Shear Reinforcement for Slabs

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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 shear heads. This report reviews other available types and makes recommendations for their design. The application of these recommendations is illustrated through a numerical example.

Keywords: column-slab junction; concrete design; design; moment transfer; prestressed concrete; punching shear; shearheads; shear stresses; shear studs; slabs; two-way floors.

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NOTATION

\( A_c \) = area of concrete of assumed critical section
\( A_v \) = cross-sectional area of the shear studs on one peripheral line parallel to the perimeter of the column section
\( b_p, c_t \) = clear concrete cover of reinforcement to bottom and top slab surfaces, respectively.
\( c_x, c_y \) = size of a rectangular column measured in two orthogonal span directions
\( d \) = effective depth of slab
\( d_b \) = nominal diameter of flexural reinforcing bars
\( D \) = stud diameter
\( f'_c \) = specified compressive strength of concrete
\( f_{ct} \) = average splitting tensile strength of lightweight aggregate concrete
\( f_{pc} \) = average value of compressive stress in concrete in the two directions (after allowance for all prestress losses) at centroid of cross section
\( f_{sy} \) = specified yield strength of shear studs
\( h \) = overall thickness of slab
\( I_x, I_y \) = second moment of area of assumed critical section about the axis x and y
\( J_x, J_y \) = property of assumed critical section analogous to polar moment of inertia about the axes x and y
\( l \) = length of a segment of the assumed critical section
\( l_{x1}, l_{y1} \) = projections of assumed critical section on principal axes x and y
\( l_{x2}, l_{y2} \) = length of sides in the x and y directions of the critical section at \( d/2 \) from column face
\( l_s \) = length of stud (including top anchor plate thickness; see Fig. 7.1)
\( M_{ux}, M_{uy} \) = factored unbalanced moments transferred between the slab and the column about centroidal axes x and y of the assumed critical section
\( n_x, n_y \) = numbers of studs per line/strip running in x and y directions
\( s \) = spacing between peripheral lines of studs
\( s_{o} \) = spacing between first peripheral line of studs and column face
\( v_c \) = nominal shear strength provided by concrete in presence of shear studs
\( v_R \) = nominal shear strength at a critical section
\( v_s \) = nominal shear strength provided by studs
\( v_d \) = maximum shear stress due to factored forces
\( V_p \) = vertical component of all effective prestress forces crossing the critical section
\( V_u \) = factored shear force
\( x, y \) = coordinates of the point at which \( v_u \) is maximum with respect to the centroidal principal axes x and y of the assumed critical section
\( \alpha \) = distance between column face and a critical section divided by \( d \)
\( \alpha_s \) = dimensionless coefficient equal to 40, 30, and 20 for interior, edge and corner columns, respectively
\( \beta_c \) = ratio of long side to short side of column cross section
\( \beta_p \) = constant used to compute \( v_c \) in prestressed slabs
\( \gamma_{ux}, \gamma_{uy} \) = fraction of moment between slab and column that is considered transferred by eccentricity of the shear about the axes x and y of the assumed critical section
\( \phi \) = strength-reduction factor = 0.85

CHAPTER 1—INTRODUCTION

1.1—Objectives

In flat-plate floors, slab-column connections are subjected to high shear stresses produced by the transfer of axial loads and bending moments between slab and columns. Section 11.12.3 of ACI 318 allows the use of shear reinforcement for slabs and footings in the form of bars, as in the vertical legs of stirrups. ACI 318R emphasizes the importance of anchorage details and accurate placement of the shear reinforcement, especially in thin slabs. A general procedure for evaluation of the punching shear strength of slab-column connections is given in Section 11.12 of ACI 318.

Shear reinforcement consisting of vertical rods (studs) or the equivalent, mechanically anchored at each end, can be used. In this report, all types of mechanically-anchored shear reinforcement are referred to as “shear stud” or “stud.” To be fully effective, the anchorage must 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

These recommendations are for the design of shear reinforcement using shear studs in slabs. The design is in accordance with ACI 318, treating a stud as the equivalent of a vertical branch of a stirrup. A numerical design example is included.

1.3—Evolution of the practice

Extensive tests\(^1\)\(^-\)\(^6\) have confirmed the effectiveness of mechanically-anchored shear reinforcement, such as shown in Fig. 1.1,\(^7\) in increasing the strength and ductility of slab-column connections subjected to concentric punching or punching combined with moment. The Canadian Concrete Design Code (CSA A23.3) and the German Construction Supervising Authority, Berlin,\(^8\) allow the use of shear studs for flat slabs (Fig. 1.2). Design rules have been presented\(^9\) for application of British Standard BS 8110 to stud design for slabs. Various
forms of such devices were applied and tested by other investigators, as described in Appendix A.

CHAPTER 2—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 must be as close as possible to the structural member’s outer surfaces. This means that the vertical part of the shear reinforcement must be as tall as possible to avoid the possibility of cracks passing above or below it. When the shear reinforcements are not as tall as possible, they may not intercept all inclined shear cracks. Anchorage of shear reinforcement in slabs is achieved by mechanical ends (heads), bends, and hooks. Tests\textsuperscript{1} have shown, however, that movement occurs at the bends of shear reinforcement, at Point A of Fig. 2.1, before the yield strength can be reached in the shear reinforcement, causing a loss of tension. Furthermore, the concrete within the bend in the stirrups is subjected to stresses that could exceed 0.4 times the stirrup’s yield stress, $f_{yv}$, causing concrete crushing. When $f_{yv}$ is 60 ksi (400 MPa), the average compressive stress on the concrete under the bend can reach 24 ksi (160 MPa) and local crushing can occur. These difficulties, including the consequences of improper stirrup details, have also been discussed by others.\textsuperscript{10-13} The movement at the end of the vertical leg of a stirrup can be reduced by attachment to a flexural reinforcement bar as shown, at Point B of Fig. 2.1.

CHAPTER 3—DESIGN PROCEDURE

3.1—Strength requirement

This chapter presents the design procedure for mechanically-anchored shear reinforcement required in the slab in the vicinity of a column transferring moment and shear. The requirements of ACI 318 are satisfied and a stud is treated as...
the equivalent of one vertical leg of a stirrup. The equations of Section 3.3.2 apply when conventional stirrups are used. The shear studs shown in Fig. 1.2 can also represent individual legs of stirrups.

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

\[ v_u \leq \phi v_n \]  
(3.1)

in which \( v_u \) is the shear stress in the critical section caused by the transfer between the slab and the column of factored axial force or factored axial force combined with moment; and \( v_n \) is the nominal shear strength (Eq. 3.5 to 3.9).

Eq. 3.1 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 but need not approach closer than \( d/2 \) to the outermost peripheral line of shear studs.

3.2—Calculation of factored shear strength \( v_u \)

The maximum factored shear stress \( v_u \) at a critical section produced by the combination of factored shear force \( V_u \) and unbalanced moments \( M_{ux} \) and \( M_{uy} \), is given in Section R11.12.6.2 of ACI 318R:

\[ v_u = \frac{V_u}{A_c} + \frac{\gamma_{ux} M_{ux}}{J_x} + \frac{\gamma_{uy} M_{uy}}{J_y} \]  
(3.2)

in which
- \( A_c \) = area of concrete of assumed critical section
- \( x, y \) = coordinate of the point at which \( v_u \) is maximum with respect to the centroidal principal axes \( x \) and \( y \) of the assumed critical section
- \( M_{ux}, M_{uy} \) = factored unbalanced moments transferred between the slab and the column about the centroidal axes \( x \) and \( y \) of the assumed critical section, respectively
- \( \gamma_{ux}, \gamma_{uy} \) = fraction of moment between slab and column that is considered transferred by eccentricity of shear about the axes \( x \) and \( y \) of the assumed critical section. The coefficients \( \gamma_{ux} \) and \( \gamma_{uy} \) are given by:

\[ \gamma_{ux} = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{l_{x1}/l_{x1}}} \]  
(3.3)

\[ \gamma_{uy} = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{l_{y1}/l_{y1}}} \]  
(3.3)

where \( l_{x1} \) and \( l_{y1} \) are lengths of the sides in the \( x \) and \( y \) directions of the critical section at \( d/2 \) from column face (Fig. 3.1a).

\( J_x, J_y \) = property of assumed critical section, analogous to polar amount of inertia about the axes \( x \) and \( y \), respectively.

In the vicinity of an interior column, \( J_x \) for a critical section at \( d/2 \) from column face (Fig. 3.1a) is given by:

\[ J_y = d \left[ \frac{l_{x1}^3}{6} + \frac{l_{x1} l_{y1}^2}{2} \right] + \frac{l_{y1} d^3}{6} \]  
(3.4)

To determine \( J_x \), interchange the subscripts \( x \) and \( y \) in Eq. (3.4). For other conditions, any rational method may be used (Appendix B).

3.3—Calculation of shear strength \( v_n \)

Whenever the specified compressive strength of concrete \( f'_c \) is used in Eq. (3.5) to (3.10), its value must be in lb per in.\(^2\). For prestressed slabs, see Chapter 4.

3.3.1 Shear strength without shear reinforcement—For non-prestressed 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:

\[ a) \quad v_n = \left( 2 + \frac{4}{\beta_c} \right) \sqrt{f_c'} \]  
(3.5)

where \( \beta_c \) is the ratio of long side to short side of the column cross section.

\[ b) \quad v_n = \left( \frac{\alpha_s d}{b_o} + 2 \right) \sqrt{f_c'} \]  
(3.6)

where \( \alpha_s \) is 40 for interior columns, 30 for edge columns, 20 for corner columns, and

\[ c) \quad v_n = 4 \sqrt{f_c'} \]  
(3.7)

At a critical section outside the shear-reinforced zone,

\[ v_n = 2 \sqrt{f_c'} \]  
(3.8)

Eq. (3.1) should be checked first at a critical section at \( d/2 \) from the column face (Fig. 3.1a). If Eq. (3.1) is not satisfied, shear reinforcement is required.

3.3.2 Shear strength with studs—The shear strength \( v_n \) at a critical section at \( d/2 \) from the column face should not be taken greater than \( 6 \sqrt{f_c'} \) when stud shear reinforcement is provided. The shear strength at a critical section within the shear-reinforced zone should be computed by:

\[ v_n = v_c + v_s \]  
(3.9)

in which

\[ v_c = 2 \sqrt{f_c'} \]  
(3.10)

\[ v_s = \frac{A_v f_{yy}}{b_o s} \]  
(3.11)

where \( A_v \) is the cross-sectional area of the shear studs on one peripheral line parallel to the perimeter of the column section; \( s \) is the spacing between peripheral lines of studs.

The distance \( s_o \) between the first peripheral line of shear studs and the column face should not be smaller than \( 0.35d \). The upper limits of \( s_o \) and the spacing \( s \) between the peripheral lines should be:

\[ s_o \leq 0.4d \]  
(3.12)

\[ s \leq 0.5d \]  
(3.13)

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 studs. Similarly, the upper limit of \( s \) is to avoid failure between consecutive peripheral lines of studs.

The shear studs 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 studs [Fig. 3.1(b) and 3.2] does not exceed \( \phi v_n \), where \( v_n \) is calculated using Eq. (3.8).

3.4—Design procedure

The values of \( f_c', f_{yy}, M_u, V_u, h, \) and \( d \) are given. The design of stud shear reinforcement can be performed by the following steps:

1. At a critical section at \( d/2 \) from column face, calculate \( v_n \) and \( v_s \) by Eq. (3.2) and (3.5) to (3.7). If \( (v_u/\phi) \leq v_n \), no shear reinforcement is required.

2. If \( (v_u/\phi) > v_n \), calculate the contribution of concrete \( v_c \) to the shear strength [Eq. (3.10)] at the same critical section. The difference \( [(v_u/\phi) - v_c] \) gives the shear stress \( v_s \) to be resisted by studs.

3. Select \( s_o \) and stud spacing \( s \) within the limitations of Eq. (3.12) and (3.13), and calculate the required area of stud for one peripheral line \( A_v \), by solution of Eq. (3.11). Find the minimum number of studs per peripheral line.

4. Repeat Step 1 at a trial critical section at \( ad \) from column face to find the section where \( (v_u/\phi) \leq 2 \sqrt{f_c'} \). No other
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section needs to be checked, and $s$ is to be maintained constant. Select the distance between the column face and the outermost peripheral line of studs to be $\geq (ad - d/2)$.

The position of the critical section can be determined by selection of $n_x$ and $n_y$ (Fig. 3.2), in which $n_x$ and $n_y$ are numbers of studs per line running in $x$ and $y$ directions, respectively. For example, the distance in the $x$ direction between the column face and the critical section is equal to $s_o + (n_x - 1) s + d/2$. The two numbers $n_x$ and $n_y$ need not be equal; but each must be $\geq 2$.

5. Arrange studs to satisfy the detailing requirements described in Appendix A.

The trial calculations involved in the above steps are suitable for computer use.  

CHAPTER 4—PRESTRESSED SLABS

4.1—Nominal shear strength

When a slab is prestressed in two directions, the shear strength of concrete at a critical section at $d/2$ from the column face where stud shear reinforcement is not provided is given by ACI 318:

$$v_n = \beta_p \sqrt{f'c} + 0.3f_{pc} + \frac{V_p}{b_o d}$$

(4.1)

where $\beta_p$ is the smaller of 3.5 and $(\alpha d/b_o + 1.5)$; $f_{pc}$ is the average value of compressive stress in the two directions (after allowance for all prestress losses) at centroid of cross section; $V_p$ is the vertical component of all effective prestress forces crossing the critical section. Eq. (4.1) is applicable only if the following are satisfied:

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

b) $f'c$ in Eq. (4.1) is not taken greater than 5000 psi; and

c) $f_{pc}$ in each direction is not less than 125 psi, nor taken greater than 500 psi.

If any of the above conditions are not satisfied, the slab should be treated as non-prestressed and Eq. (3.5) to (3.8) apply. Within the shear-reinforced zone, $v_n$ is to be calculated by Eq. (3.9).

In thin slabs, the slope of the tendon profile is hard to control. Special care should be exercised in computing $V_p$ in Eq. (4.1), to the sensitivity of its value to the as-built tendon profile. When it is uncertain that the actual construction will match design assumption, a reduced or zero value for $V_p$ should be used in Eq. (4.1).

Further, use of the shear device such as shown in Fig. 1.1 demonstrated a higher shear capacity. Other researchers, as briefly mentioned in Appendix A, successfully applied other configurations. Based on these results, following additions to ACI 318 are proposed to apply when the shear reinforcement is composed of studs with mechanical anchorage capable of developing the yield strength of the rod; the values given in Section 5.2 through 5.5 may be used.

5.2—Value for $v_c$

The nominal shear strength provided by the concrete in the presence of shear studs, using Eq. (3.9), can be taken as $v_c = \frac{3.5 \sqrt{f'c}}{2} \text{ instead of } \frac{2 \sqrt{f'c}}{2}$. Discussion on the design value of $v_c$ is given in Appendix C.

5.3—Upper limit for $v_n$

The nominal shear strength $v_n$ resisted by concrete and steel in Eq. (3.9) can be taken as high as $8 \sqrt{f'c}$ instead of $6 \sqrt{f'c}$. This enables the use of thinner slabs. Experimental data showing that the higher value of $v_n$ can be used are included in Appendix C.

5.4—Upper limit for $s$

The upper limits for $s$ can be based on the value of $v_n$ at the critical section at $d/2$ from column face:

$$s \leq \frac{0.75d}{\phi} \text{ when } \frac{v_n}{\phi} \leq 6 \sqrt{f'c}$$

(5.1)

$$s \leq \frac{0.5d}{\phi} \text{ when } \frac{v_n}{\phi} > 6 \sqrt{f'c}$$

(5.2)

When stirrups are used, ACI 318 limits $s$ to $d/2$. The higher limit for $s$ given by Eq. (5.1) for stud spacing is again justified by tests (see Appendix C).

As mentioned earlier in Chapter 2, a vertical branch of a stirrup is less effective than a stud in controlling shear cracks for two reasons: a) The stud stem is straight over its full length, whereas the ends of the stirrup branch are curved; and b) The anchor plates at the top and bottom of the stud ensure that the specified yield strength is provided at all sections of the stem. In a stirrup, the specified yield strength can be developed only over the middle portion of the vertical legs when they are sufficiently long.

5.5—Upper limit for $f_y$

Section 11.5.2 of ACI 318 limits the design yield strength for stirrups as shear reinforcement to 60,000 psi. Research 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 yield stress higher than 72,000 psi, without excessive reduction of shear resistance of concrete. Thus, when studs are used, $f_y$ can be as high as 72,000 psi.

CHAPTER 5—SUGGESTED HIGHER ALLOWABLE VALUES FOR $v_c$, $v_m$, $s$, AND $f_{yv}$

5.1—Justification

Section 11.5.3 of ACI 318 requires that “stirrups and other bars or wires used as shear reinforcement shall extend to a distance $d$ from extreme compression fiber and shall be anchored at both ends according to Section 12.13 to develop the design yield strength of reinforcement.” Test results show that studs with anchor heads of area equal to 10 times the cross section area of the stem clearly satisfied that requirement.
accurately. Tolerances for these dimensions should not exceed \( \pm 0.5 \) in. If this requirement is not met, a punching shear crack can traverse the slab thickness without intersecting the shear reinforcing elements. Tolerance for the distance between column face and outermost peripheral line of studs should not exceed \( \pm 1.5 \) in.

**CHAPTER 7—DESIGN EXAMPLE**

The design procedure presented in Chapter 3 is illustrated by a numerical example for an interior column of a non-pre-stressed slab. A design example for studs at edge column is presented elsewhere.\(^{18}\) There is divergence of opinions with respect to the treatment of corner and irregular columns.\(^{18,20}\)

The design of studs is required at an interior column based on the following data: column size \( c \) by \( c_y = 12 \times 20 \) in.; slab thickness = 7 in.; concrete cover = 0.75 in.; yield strength of studs \( f_y = 60 \) ksi; and flexural reinforcement nominal diameter = 5/8 in. The factored forces transferred from the column to the slab are: \( V_y = 110 \) kip and \( M_{uy} = 50 \) ft-kip. The five steps of design outlined in Chapter 3 are followed:

**Step 1**—The effective depth of slab

\[
d = 7 - 0.75 - (5/8) = 5.62 \text{ in.}
\]

Properties of a critical section at \( d/2 \) from column face shown in Fig. 7.1: \( b_o = 86.5 \) in.; \( A_e = 486 \) in.\(^2\); \( J_y = 28.0 \times 10^3 \) in.\(^4\); \( l_{s1} = 17.62 \) in.; \( l_{s1} = 25.62 \) in.

The fraction of moment transferred by shear [Eq. (3.3)]:

\[
\gamma_{vy} = 1 - \frac{1}{1 + \frac{17.62}{8.81 \times 25.62}} = 0.36
\]

The maximum shear stress occurs at \( x = 17.62/2 = 8.81 \) in. and its value is [Eq. (3.2)]:

\[
v_u = \frac{110 \times 1000}{486} + \frac{0.36(50 \times 12,000)8.81}{28.0 \times 10^3} = 294 \text{ psi}
\]

\[
\frac{v_u}{\phi} = \frac{294}{0.85} = 346 \text{ psi} = 5.5 \sqrt{f_y}
\]

The nominal shear stress that can be resisted without shear reinforcement at the critical section considered [Eq. (3.5) to (3.7)]:

\[
v_n = \left( 2 + \frac{4}{1.67} \right)\sqrt{f_y} = 4.4 \sqrt{f_y}
\]

\[
v_n = \left[ \frac{40(5.62)}{86.5} + 2 \right]\sqrt{f_y} = 4.6 \sqrt{f_y}
\]

\[
v_n = 4 \sqrt{f_y}
\]

use the smallest value: \( v_n = 4 \sqrt{f_y} = 253 \text{ psi} \)

**Fig. 7.1**—Section in slab perpendicular to shear stud line.

**Step 2**—The quantity \( v_u/\phi \) is greater than \( v_u \), indicating that shear reinforcement is required; the same quantity is less than the upper limit \( v_n = 6 \sqrt{f_y} \), which means that the slab thickness is adequate.

The shear stress resisted by concrete in the presence of the shear reinforcement [Eq. (3.10)] at the same critical section:

\[
v_c = 2 \sqrt{f_y} = 126 \text{ psi}
\]

Use of Eq. (3.1), (3.9), and (3.11) gives:

\[
v_s \geq \frac{v_u}{\phi} - v_c = 346 - 126 = 220 \text{ psi}
\]

\[
\frac{A_v}{s} \geq \frac{v_u b_o}{s} = \frac{220(86.5)}{60,000} = 0.32 \text{ in.}
\]

**Step 3**—

\[
s_o \leq 0.4 d = 2.25 \text{ in.}; s \leq 0.5 d = 2.8 \text{ in.}
\]

This example has been provided for one specific type of shear stud reinforcement, but the approach can be adapted and used also for other types mentioned in Appendix A.

Try 3/8 in. diameter studs welded to a bottom anchor strip 3/16 x 1 in. Taking cover of 3/4 in. at top and bottom, the length of stud \( l_s \) (Fig. 7.1) should not exceed:

\[
l_{s_{max}} = 7 - 2\left( \frac{3}{4} - \frac{3}{16} \right) = 5 \frac{5}{16} \text{ in.}
\]

or the overall height of the stud, including the two anchors, should not exceed 5.5 in.

Also, \( l_s \) should not be smaller than:

\[
l_{s_{min}} = l_{s_{max}} - \text{ one bar diameter of flexural reinforcement}
\]

\[
l_{s_{min}} = 5 \frac{5}{16} - \frac{5}{8} = 4 \frac{11}{16} \text{ in.}
\]
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Choose \( l_s = 5\text{-}1/4 \) in. With 10 studs per peripheral line, choose the spacing between peripheral lines, \( s = 2.75 \) in., and the spacing between column face and first peripheral line, \( s_o = 2.25 \) in. (Fig. 7.2).

This value is greater than 0.32 in., indicating that the choice of studs and their spacing is adequate.

Step 4—Properties of critical section at 4\( d \) from column face [Fig. 3.1(b):]
\[ \alpha = 4.0; \ \alpha d = 4(5.62) = 22.5 \text{ in.}; \]
\[ l_{x1} = 14.3 \text{ in.}; l_{y1} = 22.3 \text{ in.}; \]
\[ l_{x2} = 57.0 \text{ in.}; l_{y2} = 65.0 \text{ in.}; \]
\[ l = 30.2 \text{ in.}; b_o = 194.0 \text{ in.}; \]
\[ A_c = 1090 \text{ in.}^2; J_c = 449.5 \times 10^3 \text{ in.}^4. \]

The maximum shear stress in the critical section occurs on line AB at:

\[ x = 57/2 = 28.5 \text{ in.}; \text{ Eq. (3.2) gives:} \]

\[ v_u = \frac{110,000}{1090} + \frac{0.36(50 \times 12,000)28.5}{449.5 \times 10^3} = 115 \text{ psi} \]

\[ v_u = \frac{115}{0.85} = 135 \text{ psi} \]

\[ v_n = 2 \sqrt{f_c'} = 126 \text{ psi} \]

The value \( v_u/\phi = 135 \text{ psi} \) is greater than \( v_n = 126 \text{ psi} \), which indicates that shear stress should be checked at \( \alpha d > 4 \). Try eight peripheral lines of studs; distance between column face and outermost peripheral line of studs:

\[ \alpha d = s_o + 7s = 2.25 + 7(2.75) = 22 \text{ in.} \]

Check shear stress at a critical section at a distance from column face:

\[ \alpha d = 22 + d/2 = 22.0 + 5.62/2 = 24.8 \text{ in.} \]

\[ \alpha = \frac{24.8}{d} = \frac{24.8}{5.62} = 4.4 \]

\[ v_u = 125 \text{ psi} \]

\[ v_n = 2 \sqrt{f_c'} = 126 \text{ psi} \]

Step 5—The value of \( v_u/\phi \) is less than \( v_n \), which indicates that details of stud arrangement as shown in Fig. 7.2 are adequate.

The value of \( V_u \) used to calculate the maximum shear stress could have been reduced by the counteracting factored load on the slab area enclosed by the critical section.

If the higher allowable values of \( v_c \) and \( s \) proposed in Chapter 5 are adopted in this example, it will be possible to use only six peripheral lines of studs instead of eight, with spacing \( s = 4.0 \) in., instead of 2.75 in. used in Fig. 7.2.

CHAPTER 8—REQUIREMENTS FOR SEISMIC-RESISTANT SLAB-COLUMN CONNECTIONS IN REGIONS OF SEISMIC RISK

Connections of columns with flat plates should not be considered in design as part of the system resisting lateral forces. However, due to the lateral movement of the structure in an earthquake, the slab-column connections transfer vertical shearing force \( V \) combined with reversal of moment \( M \). Experiments\(^{21-23} \) were conducted on slab-column connections to simulate the effect of interstorey drift in a flat-slab structure. In these tests, the column was transferring a constant shearing force \( V \) and cyclic moment reversal with increasing magnitude. The experiments showed that, when the slab is provided with stud shear reinforcement the connections behave in a ductile fashion. They can withstand, without failure, drift ratios varying between 3 and 7\%, depending upon...
the magnitude of $V$. The drift ratio is defined as the difference between the lateral displacements of two successive floors divided by the floor height. For a given value $V_u$, the slab can resist a moment $M_u$, which can be determined by the procedure and equations given in Chapters 3 and 5; but the value of $v_c$ should be limited to:

$$v_c = \frac{1.5}{f_c'}$$

(8.1)

This reduced value of $v_c$ is based on the same experiments, which indicate that the concrete contribution to the shear resistance is diminished by the moment reversals. This reduction is analogous to the reduction of $v_c$ to 0 by Section 21.3.4.2 of ACI 318 for framed members.

CHAPTER 9—REFERENCES

9.1—Recommended references

The documents of the various standards-producing organizations referred to in this document are listed below with their serial designation.

American Concrete Institute
318/318R Building Code Requirements for Structural Concrete and Commentary

British Standards Institution
BS 8110 Structural Use of Concrete

Canadian Standards Association
CSA-A23.3 Design of Concrete Structures for Buildings

The above publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 9094
Farmington Hills, MI 48333-9094

British Standards Institution
2 Park Street
London W1A 2BS
England

Canadian Standards Association
178 Rorldale Blvd.
Rexdale, Ontario M9W 1R3
Canada

9.2—Cited references


7. U.S. patent No. 4406103. Licensee: Deha, represented by Decon, Medford, NJ, and Brampton, Ontario.


APPENDIX A—DETAILS OF SHEAR STUDS
A.1—Geometry of stud shear reinforcement

Several types and configurations of shear studs have been reported in the literature. Shear studs mounted on a continuous steel strip, as discussed in the main text of this report, have been developed and investigated. Headed reinforcing bars were developed and applied in Norway for high-strength concrete structures, and it was reported that such applications improved the structural performance significantly. Another type of headed shear reinforcement was implemented for increasing the punching shear strength of lightweight concrete slabs and shells. Several other approaches for mechanical anchorage in shear reinforcement can be used. Several types are depicted in Fig. A1; the figure also shows the required details of stirrups when used in slabs according to ACI 318R.

The anchors should be in the form of circular or rectangular plates, and their area must be sufficient to develop the specified yield strength of studs $f_{yw}$. It is recommended that
the performance of the shear stud reinforcement be verified before their use. The user can find such information in the cited references.

A.2—Stud arrangements
Shear studs in the vicinity of rectangular columns should be arranged on peripheral lines. The term peripheral line is used in this report to mean a line running parallel to and at constant distance from the sides of the column cross section. Fig. 3.2 shows a typical arrangement of stud shear reinforcement in the vicinity of a rectangular interior, edge, and corner columns. Tests\(^1\) showed that studs are most effective near column corners. For this reason, shear studs in Fig. 3.2(a), (b), and (c) are aligned with column faces. In the direction parallel to a column face, the distance \(g\) between lines of shear studs should not exceed \(2d\), where \(d\) is the effective depth of the slab. When stirrups are used, the same limit for \(g\) should be observed [Fig. A1(a)].

The stud arrangements for circular columns are shown in Fig. A2. The minimum number of peripheral lines of shear studs, in the vicinity of rectangular and circular columns, is two.

A.3—Stud length
The studs are most effective when their anchors are as close as possible to the top and bottom surfaces of the slab. Unless otherwise protected, the minimum concrete cover of the anchors should be as required by Section 7.7 of ACI 318. The cover of the anchors should not exceed the minimum cover plus one half bar diameter of flexural reinforcement (Fig. 7.1). The mechanical anchors should be placed in the forms above reinforcement supports, which insure the specified concrete cover.

**APPENDIX B—PROPERTIES OF CRITICAL SECTIONS OF GENERAL SHAPE**
This appendix is general; it applies regardless of the type of shear reinforcement used. Fig. 3.1 shows the top view of critical sections for shear in slab in the vicinity of interior column. The centroidal \(x\) and \(y\) axes of the critical sections, \(V_u\), \(M_{ux}\), and \(M_{uy}\), are shown in their positive directions. The shear force \(V_u\) is acting at the column centroid; \(V_u\), \(M_{ux}\), and \(M_{uy}\) represent the effects of the column on the slab.

In use of Eq. (3.2), \(v_u\) for a section of general shape (for example, Fig. 3.1 and 3.2), the parameters \(J_x\) and \(J_y\) may be calculated using the formulas:

\[
J_x = \int y^2 \, dA \\
J_y = \int x^2 \, dA \\
J = \int (x^2 + y^2) \, dA
\]
Fig. B2—Equations for $\gamma_x$ and $\gamma_y$ applicable for critical sections at $d/2$ from column face and outside shear-reinforced zone. Note: $l_x$ and $l_y$ are projections of critical sections on directions of principal $x$ and $y$ axes.

Table C1—List of references on slab-column connections tests using stud shear reinforcement

<table>
<thead>
<tr>
<th>Experiment no.</th>
<th>Reference no.</th>
<th>Experiment no.</th>
<th>Reference no.</th>
<th>Experiment no.</th>
<th>Reference no.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 to 5</td>
<td>33</td>
<td>16 to 18</td>
<td>16</td>
<td>26 to 29</td>
<td>38</td>
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<tr>
<td>6, 7</td>
<td>See footnote*</td>
<td>19 to 20</td>
<td>35</td>
<td>30 to 36</td>
<td>3</td>
</tr>
<tr>
<td>8, 9</td>
<td>34</td>
<td>21 to 24, 37</td>
<td>36</td>
<td>42</td>
<td>39</td>
</tr>
<tr>
<td>10 to 15</td>
<td>17</td>
<td>25, 38 to 41</td>
<td>37</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>


Table C2—Slabs with stud shear reinforcement failing within shear-reinforced zone

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Square column size, in.</th>
<th>$f_{ct}$, in.</th>
<th>$d$, in.</th>
<th>$d/l$</th>
<th>$\sigma_{d}$</th>
<th>$V_{u}$, kip</th>
<th>$M_{u}$, kip-in.</th>
<th>$V_{v}$</th>
<th>$f_{v}$</th>
<th>$A_{v}$, in.$^2$</th>
<th>$V_{test}/V_{code}$</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
<td>(5)</td>
<td>(6)</td>
<td>(7)</td>
<td>(8)</td>
</tr>
<tr>
<td>20</td>
<td>7.87</td>
<td>5660</td>
<td>6.30</td>
<td>0.75</td>
<td>714</td>
<td>599</td>
<td>64.1</td>
<td>1.402</td>
<td>1.00</td>
<td>Interior column</td>
<td></td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>9.84</td>
<td>4100</td>
<td>4.49</td>
<td>0.70</td>
<td>214</td>
<td>528</td>
<td>55.1</td>
<td>1.14</td>
<td></td>
<td>Edge column</td>
<td></td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>9.84</td>
<td>4030</td>
<td>4.49</td>
<td>0.70</td>
<td>528</td>
<td>590</td>
<td>55.1</td>
<td>1.28</td>
<td></td>
<td>Edge column</td>
<td></td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>9.84</td>
<td>4080</td>
<td>4.49</td>
<td>0.70</td>
<td>714</td>
<td>641</td>
<td>55.1</td>
<td>1.39</td>
<td></td>
<td>Edge column</td>
<td></td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>9.84</td>
<td>4470</td>
<td>4.49</td>
<td>0.70</td>
<td>599</td>
<td>693</td>
<td>55.1</td>
<td>1.66</td>
<td></td>
<td>Edge column</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>9.84</td>
<td>4890</td>
<td>4.49</td>
<td>0.75</td>
<td>570</td>
<td>667</td>
<td>55.1</td>
<td>1.48</td>
<td></td>
<td>Edge column</td>
<td></td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>9.84</td>
<td>5660</td>
<td>4.49</td>
<td>0.75</td>
<td>570</td>
<td>641</td>
<td>55.1</td>
<td>1.570</td>
<td></td>
<td>Interior column</td>
<td></td>
<td></td>
</tr>
<tr>
<td>27</td>
<td>9.84</td>
<td>6610</td>
<td>4.49</td>
<td>0.75</td>
<td>665</td>
<td>667</td>
<td>55.1</td>
<td>1.570</td>
<td></td>
<td>Interior column</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>9.84</td>
<td>6610</td>
<td>4.49</td>
<td>0.75</td>
<td>665</td>
<td>667</td>
<td>55.1</td>
<td>1.570</td>
<td></td>
<td>Interior column</td>
<td></td>
<td></td>
</tr>
<tr>
<td>29</td>
<td>9.84</td>
<td>5470</td>
<td>4.49</td>
<td>0.75</td>
<td>117</td>
<td>454</td>
<td>40.3</td>
<td>1.320</td>
<td></td>
<td>Interior column</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>9.84</td>
<td>4210</td>
<td>4.45</td>
<td>0.88</td>
<td>113</td>
<td>444</td>
<td>47.1</td>
<td>0.460</td>
<td></td>
<td>Interior column</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Semi-lightweight concrete: $\sqrt[6.7]{f_{ct}}$ is replaced in calculation by $f_{ct}/6.7$. $f_{ct}$ is average splitting tensile strength of lightweight aggregate concrete; $f_{ct}$ used here = 377 psi, determined experimentally.

$V_{v,calc}$ is smaller of $8\sqrt[6.7]{f_{ct}}$ and $(3\sqrt[6.7]{f_{ct}} + v_{c})$, where $v_{c} = A_{v}f_{ct}/(b_{y}l)$. 

Mean: 1.18

Coefficient of variation: 0.17
be approximated by the second moments of area \( I_x \) and \( I_y \) given in Eq. (B-2) and (B-3). The coefficients \( \gamma_{vx} \) and \( \gamma_{vy} \) are given in Fig. B2, which is based on finite element studies.\(^{30,31}\)

The critical section perimeter is generally composed of straight segments. The values of \( A_c, I_x, \) and \( I_y \) can be determined by summation of the contribution of the segments (Fig. 3.2):

\[
A_c = d \sum l \quad (B-1)
\]

\[
I_x = d \sum \left[ \frac{l}{3} (y_j^2 + y_j y_i + y_i^2) \right] \quad (B-2)
\]

\[
I_y = d \sum \left[ \frac{l}{3} (x_i^2 + x_i x_j + x_j^2) \right] \quad (B-3)
\]

where \( x_i, y_i, x_j, \) and \( y_j \) are coordinates of Points \( I \) and \( j \) at the extremities of the segment whose length is \( l \) (Fig. B1).

When the maximum \( v_u \) occurs at a single point on the critical section, rather than on a side, the peak value of \( v_u \) does not govern the strength due to stress redistribution.\(^{21}\) In this case, \( v_u \) may be investigated at a point located at a distance 0.4\( d \) from the peak point. This will give a reduced \( v_u \) value compared with the peak value; the reduction should not be allowed to exceed 15%.

### APPENDIX C—VALUES OF \( v_c \) WITHIN SHEAR REINFORCED ZONE

This design procedure of the shear reinforcement requires calculation of \( v_u = v_c + v_s \) at the critical section at \( d/2 \) from the column face. The value allowed for \( v_c \) is \( 2\sqrt{f_c' } \) when stirrups are used, and \( 3\sqrt{f_c' } \) when shear studs are used. The reason for the higher value of \( v_c \) for slabs with shear stud

---

**Table C3—Experiments with maximum shear stress \( v_u \) at critical section of \( d/2 \) from column face exceeding \( 8\sqrt{f_c' } \) (slabs with stud shear reinforcement)**

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Column size, in.*</th>
<th>( f_c' ), psi</th>
<th>( \sqrt{f_c' } ), psi</th>
<th>Tested capacities</th>
<th>( M_{at\ critical\ section\ centroid},\ kip-in. )</th>
<th>( d, ) in.</th>
<th>Maximum shear stress ( v_u ), psi</th>
<th>( v_u / 8\sqrt{f_c' } )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.81 sq.</td>
<td>6020</td>
<td>621</td>
<td>476</td>
<td>0</td>
<td>9.06</td>
<td>629</td>
<td>1.07</td>
</tr>
<tr>
<td>2</td>
<td>11.81 sq.</td>
<td>5550</td>
<td>589</td>
<td>428</td>
<td>0</td>
<td>8.86</td>
<td>585</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>11.81 sq.</td>
<td>3250</td>
<td>456</td>
<td>346</td>
<td>0</td>
<td>8.66</td>
<td>488</td>
<td>1.07</td>
</tr>
<tr>
<td>4</td>
<td>19.68 cr.</td>
<td>5550</td>
<td>589</td>
<td>665</td>
<td>0</td>
<td>10.51</td>
<td>667</td>
<td>1.13</td>
</tr>
<tr>
<td>5</td>
<td>14.57 sq.</td>
<td>6620</td>
<td>651</td>
<td>790</td>
<td>0</td>
<td>11.22</td>
<td>682</td>
<td>1.05</td>
</tr>
<tr>
<td>6</td>
<td>12.60 cr.</td>
<td>5870</td>
<td>613</td>
<td>600</td>
<td>0</td>
<td>9.33</td>
<td>934</td>
<td>1.52</td>
</tr>
<tr>
<td>7</td>
<td>12.60 cr.</td>
<td>6020</td>
<td>621</td>
<td>620</td>
<td>0</td>
<td>9.33</td>
<td>965</td>
<td>1.55</td>
</tr>
<tr>
<td>8</td>
<td>10.23 sq.</td>
<td>3120</td>
<td>447</td>
<td>271</td>
<td>0</td>
<td>8.07</td>
<td>459</td>
<td>1.03</td>
</tr>
<tr>
<td>9</td>
<td>10.23 sq.</td>
<td>3270</td>
<td>457</td>
<td>343</td>
<td>0</td>
<td>8.07</td>
<td>582</td>
<td>1.27</td>
</tr>
<tr>
<td>10</td>
<td>7.48 cr.</td>
<td>3310</td>
<td>460</td>
<td>142</td>
<td>0</td>
<td>5.83</td>
<td>582</td>
<td>1.26</td>
</tr>
<tr>
<td>11</td>
<td>7.48 cr.</td>
<td>3260</td>
<td>456</td>
<td>350</td>
<td>0</td>
<td>9.60</td>
<td>679</td>
<td>1.48</td>
</tr>
<tr>
<td>12</td>
<td>7.48 cr.</td>
<td>4610</td>
<td>543</td>
<td>159</td>
<td>0</td>
<td>6.02</td>
<td>623</td>
<td>1.14</td>
</tr>
<tr>
<td>13</td>
<td>7.48 cr.</td>
<td>3050</td>
<td>441</td>
<td>128</td>
<td>0</td>
<td>5.91</td>
<td>516</td>
<td>1.17</td>
</tr>
<tr>
<td>14</td>
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<td>3340</td>
<td>462</td>
<td>278</td>
<td>0</td>
<td>9.72</td>
<td>530</td>
<td>1.14</td>
</tr>
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<td>15</td>
<td>7.48 cr.</td>
<td>3160</td>
<td>449</td>
<td>255</td>
<td>0</td>
<td>9.76</td>
<td>482</td>
<td>1.07</td>
</tr>
<tr>
<td>16</td>
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<td>4630</td>
<td>544</td>
<td>207</td>
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<td>728</td>
<td>1.34</td>
</tr>
<tr>
<td>17</td>
<td>9.25 cr.</td>
<td>5250</td>
<td>580</td>
<td>216</td>
<td>0</td>
<td>6.14</td>
<td>725</td>
<td>1.25</td>
</tr>
<tr>
<td>18</td>
<td>9.25 cr.</td>
<td>5290</td>
<td>582</td>
<td>234</td>
<td>0</td>
<td>6.50</td>
<td>725</td>
<td>1.24</td>
</tr>
<tr>
<td>19</td>
<td>7.87 sq.</td>
<td>5060</td>
<td>569</td>
<td>236</td>
<td>0</td>
<td>6.30</td>
<td>661</td>
<td>1.16</td>
</tr>
<tr>
<td>20</td>
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<td>5660</td>
<td>601</td>
<td>214</td>
<td>0</td>
<td>6.30</td>
<td>599</td>
<td>1.00</td>
</tr>
<tr>
<td>21*</td>
<td>9.84 sq.</td>
<td>4100</td>
<td>513</td>
<td>47.4</td>
<td>651</td>
<td>4.49</td>
<td>491</td>
<td>1.03</td>
</tr>
<tr>
<td>22*</td>
<td>9.84 sq.</td>
<td>4030</td>
<td>508</td>
<td>52.8</td>
<td>730</td>
<td>4.49</td>
<td>552</td>
<td>1.16</td>
</tr>
<tr>
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<td>4080</td>
<td>511</td>
<td>26.9</td>
<td>798</td>
<td>4.49</td>
<td>708</td>
<td>1.25</td>
</tr>
<tr>
<td>24*</td>
<td>9.84 sq.</td>
<td>4470</td>
<td>535</td>
<td>27.2</td>
<td>847</td>
<td>4.49</td>
<td>755</td>
<td>1.29</td>
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<tr>
<td>25</td>
<td>9.84 sq.</td>
<td>4280</td>
<td>523</td>
<td>135</td>
<td>0</td>
<td>4.45</td>
<td>532</td>
<td>1.02</td>
</tr>
<tr>
<td>26</td>
<td>9.84 sq.</td>
<td>4890</td>
<td>559</td>
<td>33.7</td>
<td>1434</td>
<td>4.49</td>
<td>1434</td>
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<tr>
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<td>5660</td>
<td>602</td>
<td>67.4</td>
<td>1257</td>
<td>4.49</td>
<td>1257</td>
<td>1.06</td>
</tr>
<tr>
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<td>5920</td>
<td>615</td>
<td>67.4</td>
<td>1328</td>
<td>4.49</td>
<td>1328</td>
<td>1.08</td>
</tr>
<tr>
<td>29</td>
<td>9.84 sq.</td>
<td>6610</td>
<td>651</td>
<td>101</td>
<td>929</td>
<td>4.49</td>
<td>924</td>
<td>1.03</td>
</tr>
</tbody>
</table>

| Mean       | 1.17             |
| Coefficient of variation | 0.13 |

*Column 2 gives side dimension of square (sq.) columns or diameter of circular (cr.) columns.

†Edge slab-column connections. Other experiments are on interior slab-column connections.
reinforcement is the almost slip-free anchorage of the studs. In structural elements reinforced with conventional stirrups, the anchorage by hooks or 90-deg bends is subject to slip, which can be as high as 0.04 in. when the stress in the stirrup leg approaches its yield strength.\(^{32}\) This slip is detrimental to the effectiveness of stirrups in slabs because of their relative small depth compared with beams. The influence of the slip is manifold:

- Increase in width of the shear crack;
- Extension of the shear crack into the compression zone;
- Reduction of the shear resistance of the compression zone; and
- Reduction of the shear friction across the crack.

All of these effects reduce the shear capacity of the concrete in slabs with stirrups. To reflect the stirrup slip in the shear resistance equations, refinement of the shear failure model is required. The empirical equation \(v_n = v_c + v_s\) adopted in almost all codes, is not the ideal approach to solve the shear design problem. A mechanics-based model that is acceptable for codes is not presently available. There is, however, enough experimental evidence that use of the empirical equation \(v_n = v_c + v_s\), with \(v_c = 3 \sqrt{f_{ctc}}\) gives a safe design for slabs with stud shear reinforcement. This approach is adopted in Canadian code (CSA 23.3).

Numerous test slab-column connections reinforced with shear studs are reported in the literature (Table C1). In the majority of these, the failure is at sections outside the shear-reinforced zone. Table C2 lists only the tests in which the failure occurred within the shear-reinforced zone. Column 12 of Table C2 gives the ratio \(v_{test}/v_{code}\) where \(v_{code}\) is the

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Column size, in.</th>
<th>(f_y^2), psi</th>
<th>(d), in.</th>
<th>(s/d)</th>
<th>(V_c^c), kip</th>
<th>(M) at critical section centroid, kip-in.</th>
<th>Maximum shear stress (f_{sv}), psi</th>
<th>(A_v), in(^2)</th>
<th>(v_{test}), psi</th>
<th>(V_{test}/V_{code}^a)</th>
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</table>

| Mean        | 1.31 |
| Coefficient of variation | 0.23 |

\(^{1}\) Slab 30 is semi-lightweight. Concrete \(f_{ctc}\) replaced in calculations by \(f_y/6.7\); \(f_y\) average splitting tensile strength of lightweight aggregate concrete; \(f_y\) used here = 377 psi, determined experimentally.

\(^{2}\) Column 2 gives side dimension of square (sq.) columns, or diameter of circular (cr.) columns.

\(^{3}\) For cube strengths, concrete cylinder strength in Column 3 calculated using \(f_y = 0.83f_{ctc}\).

\(^{4}\) Column 9 is maximum shear stress at failure in critical section at \(d/2\) from column face.

\(^{5}\) \(v_{test}\) is value allowed by ACI 318 combined with proposed equations in Chapter 5. \(v_{test}\) calculated at \(d/2\) from column face when failure is within stud zone and at section at \(d/2\) from outermost studs when failure is outside shear-reinforced zone.

\(^{6}\) \(v_{code}\) is the average splitting tensile strength of lightweight aggregate concrete; \(\bar{v}_{code}\) not given for slabs that failed within stud zone.

\(^{7}\) \(v_{test}\) is value allowed by ACI 318 combined with proposed equations in Chapter 5. \(v_{test}\) calculated at \(d/2\) from column face when failure is within stud zone and at section at \(d/2\) from outermost studs when failure is outside shear-reinforced zone.

**Table C4—Slabs with stud shear reinforcement having \(s\) approximately equal to or greater than 0.75\(d\)**
value allowed by ACI 318, but with $v_c = 3\sqrt{f'_c}$ (instead of $2\sqrt{f'_c}$). The values of $v_{test}/v_{code}$ being greater than 1.0 indicate there is safety of design with $v_c = 3\sqrt{f'_c}$.

Table C3 summarizes experimental data of numerous slabs in which the maximum shear stress $v_n$ obtained in test, at the critical section at $d/2$ from column face, reaches or exceeds $8\sqrt{f'_c}$. Table C3 indicates that $v_n$ can be safely taken equal to $8\sqrt{f'_c}$ (Section 5.3).

Table C4 gives the experimental results of slabs having stud shear reinforcement with the spacing between studs greater or close to the upper limit given by Eq. (5.1). In Table C4, $v_{code}$ is the nominal shear stress calculated by ACI 318, combined with the provisions suggested in Chapter 5. The value $v_{code}$ is calculated at $d/2$ from column face when failure is within the shear-reinforced zone, or at a section at $d/2$ from the outermost studs when failure occurs outside the shear-reinforced zone. The ratio $v_{test}/v_{code}$ being greater than 1.0 indicates that it is safe to use studs spaced at the upper limit set by Eq. (5.1) and calculate the strength according to ACI 318 combined with the provisions in Chapter 5.