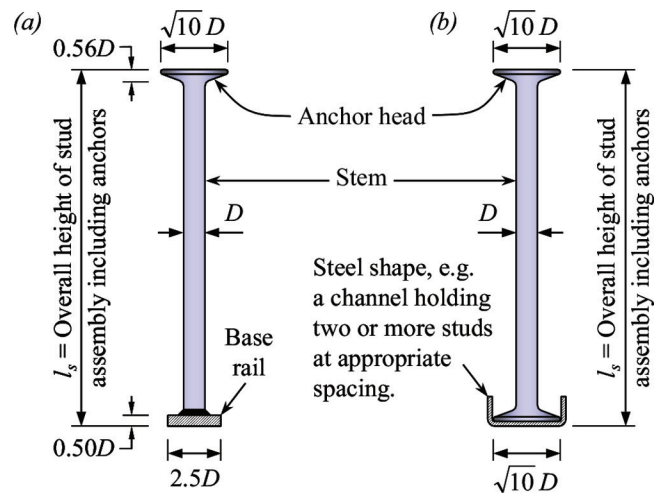


Fig. 1.3a—Stud assemblies conforming to ASTM A1044/A1044M: (a) single-headed studs welded to a base rail; and (b) double-headed studs crimped into a steel channel.

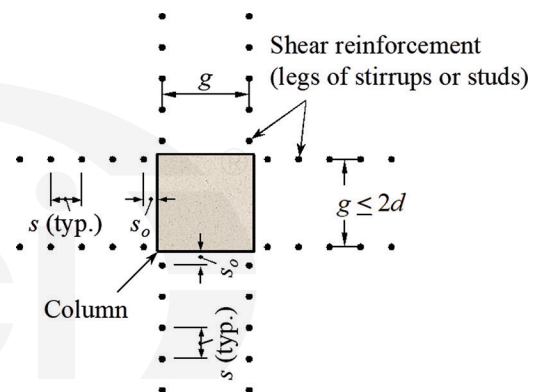


Fig. 1.3b—Top view of flat plate showing arrangement of shear reinforcement in vicinity of interior column.

1989; Cao 1993; Brown and Dilger 1994; Megally 1998; Birkle 2004; Ritchie and Ghali 2005; Gayed and Ghali 2006) have confirmed the effectiveness of mechanically anchored shear reinforcement, such as shown in Fig. 1.3a, in increasing the strength and ductility of slab-column connections subjected to concentric punching or punching combined with moment. Stud assemblies consisting of either a single-head stud attached to a steel base rail by welding (Fig. 1.3a(a)) or double-headed studs mechanically crimped into a nonstructural steel channel (Fig. 1.3a(b)) are specified in ASTM A1044/A1044M. Figure 1.3b is a top view of a slab that shows a typical arrangement of shear reinforcement (stirrup legs or studs) in the vicinity of an interior column. ACI 318 requires that the spacing g between adjacent stirrup legs or studs, measured on the first peripheral line of shear reinforcement, be equal to or less than $2d$. Recommendations for the distances s_o and s are given in Chapter 4.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A_c = area of concrete of assumed critical section, in² (mm²)

A_v	= cross-sectional area of shear reinforcement on one peripheral line parallel to perimeter of column section, in. ² (mm ²)	s	= spacing between peripheral lines of shear reinforcement, in. (mm)
b_o	= length of perimeter of critical section, in. (mm)	s_o	= spacing between first peripheral line of shear reinforcement and column face, in. (mm)
c_b, c_t	= clear concrete cover of reinforcement to bottom and top slab surfaces, respectively, in. (mm)	V_p	= vertical component of all effective prestress forces crossing the critical section, lb (N)
c_x, c_y	= size of rectangular column measured in two orthogonal span directions, x and y , respectively, in. (mm)	V_u	= factored shear force, lb (N)
D	= stud diameter, in. (mm)	v_c	= nominal shear strength provided by concrete in presence of shear reinforcement, psi (MPa)
d	= effective depth of slab; average of distances from extreme compression fiber to centroids of tension reinforcements running in two orthogonal directions, in. (mm)	v_n	= nominal shear strength at critical section, psi (MPa)
d_b	= nominal diameter of flexural reinforcing bars, in. (mm)	v_s	= nominal shear strength provided by shear reinforcement, psi (MPa)
f'_c	= specified compressive strength of concrete, psi (MPa)	v_u	= maximum shear stress due to factored forces, psi (MPa)
f_{ct}	= average splitting tensile strength of lightweight aggregate concrete, psi (MPa)	w_{sd}	= super-imposed dead load per unit area, lb/ft ² (kN/m ²)
f_{pc}	= average value of compressive stress in concrete in two directions (after allowance for all prestress losses) at centroid of cross section, psi (MPa)	x, y	= coordinates of point on perimeter of shear critical section with respect to centroidal principal axes x and y , in. (mm)
f_{yt}	= specified yield strength of shear reinforcement, psi (MPa)	$\bar{x}, \bar{y}$	= coordinates of point on perimeter of shear critical section with respect to centroidal nonprincipal axes $\bar{x}$ and $\bar{y}$, in. (mm)
g	= distance between adjacent stirrup legs or studs, measured in a direction parallel to a column face, in. (mm)	α_s	= dimensionless coefficient equal to 40, 30, and 20 for interior, edge, and corner columns, respectively
h	= overall thickness of slab, in. (mm)	β	= ratio of long side to short side of column cross section
J_c	= property of assumed critical section, defined by ACI 318 as analogous to polar moment of inertia, in. ⁴ (mm ⁴)	γ_{conc}	= density of concrete, lb/ft ³ (kN/m ³)
J_{cx}, J_{cy}	= property of assumed critical section of any shape, equal to d multiplied by second moment of perimeter about x or y axis, respectively (Appendix B), in. ⁴ (mm ⁴)	γ_{vx}, γ_{vy}	= factor used to determine moment about the axes x and y between slab and column that is transferred by shear stress at assumed critical section
$J_{\bar{x}\bar{y}}$	= d times product of inertia of assumed shear critical section about nonprincipal axes $\bar{x}$ and $\bar{y}$, in. ⁴ (mm ⁴)	λ	= modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normalweight concrete of the same compressive strength
L	= length of segment of assumed critical section, in. (mm)	ϕ	= strength reduction factor = 0.75
l_s	= overall specified height of headed stud assembly including anchors, in. (mm)	κ	= fraction of service dead load to be balanced by effective prestressing
l_x, l_y	= projections of assumed critical section on principal axes x and y , in. (mm)		
l_{x1}, l_{y1}	= length of sides in x - and y -directions of critical section at $d/2$ from column face, in. (mm)		
l_{x2}, l_{y2}	= length of sides in x - and y -directions of critical section at $d/2$ outside outermost legs of shear reinforcement, in. (mm)		
M_{sc-x}			
M_{sc-y}	= factored slab moments resisted by supporting column about centroidal principal axes x and y of assumed critical section, lb-in. (N-m)		
$M_{sc-\bar{x}}$			
$M_{sc-\bar{y}}$	= factored slab moments resisted by column about the centroidal nonprincipal $\bar{x}$ or $\bar{y}$ axis, lb-in. (N-m)		
M_{scOx}			
M_{scOy}	= factored slab moment resisted by column about x or y axis through the column's centroid O , lb-in. (N-m)		
n	= number of studs or stirrup legs per line running in x - or y -direction		

2.2—Definitions

Please refer to the latest version of “ACI Concrete Terminology” for a comprehensive list of definitions. Definitions provided herein complement that resource.

headed shear stud reinforcement—reinforcement conforming to ASTM A1044/A1044M and composed of vertical rods anchored mechanically near the bottom and top surfaces of the slab.

shear cap—thickened portion of the slab around the column with plan dimensions not conforming with the ACI 318 requirements for drop panels.

shear critical section—cross section, having depth d and perpendicular to the plane of the slab, where shear stresses should be evaluated; two shear critical sections should be considered: (1) at $d/2$ from column periphery, and (2) at $d/2$ from the outermost peripheral line of shear reinforcement (if provided).

slab moment resisted by supporting column—sum of moments at the ends of the column above and below a slab-column joint.

CHAPTER 3—ROLE OF SHEAR REINFORCEMENT

Shear reinforcement is required to intercept shear cracks and prevent them from widening. The intersection of shear reinforcement and cracks can be anywhere over the height of the shear reinforcement. The strain in the shear reinforcement is highest at that intersection.

Effective anchorage is essential, and its location should be as close as possible to the structural member's outer surfaces. This means that the vertical part of the shear reinforcement should be as tall as possible to avoid the possibility of cracks passing above or below it. When the shear reinforcement is not as tall as possible, it may not intercept all inclined shear cracks. Anchorage of shear reinforcement in slabs is achieved by mechanical ends (heads), bends, and hooks.

CHAPTER 4—PUNCHING SHEAR DESIGN EQUATIONS

4.1—Strength requirement

This chapter presents the design procedure of ACI 318 when stirrups or headed studs are required in the slab in the vicinity of a column transferring moment and shear. The equations of 4.3.2 and 4.3.3 apply when stirrups and headed studs are used, respectively.

Design of critical slab sections perpendicular to the plane of a slab should be based upon

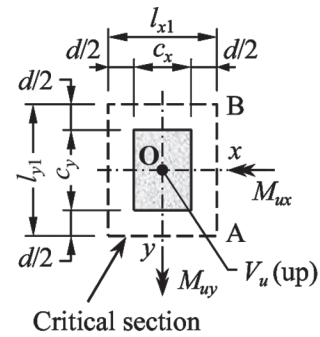
$$v_u \leq \phi v_n \quad (4.1)$$

in which v_u (in psi or MPa) is the shear stress in the critical section caused by the transfer, between the slab and the column, of factored shearing force or factored shearing force combined with moment; v_n is the nominal shear strength (psi or MPa); and ϕ is the strength reduction factor.

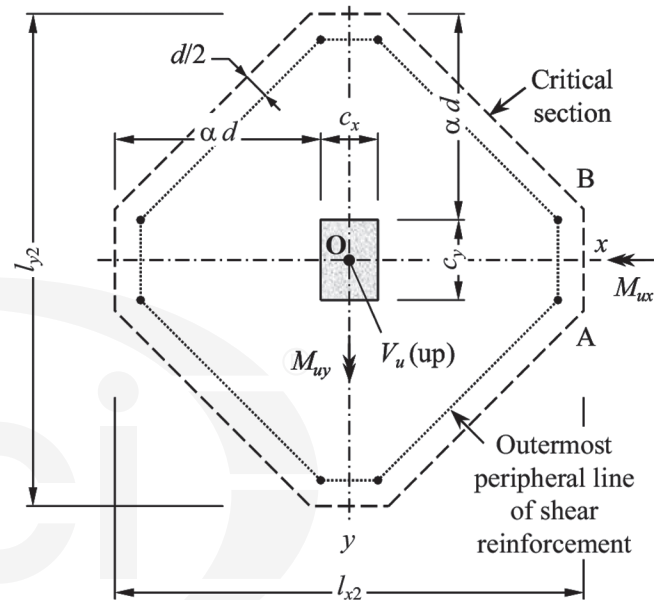
Equation (4.1) should be satisfied at a critical section perpendicular to the plane of the slab at a distance $d/2$ from the column perimeter and located so that its perimeter b_o is minimum (Fig. 4.1(a)). It should also be satisfied at a critical section at $d/2$ from the outermost peripheral line of the shear reinforcement (Fig. 4.1(b)), where d is the average of distances from extreme compression fiber to the centroids of the tension reinforcements running in two orthogonal directions. Figure 4.1(a) indicates the positive directions of the internal force V_u and moments M_{sc-x} , and M_{sc-y} that the column exerts on the slab.

4.2—Calculation of factored shear stress V_u

ACI 318 requires that the shear stress resulting from moment transfer by eccentricity of shear be assumed to vary linearly about the centroid of the shear critical section; the code's equation was developed considering uniaxial moment transfer between the slab and its supporting columns. However, this guide recommends the use of Eq. (4.2a) that accounts for the general case of biaxial moment transfer. The shear stress distribution, calculated by Eq. (4.2a), satisfies the linear variation requirement of the code. The maximum factored shear stress v_u at a critical section produced by the



(a) At $d/2$ from column face



(b) At $d/2$ from outermost peripheral line of shear reinforcement

Fig. 4.1—Critical sections for shear in slab in vicinity of interior column. Positive directions of V_u , M_{sc-x} , and M_{sc-y} are indicated in (a).

combination of factored shear force V_u and moments M_{sc-x} and M_{sc-y} is

$$v_u = \frac{V_u}{A_c} + \frac{\gamma_{vy} M_{sc-y}}{J_{cy}} x + \frac{\gamma_{vx} M_{sc-x}}{J_{cx}} y \quad (4.2a)$$

The coefficients γ_{vx} and γ_{vy} are given by

$$\left. \begin{aligned} \gamma_{vx} &= 1 - \frac{1}{1 + \frac{2}{3} \sqrt{l_{y1}/l_{x1}}} \\ \gamma_{vy} &= 1 - \frac{1}{1 + \frac{2}{3} \sqrt{l_{x1}/l_{y1}}} \end{aligned} \right\} \quad (4.2b)$$

where l_{x1} and l_{y1} are lengths of the sides in the x - and y -directions of a rectangular critical section at $d/2$ from the column face (Fig. 4.1(a)). For a shear critical section outside the shear-

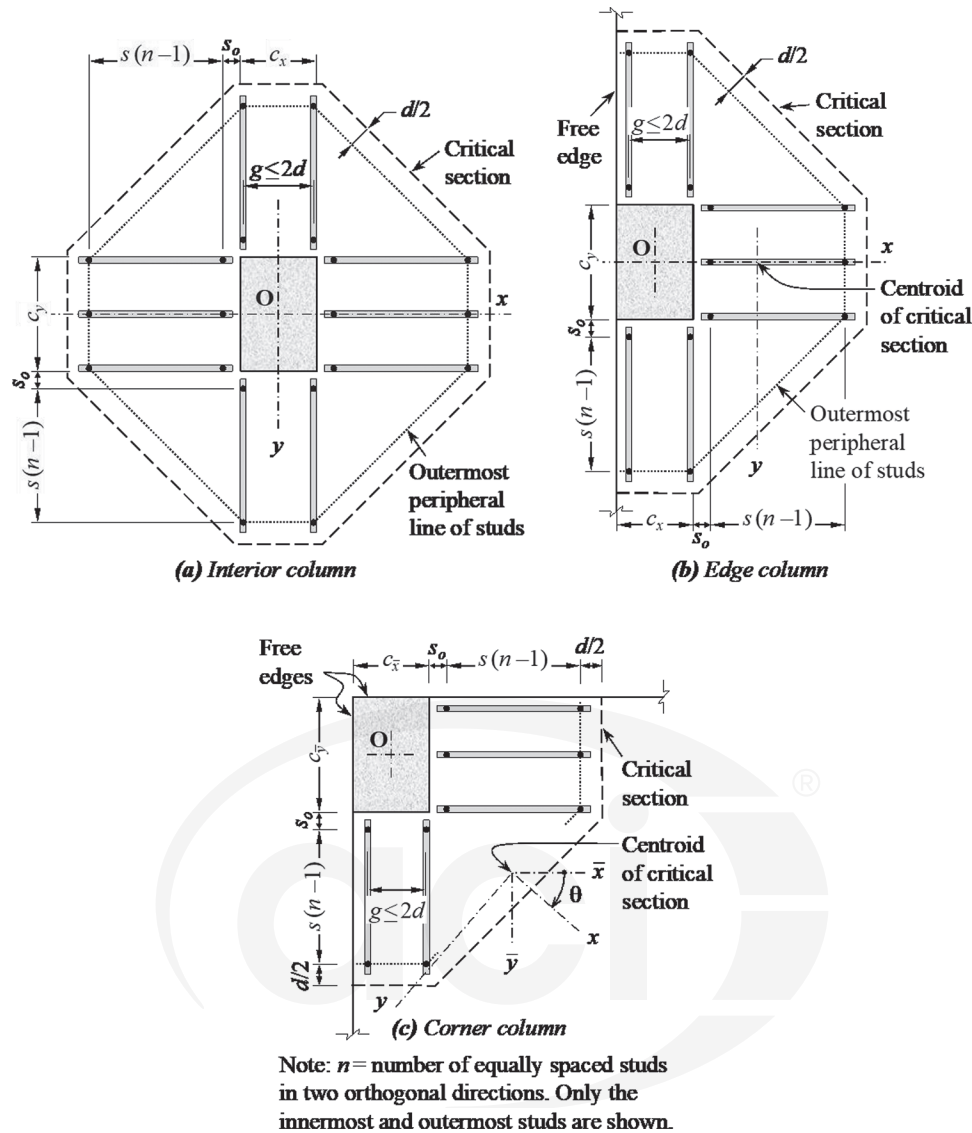


Fig. 4.2—Typical arrangement of shear studs and critical sections outside shear-reinforced zone.

reinforced zone of an interior column (Fig. 4.1(b)), Eq. (4.2b) can be used with l_{x1} and l_{y1} replaced by l_{x2} and l_{y2} , respectively. Appendix B gives equations for J_{cx} , J_{cy} , γ_{vax} , and γ_{vy} for a shear critical section of any shape. For a shear critical section in the shape of a closed rectangle, the shear stress due to V_u combined with M_{sc-y} , ACI 318 gives Eq. (4.2a) with $M_{sc-x} = 0$ and J_{cy} replaced by J_c , which is defined as property of assumed critical section analogous to polar moment of inertia. For the closed rectangle in Fig. 4.1(a), ACI 318 gives

$$J_c = d \left[\frac{l_{x1}^3}{6} + \frac{l_{y1} l_{x1}^2}{2} \right] + \frac{l_{x1} d^3}{6} \quad (4.2c)$$

The first term on the right-hand side of this equation is equal to J_{cy} ; the ratio of the second term to the first is commonly less than 3 percent. The value of v_u obtained by the use of J_{cy} in Eq. (4.2a) differs on the safe side from the value obtained with J_c .

When the centroid of the shear critical section does not coincide with O , the centroid of the column (Fig. 4.2(b) and (c)), the slab moment M_{sc-x} or M_{sc-y} about x - or y -axis through the centroid of shear critical section is related to the slab moment M_{sc-Ox} or M_{sc-Oy} about the x - or y -axis through O by

$$M_{sc-x} = M_{sc-Ox} + V_u y_O; \quad M_{sc-y} = M_{sc-Oy} + V_u x_O \quad (4.2d)$$

where (x_O, y_O) are the coordinates of O with respect to the centroid of the shear critical section along the centroidal principal x - and y -axes.

For the shear critical section in Fig. 4.2(c), the moments about the centroidal nonprincipal axes $\bar{x}$ and $\bar{y}$ ($M_{sc-\bar{x}}$ and $M_{sc-\bar{y}}$) are equivalent to the moments about the x and y axes (M_{sc-x} and M_{sc-y}) that are given by Eq. (4.2e).

$$\begin{aligned} M_{sc-x} &= M_{sc-\bar{x}} \cos\theta - M_{sc-\bar{y}} \sin\theta; \\ M_{sc-y} &= M_{sc-\bar{x}} \sin\theta + M_{sc-\bar{y}} \cos\theta \end{aligned} \quad (4.2e)$$

where θ is the angle of rotation of the axes $\bar{x}$ and $\bar{y}$ to coincide with the principal axes.

4.3—Calculation of shear strength v_n

Whenever the specified compressive strength of concrete f'_c is used in Eq. (4.3.1a), (4.3.1b), (4.3.1c), (4.3.1d), and (4.3.2b), its value must be in pounds per square inch; when f'_c is in MPa, the SI unit equations are used (ACI 318-14 Section 22.6). For prestressed slabs, refer to Chapter 5.

4.3.1 Shear strength without shear reinforcement—For nonprestressed slabs, the shear strength of concrete at a critical section at $d/2$ from column face, where shear reinforcement is not provided, should be the smallest of

$$v_n = \left(2 + \frac{4}{\beta}\right) \lambda \sqrt{f'_c} \quad (\text{in.-lb units})$$

$$v_n = 0.17 \left(1 + \frac{2}{\beta}\right) \lambda \sqrt{f'_c} \quad (\text{SI units}) \quad (4.3.1a)$$

where β is the ratio of long side to short side of the column cross section; λ is a reduction factor for light-weight concretes as defined in ACI 318-14 Section 19.2.4.2.

$$v_n = \left(\frac{\alpha_s d}{b_o} + 2\right) \lambda \sqrt{f'_c} \quad (\text{in.-lb units})$$

$$v_n = 0.083 \left(2 + \frac{\alpha_s d}{b_o}\right) \lambda \sqrt{f'_c} \quad (\text{SI units}) \quad (4.3.1b)$$

where α_s is 40 for interior columns, 30 for edge columns, 20 for corner columns, and

$$v_n = 4 \lambda \sqrt{f'_c} \quad (\text{in.-lb units})$$

$$v_n = 0.33 \lambda \sqrt{f'_c} \quad (\text{SI units}) \quad (4.3.1c)$$

At a critical section outside the shear-reinforced zone,

$$v_n = 2 \lambda \sqrt{f'_c} \quad (\text{in.-lb units})$$

$$v_n = 0.17 \lambda \sqrt{f'_c} \quad (\text{SI units}) \quad (4.3.1d)$$

Equation (4.1) should be checked first at a critical section at $d/2$ from the column face (Fig. 4.1(a)). If Eq. (4.1) is not satisfied, shear reinforcement or some other change is needed.

4.3.2 Shear strength with stirrups—ACI 318-14 Section 22.6.7.1 permits the use of stirrups as shear reinforcement when the slab effective depth $d \geq 6$ in. (152 mm) but not less than 16 times the diameter of the stirrups. When stirrup shear reinforcement is used, ACI 318 requires that the maximum factored shear stress at $d/2$ from column face satisfy: $v_u \leq 6\sqrt{f'_c}$ (in.-lb units) ($0.5\sqrt{f'_c}$ [SI units]). The shear strength at a critical section within the shear-reinforced zone should be computed by

$$v_n = v_c + v_s \quad (4.3.2a)$$

in which

$$v_c = 2 \lambda \sqrt{f'_c} \quad (\text{in.-lb units})$$

$$v_c = 0.17 \lambda \sqrt{f'_c} \quad (\text{SI units}) \quad (4.3.2b)$$

and

$$v_s = \frac{A_v f_{yt}}{b_o s} \quad (4.3.2c)$$

where A_v is the cross-sectional area of the shear reinforcement legs on one peripheral line parallel to the perimeter of the column section, and s is the spacing between peripheral lines of shear reinforcement.

The upper limits, permitted by ACI 318, of s_o and the spacing s between the peripheral lines are

$$s_o \leq 0.5d \quad (4.3.2d)$$

$$s \leq 0.5d \quad (4.3.2e)$$

where s_o is the distance between the first peripheral line of shear reinforcement and the column face (Fig. 4.2). The upper limit of s_o is intended to eliminate the possibility of shear failure between the column face and the innermost peripheral line of shear reinforcement. Similarly, the upper limit of s is to avoid failure between consecutive peripheral lines of stirrups. A line of stirrups too close to the column can be ineffective in intercepting shear cracks; thus, s_o should not be smaller than $0.35d$.

The shear reinforcement should extend away from the column face so that the shear stress v_u at a critical section at $d/2$ from outermost peripheral line of shear reinforcement (Fig. 4.1(b) and 4.2) does not exceed ϕv_n , where v_n is calculated using Eq. (4.3.1d).

4.3.3 Shear strength with studs—ACI 318-14 Section 25.7.1.1 requires that:

Stirrups shall extend as close to the compression and tension surfaces of the member as cover requirements and proximity of other reinforcement permits and shall be anchored at both ends. Where used as shear reinforcement, stirrups shall extend a distance d from extreme compression fiber.

Test results (Dilger and Ghali 1981; Andr  1981a,b; Van der Voet et al. 1982; Mokhtar et al. 1985; Elgabry and Ghali 1987; Mortin and Ghali 1991; Dilger and Shatila 1989; Cao 1993; Brown and Dilger 1994; Megally 1998; Birkle 2004; Ritchie and Ghali 2005; Gayed and Ghali 2006) showed that studs, with anchor heads of area equal to 10 times the cross-sectional area of the shank, clearly satisfied the anchorage requirement. Furthermore, the use of shear studs, shown in Fig. 1.3a, demonstrated a higher shear capacity than that obtained with stirrups. Other researchers (Gayed and Ghali 2004; McLean et al. 1990; Muller et al. 1984; Ghali et al. 1974) successfully applied other configurations. Based on

these results, **ACI 318** permits the values given as follows when the shear reinforcement is composed of headed studs with mechanical anchorage capable of developing the yield strength of the rod.

The nominal shear strength provided by the concrete in the presence of headed shear studs, using Eq. (4.3.2a), is taken as

$$\begin{aligned} v_c &= 3\lambda\sqrt{f'_c} \quad (\text{in.-lb units}) \\ v_c &= 0.25\lambda\sqrt{f'_c} \quad (\text{SI units}) \end{aligned} \quad (4.3.3a)$$

instead of $2\lambda\sqrt{f'_c}$ (in.-lb units) ($0.17\lambda\sqrt{f'_c}$ [SI units]). Discussion on the design value of v_c is given in **Appendix C**. The nominal shear strength v_n (psi or MPa) resisted by concrete and steel in Eq. (4.3.2a) can be taken as high as $8\sqrt{f'_c}$ (in.-lb units) ($0.66\sqrt{f'_c}$ [SI units]) instead of $6\sqrt{f'_c}$ (in.-lb units) ($0.5\sqrt{f'_c}$ [SI units]). This enables the use of thinner slabs. Experimental data showing that the higher value of v_n can be reached are included in Appendix C.

ACI 318 permits upper limits for s based on the value of v_u at the critical section at $d/2$ from column face

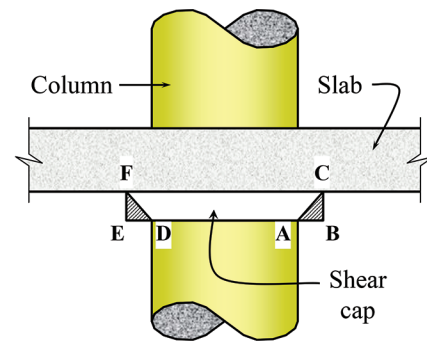
$$\begin{aligned} s \leq 0.75d \quad \text{when} \quad \frac{v_u}{\phi} \leq 6\sqrt{f'_c} \quad (\text{in.-lb units}) \\ (0.5\sqrt{f'_c} \text{ [SI units]}) \end{aligned} \quad (4.3.3b)$$

$$\begin{aligned} s \leq 0.5d \quad \text{when} \quad \frac{v_u}{\phi} > 6\sqrt{f'_c} \quad (\text{in.-lb units}) \\ (0.5\sqrt{f'_c} \text{ [SI units]}) \end{aligned} \quad (4.3.3c)$$

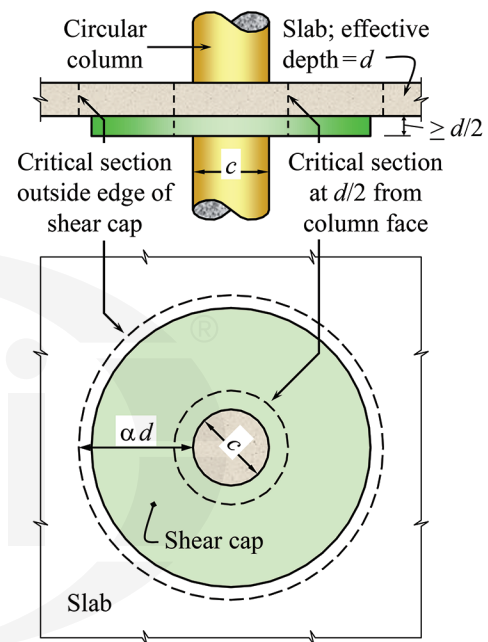
When stirrups are used, ACI 318 limits s to $d/2$. The higher limit for s given by Eq. (4.3.3b) for headed shear stud spacing is again justified by tests (Seible et al. 1980; Andr  1981a,b; Van der Voet et al. 1982; Mokhtar et al. 1985; Elgabry and Ghali 1987; Institut f r Werkstoffe im Bauwesen 1996; Regan 1996a,b; Sherif 1996).

Section 7.3 will show that a vertical branch of a stirrup is less effective than a stud in controlling shear cracks for two reasons: 1) the shank of the headed stud is straight over its full length, whereas the ends of the stirrup leg are curved; and 2) the anchor heads at the top and the bottom of the stud ensure that the specified yield strength is provided at all sections of the shank. In a stirrup, the specified yield strength can be developed only over the middle portion of the vertical legs when they are sufficiently long.

ACI 318-14 Section 20.2.2.4 limits the design yield strength for stirrups as shear reinforcement to 60,000 psi (414 MPa). Research (Otto-Graf-Institut 1996; Regan 1996a; Institut f r Werkstoffe im Bauwesen 1996) has indicated that the performance of higher-strength studs as shear reinforcement in slabs is satisfactory. In this experimental work, the stud shear reinforcement in slab-column connections reached a yield stress higher than 72,000 psi (500 MPa), without excessive reduction of shear resistance of concrete. Thus, when studs are used, f_{yt} can be as high as 72,000 psi (500 MPa). In **ASTM A1044/A1044M**, the minimum speci-



(a) Non-recommended design.



(b) Example of design recommended in Section 4.3.4.

Fig. 4.3.4a—Shear cap design.

fied yield strength of headed shear studs is 51,000 psi (350 MPa) based on what has been commercially available in 2005; higher yield strengths are expected in future versions of ASTM A1044/A1044M. ACI 318 requires conformance with ASTM A1044/A1044M; thus, it limits f_{yt} to 51,000 psi (350 MPa).

4.3.4 Shear caps—Figure 4.3.4a(a) shows, for simplicity of presentation, a circular shear cap whose purpose is to increase the shear capacity without using shear reinforcement. The plan dimensions of the shear cap are governed by assuming that the shear strength at $d/2$ from the edges of the cap is governed by Eq. (4.3.1a) to (4.3.1c). This type of shear cap rarely contains reinforcement other than the vertical bars of the column because its plan dimensions are small; with or without reinforcement, this practice is not recommended. Experiments (Megally and Ghali 2002) show that the failure of the shear cap is accompanied by a sudden separation of wedges ABC and DEF from the shear cap and brittle failure of the connection; such failure may occur in service. The

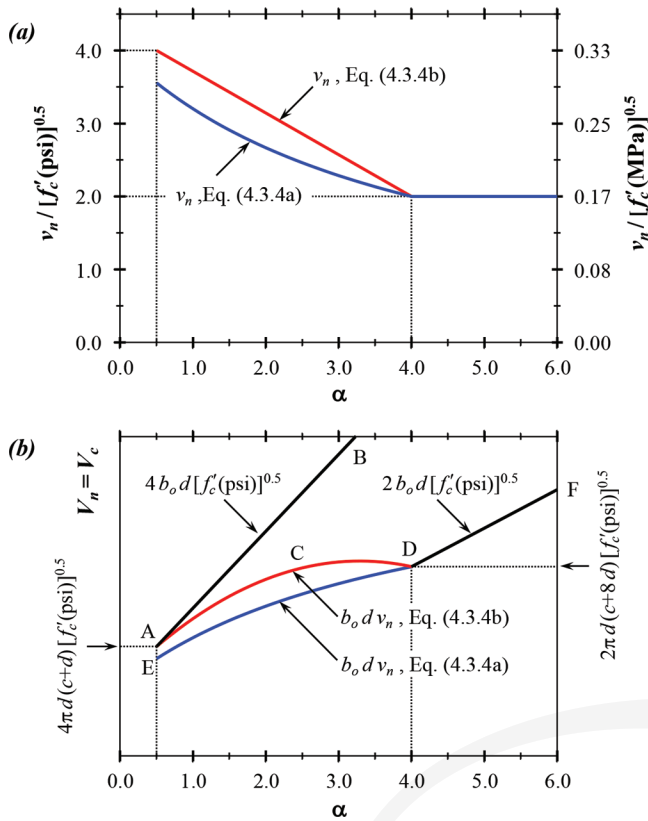


Fig. 4.3.4b—Variation of: (a) v_n ; and (b) V_n with the distance between the shear critical section and the column face ($= \alpha d$).

volume of concrete within the wedges ABC and DEF is too small to offer significant anchorage of the reinforcement that may be provided in the shear cap to prevent the separation of the wedges. Analyses and finite-element studies indicate that when moment, caused by wind or earthquake, is great enough to produce net downward shear stress, an inverted punching failure occurs; in such case, the additional depth provided by the shear cap is not effective to augment the punching shear capacity (Wey and Durrani 1992; Megally and Ghali 2002).

The plan dimensions of the shear cap should be sufficiently large such that the maximum shear stresses at two critical sections (Fig. 4.3.4a(b)) satisfy Eq. (4.1). The critical sections are at $d/2$ from the column face (where d is the total depth within the cap) within the shear cap, and at $d/2$ outside the edges of the shear cap (where d is the slab depth without cap). At $d/2$ from the column, v_n is calculated by Eq. (4.3.1a) to (4.3.1c) in the absence of shear reinforcement. At $d/2$ outside the edges of the shear cap, v_n is calculated by Eq. (4.3.1d). The extent of the shear cap should be the same as the extent of the shear reinforcement when it is used in place of the shear cap.

Based on experimental data, Eligehausen (1996) and Dilger and Ghali (1981) proposed Eq. (4.3.4a) and (4.3.4b), respectively, for the shear strength at critical sections at αd from the column faces.

$$v_n(\alpha) = \left(\frac{1}{1 + 0.25\alpha} \right) 4\sqrt{f'_c}; \quad 0.5 < \alpha < 4.0 \text{ (in.-lb units)}$$

$$v_n(\alpha) = \left(\frac{1}{1 + 0.25\alpha} \right) \frac{\sqrt{f'_c}}{3}; \quad 0.5 < \alpha < 4.0 \text{ (SI units)}$$

$$(4.3.4a)$$

$$v_n(\alpha) = \left(\frac{7.5 - \alpha}{7} \right) 4\sqrt{f'_c}; \quad 0.5 < \alpha < 4.0 \text{ (in.-lb units)}$$

$$v_n(\alpha) = \left(\frac{7.5 - \alpha}{7} \right) \frac{\sqrt{f'_c}}{3}; \quad 0.5 < \alpha < 4.0 \text{ (SI units)}$$

$$(4.3.4b)$$

At $\alpha > 4$, the one-way shear strength (Eq. (4.3.1d)) is assumed. Accordingly, as α is increased, the shear strength (psi or MPa) drops (Fig. 4.3.4b(a)), while the area of the shear critical section increases. Figure 4.3.4b(b) shows the variation of the shear strength, $V_n = v_n(\alpha)b_o d$ for a circular column of diameter c , transferring shearing force without moment. Line AB represents V_n when v_n (psi) = $4\sqrt{f'_c}$ (where v_n is independent of α); this greatly overestimates V_n compared with line ACDF or EDF calculated by Eq. (4.3.4a) or (4.3.4b), respectively. Line DF represents V_n with v_n (psi) = $2\sqrt{f'_c}$. Because within Zone A to D, the variation of v_n is not established, and the increase in V_n with α is not substantial, it is herein recommended to extend the shear cap to the zone where v_n is known to be not less than the one-way shear strength.

As a design example, consider a circular column of diameter c , transferring a shearing force, V_u (lb) = $6\sqrt{f'_c} b_o d$; where $b_o = \pi(c + d)$ = the perimeter of the critical section at $d/2$ from the column face in absence of shear cap. The shear cap that satisfies the recommended design should have an approximate effective depth $\geq 1.5d$, extending such that $\alpha = 1.5(c/d) + 2$. It can be verified that this design will satisfy Eq. (4.1) at the critical sections at $d/2$ from the column face and at $d/2$ outside the edge of the shear cap.

For further justification of the recommendations in this section, consider the slab-column connection in Fig. 4.3.4c(a), having a 10 in. (250 mm) square column supporting a 7 in. (175 mm) slab having $d = 6$ in. (150 mm). Based on the potential crack AB (Fig. 4.3.4c(a)), ACI 318 permits

$$\phi V_n = \phi b_o d 4\sqrt{f'_c};$$

$$\phi V_n^{(a)} = (384 \text{ in.}^2) \phi 4\sqrt{f'_c} \quad (\text{in.-lb units})$$

$$\phi V_n = 0.33 \phi b_o d \sqrt{f'_c};$$

$$\phi V_n^{(a)} = (240 \times 10^3 \text{ mm}^2) 0.33 \phi \sqrt{f'_c} \quad (\text{SI units})$$

The design in Fig. 4.3.4c(b) is not recommended as means of increasing strength. If the ϕV_n equation is applied to the potential crack CD (Fig. 4.3.4c(b)), the predicted strength would be