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Seismic Design of Punching Shear Reinforcement in Flat Plates—Guide

Reported by Joint ACI-ASCE Committee 421

ACI PRC-421.2-21



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Seismic Design of Punching Shear Reinforcement in Flat Plates—Guide

Reported by Joint ACI-ASCE Committee 421

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During an earthquake, moments transferred at flat plate-column connections can produce significant shear stresses that increase their vulnerability to brittle punching shear failure. This guide provides recommendations for designing flat plate-column connections with sufficient ductility to withstand reversals of lateral drift without punching shear failure or loss of moment transfer capacity. Reinforced concrete flat plates with or without post-tensioning are discussed in this guide.

Keywords: ductility; flat plate; post-tensioning; punching shear; seismic design; shear reinforcement; shear stud reinforcement.

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ACI PRC-421.2-21 supersedes ACI 421.2R-07 and was adopted and published August 2021.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—General

Brittle punching failure can occur due to the transfer of shear forces combined with moments between slabs and columns. During an earthquake, significant horizontal displacement of a flat plate-column connection may occur, resulting in moments inducing shear stresses, as evidenced by the collapse of flat plate structures due to punching shear

failures in past earthquakes (Berg and Stratta 1964; Yanev et al. 1991; Mitchell et al. 1990, 1995). During the 1985 Mexico earthquake (Yanev et al. 1991), 91 waffle-slab and solid-slab buildings collapsed, and another 44 buildings suffered severe damage. Hueste and Wight (1999) studied a building with a post-tensioned flat plate that experienced punching shear failures during the 1994 Northridge, CA, earthquake. Their study provided a relationship between the level of gravity load and the maximum story drift ratio that a flat plate-column connection can undergo without punching shear failure. Even when an independent lateral-force-resisting system is provided, flat plate-column connections should be designed to accommodate the moments and shear forces associated with the displacements during earthquakes.

1.2—Scope

Seismic design should account for the displacement-induced moment and the accompanying shear forces at flat plate-column connections. This demand may be effectively addressed by changes in dimensions of certain members (for example, shear walls and column sizes), their material strengths, provision of shear reinforcement, or a combination thereof. This guide does not address changes in dimensions and materials of such members but focuses solely on the punching shear design of flat plates with or without shear reinforcement.

This guide is supplemental to ACI 421.1R and focuses on the design of flat plate-column connections with or without shear reinforcement that are subject to earthquake-induced displacement; reinforced concrete flat plates with or without post-tensioning are also discussed in this guide. Stirrups and headed shear stud reinforcement (HSSR) satisfying ASTM A1044/A1044M are the common types of shear reinforcement for flat plates. Shear stud reinforcement is composed of vertical or inclined rods anchored mechanically near the bottom and top surfaces of the slab. Forged heads or welded plates can be used as the anchorage of HSSR; the area of the head or the plate is sufficient to develop the yield strength of the stud, with negligible slip at the anchorage. The design procedure recommended in this guide was developed based on numerical studies (finite element method) and experimental research on reinforced concrete slabs subjected to cyclic drift reversals that simulate seismic effects (Megally 1998). The finite element analyses, supplemental to the experimental research, used software, constitutive relations, and models that were subject to extensive verifications by comparing the results of the analyses with the behavior observed in tests (Megally and Ghali 2000b).

ACI 318 requires that structural integrity reinforcement be provided near the bottom of the slab extending through the columns. The code requirements in ACI 318-14 are addressed (refer to Eq. (4b)). This guide supplements ACI 352.1R and ACI 421.1R, which, respectively, include recommendations such as extending a minimum amount of integrity reinforcement through the column core and providing details of design for shear reinforcement in flat plates. For post-tensioned slabs, the integrity reinforcement can be in the top of the slab as they approach the column, in which

case, the integrity reinforcement should be nonprestressed bars. ACI 352.1R also provides recommendations for the design of flat plate-column connections without slab shear reinforcement subjected to moment transfer in the inelastic-response range. The equations in this guide predict punching shear strength and drift capacity, assuming that adequate flexural reinforcement is provided at the flat plate-column connections to prevent local flexural failure (Gayed et al. 2017). Wide flexural cracks in the vicinity of support can extend deep through slab thickness, joining an inclined shear crack inducing failure; this is flexure-induced punching. The minimum amount of top flexural reinforcement above support to resist flexure-induced punching is determined by yield line analysis (Gayed and Ghali 2019). Identifying the punching cause of failure in tests can be difficult when flexural reinforcement is inadequate (Cheng et al. 2010).

1.2.1 Objective—The objective is to provide a design recommendation for flat plate-column connections with sufficient ductility to accommodate the displacement of the selected lateral-force-resisting system without punching shear failure or loss of moment transfer capacity. Reinforced concrete slabs with or without post-tensioning are covered. Minimum requirements of shear reinforcement in addition to those in ACI 421.1R are needed for ductility. The role of shear reinforcement in connections subjected to moment reversals induced by earthquakes is to ensure no collapse until a yield-line mechanism develops in the vicinity of the support. Development of the yield-line mechanism is considered evidence of ductility as defined in Section 2.2.

1.2.2 Remarks—This guide gives recommendations for the design of shear reinforcement for strength and considerations for ductility that supplement the provisions of ACI 318. The term “ductility” is the ratio of displacement at ultimate strength to the displacement at which yielding of the flexural reinforcement develops a yield-line mechanism.

ACI 318 allows the analysis of flat plate-column frames as equivalent plane frames. When the frame is not designated as part of the lateral-force-resisting system and is subjected to horizontal displacements, the width of slab strip to be included in the frame model accounting for cracking is specified in ACI 318-14 Section R8.11.2. This parameter significantly affects the resulting computed values of the moments transferred between slabs and columns.

Chapter 3 defines the story drift that should be considered in design. Chapter 4 recommends a minimum amount of shear reinforcement for certain cases. Chapter 5 describes means of increasing the shear strength of a flat plate-column connection and compares the associated ductilities. Chapter 6 calculates the moment transferred between a slab and column for use in designing the required flexural reinforcement. Chapter 7 and Appendix D discuss relevant provisions of ACI 318 and provide the design procedure and examples for interior, edge, and corner flat plate-column connections. Chapter 8 provides recommendations relevant to post-tensioned flat plates. Appendixes A and B are justifications of recommendations in the chapters. Appendix C provides equations for the properties of shear critical sections.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A_s	= area of flexural reinforcing bars, in. ² (mm ²)
A_v	= cross-sectional area of shear reinforcement on one peripheral line, in. ² (mm ²)
b_o	= length of perimeter of shear-critical section, in. (mm)
C_d	= displacement amplification factor
c_1 to c_6	= dimensions defining a post-tensioning tendon profile, in. (mm)
c_x, c_y	= column dimensions in the x - and y -directions, respectively, in. (mm)
DR_u	= ultimate story drift ratio at peak strength (in experiments), or design story drift ratio of a flat plate-column connection
d	= average of distances from extreme compression fiber to the centroid of the tension reinforcement positioned in two orthogonal directions, in. (mm)
E_c	= elastic modulus of concrete, psi (MPa)
e	= tendon eccentricity, measured from the midsurface of the slab to the centroid of the post-tensioned tendon; e is positive for a tendon situated below midsurface, in. (mm)
f'_c	= specified compressive strength of concrete, psi (MPa)
f_{pc}	= average in-plane compressive stress produced by effective post-tensioning forces in two orthogonal directions, psi (MPa)
f_{ps}	= stress in post-tensioned reinforcement at nominal flexural strength of slab, psi (MPa)
f_{py}	= specified yield strength of post-tensioned reinforcement, psi (MPa)
f_{se}	= effective stress in a post-tensioned tendon after accounting for all post-tensioning losses, psi (MPa)
f_y	= specified yield strength of flexural reinforcement, psi (MPa)
f_{yt}	= specified yield strength of shear reinforcement, psi (MPa)
h	= slab thickness, in. (mm)
h_c	= distance between centerline of prestressing tendons above support to that in the midspan, in. (mm)
I_c or I_s	= moment of inertia of column or slab about centroidal principal axis, in. ⁴ (mm ⁴)
I_e	= occupancy importance factor
I_{ec}	= equivalent moment of inertia of columns, accounting for torsional members for use in a plane-frame analysis, as opposed to a special-purpose equivalent frame analysis, in. ⁴ (mm ⁴)
J_c	= property of assumed critical section analogous to polar moment of inertia, in. ⁴ (mm ⁴)
J_x or J_y	= property of critical section = $d[(y^2 \text{ or } x^2) \cdot dl = d \times \text{second moment of perimeter about centroidal principal } x\text{- or } y\text{-axis, in.}^4 \text{ (mm}^4\text{)}$
J_{xy}	= d times product of inertia of assumed critical section about nonprincipal axes $\bar{x}$ and $\bar{y}$, in. ⁴ (mm ⁴)

K_c	= end rotational stiffness of column, moment per unit rotation	w_L	= service live load per unit area, psf (N/m ²)
K_{ec}	= end rotational stiffness of equivalent column, moment per unit rotation	w_{sd}	= superimposed dead load per unit area, psf (N/m ²)
l	= span length, in. (mm)	x, y	= coordinates of point of maximum shear stress on the critical section with respect to centroidal principal axes x and y , respectively, in. (mm); as subscripts, x and y refer to the same principal axes
l_c	= story height, in. (mm)	$\bar{x}, \bar{y}$	= coordinates of point on perimeter of shear-critical section with respect to centroidal nonprincipal axes $\bar{x}$ and $\bar{y}$, in. (mm)
l_x, l_y	= projections of shear-critical section on its principal axes x and y , respectively, in. (mm)	α_m	= factor used in calculation of an optional reduced value for the design moment for earthquake-resistant flat plate-column connections
M_{Ox}, M_{Oy}	= moment transferred between slab and a column about an axis parallel to the principal x - or y -axis and passing through O , centroid of column; positive M_{Ox} or M_{Oy} is in the same direction as positive $M_{sc,u-x}$ or $M_{sc,u-y}$, in.-lb (N·mm)	α_s	= factor that adjusts v_c for support type
M_p'	= primary bending moment due to post-tensioning, in.-lb (N·mm)	β	= ratio of long side to short side of concentrated load or reaction area
M_p''	= secondary (indeterminate) post-tensioned bending moment, in.-lb (N·mm)	β_r	= aspect ratio of the shear-critical section at $d/2$ from column face
M_{pr}	= probable flexural strength, in.-lb (N·mm)	δ_e	= story drift used in elastic frame analysis, in. (mm)
$M_{prestress}$	= bending moment produced by post-tensioning forces, in.-lb (N·mm)	δ_u	= design story drift, including inelastic deformations, in. (mm)
$M_{sc,u}$	= ultimate moment transferred between slab and a column at shear critical section centroid, in.-lb (N·mm); this definition applies where capacity is considered; where demand is considered, M_{sc} is factored moment transferred between slab and a column in design	γ_v	= fraction of the moment $M_{sc,u}$ transferred by vertical shear stresses at flat plate-column connections
$M_{sc,u-x}, M_{sc,u-y}$	= components of the moment $M_{sc,u}$ transferred between the slab and the column, in.-lb (N·mm)	η	= factor to account for rigid ends of column
P_e	= absolute value of effective post-tensioning force per unit slab width, lb/in. (N/mm)	κ	= fraction of service dead load to be balanced by effective post-tensioning
s	= spacing between peripheral lines of shear reinforcement, in. (mm)	λ	= modification factor reflecting the reduced mechanical properties of lightweight concrete, relative to normalweight concrete of the same compressive strength
s_o	= spacing between the first peripheral line of shear reinforcement and column face, in. (mm)	θ	= clockwise rotation angle of $\bar{x}$ -axis to x -axis, or $\bar{y}$ -axis to y -axis
V	= service shear force transferred between the column and the slab, lb (N)	θ_p	= angle between tangent to the tendon and the centroidal axis
V_c	= nominal punching shear strength provided by concrete, lb (N)	ρ	= slab reinforcement ratio
V_p	= shear force produced by post-tensioning forces or vertical component of effective post-tensioning force crossing the shear critical section at $d/2$ from column face, lb (N)	ρ_p	= ratio of post-tensioning reinforcement
V_p'	= primary shear force due to post-tensioning, lb (N)	ϕ	= strength reduction factor
V_p''	= secondary (indeterminate) post-tensioning shear force, lb (N)		
V_u	= ultimate shear force transferred between the slab and the column, lb (N)		
v_c	= nominal shear strength provided by concrete (expressed in stress units), psi (MPa) ($= V_c/[b_o d]$)		
v_n	= nominal shear strength (expressed in stress units), psi (MPa)		
v_s	= nominal shear strength (expressed in stress units) provided by shear reinforcement, psi (MPa)		
v_u	= maximum factored two-way shear stress at critical section, psi (MPa)		
w_c	= weight of concrete per unit volume, pcf (N/m ³)		
w_D	= service dead load per unit area, psf (N/m ²)		

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.

design displacement—total lateral displacement specified for the design-basis earthquake as required by the governing code for earthquake-resistant design.

design story drift—design displacement of one level, or floor, relative to the level above or below.

design story drift ratio—design story drift divided by the story height.

ductility—ratio of displacement at ultimate strength to the displacement at which yield of the flexural reinforcement occurs.

flexure-induced punching—punching failure through flexural crack joining a shear crack.

headed shear stud reinforcement (HSSR)—reinforcement composed of vertical rods anchored mechanically to the bottom and top surfaces of the slab.

lateral-force-resisting system—portion of the structure composed mainly of vertical members designed to resist lateral forces related to earthquake or wind effects.

probable flexural strength—flexural strength based on $1.25f_y$.

shear-critical section—section at $d/2$ from face of support, or at $d/2$ from outermost peripheral line of shear reinforcement (Fig. 3.3b and 4).

CHAPTER 3—LATERAL STORY DRIFT

3.1—Lateral-force-resisting systems

In Seismic Design Categories (SDCs) D, E, and F, flat plate buildings need to rely on a lateral-force-resisting system that limits lateral displacement. Flat plates will experience the same lateral displacements as the lateral-force-resisting system and should be designed to do so without losing their capability to support gravity loads during or after an earthquake.

The lateral-force-resisting system should have sufficient stiffness to control the lateral displacement of the structure and limit the maximum lateral story drift to the values prescribed by the general building code, as discussed in Section 3.2. The story drift ratio is defined as the story drift divided by the story height. The story height is defined as the distance between the midsurfaces of the consecutive flat plates at the top and bottom of the story of interest.

Moments $M_{sc,u}$ due to design displacements are transferred between slabs and columns and are a direct result of the lateral displacement experienced by the entire building. The transfer of shear forces V_u and moment $M_{sc,u}$ increases the risk of a punching shear failure during an earthquake.

ACI 352.1R provides connection design recommendations for flat plate-column frames and considers that such frames may be adequate as the lateral-force-resisting system in regions of low and moderate seismic hazard. This guide excludes the selection or the design of the lateral-force-resisting systems; it includes the punching shear design of flat plates, with and without shear reinforcement, subjected to gravity loads combined with moment $M_{sc,u}$ induced by the lateral drift of the flat plate-column connections.

3.2—Limits on story drift ratio

It has been recommended that flat plate structures should have the capability to withstand a design story drift ratio of at least 0.015, including inelastic deformations (Sozen 1980; ACI 352.1R; Pan and Moehle 1989).

In the force-based design approach, a static elastic analysis of the lateral-force-resisting system is performed to determine the elastic story drift δ_e (due to forces specified in ASCE/SEI 7). This value is then multiplied by factors, also given in ASCE/SEI 7, to obtain the design story drift δ_u .

ASCE/SEI 7 requires that the calculated δ_u (depending on the lateral-force-resisting system and the SDC), including inelastic deformation, not exceed 0.007 to 0.025 of the story height. The code also requires that the flat plate-column connection be adequate to carry the vertical load and the induced moments and shears resulting from the calculated design story drift.

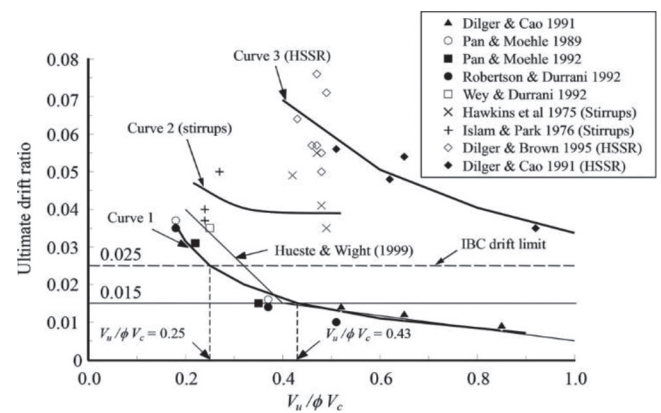


Fig. 3.3a—Effect of gravity loads on drift capacity of interior flat plate-column connections (Megally and Ghali 1994, 2000d; Hueste and Wight 1999).

The design story drift ratio may reach the upper limit of ASCE/SEI 7 without punching shear failure when the flat plate-column connections are provided with slab shear reinforcement or when the gravity load produces low punching shear stress, as discussed in Chapter 4.

3.3—Effects of gravity loads on story drift capacity

Figure 3.3a (Megally and Ghali 1994, 2000d; Hueste and Wight 1999) shows the variation of the observed ultimate story drift ratio DR_u for interior flat plate-column connections transferring gravity shear forces V_u with the ratio $V_u/\phi V_c$ when subjected to reversals of cyclic drift. The experimental values of DR_u are compared with the design story drift δ at peak strength, divided by the story height. V_u is the ultimate shear force transferred between the slab and the column at failure; V_c is the nominal punching shear strength of the flat plate-column connection without shear reinforcement in the absence of moment transfer. Each data point represents the combination of $V_u/\phi V_c$ and the corresponding ultimate story drift ratio DR_u at which a test specimen failed in punching shear.

Curves 1, 2, and 3 shown in Fig. 3.3a fit the results of experiments reported in the literature on interior flat plate-column connections without shear reinforcement, with conventional stirrups, and with headed shear stud reinforcement (HSSR), respectively (Pan and Moehle 1989, 1992; Robertson and Durrani 1992; Wey and Durrani 1992; Hawkins et al. 1975; Islam and Park 1976; Dilger and Brown 1995; Brown 2003; Dilger and Cao 1991; Cao 1993). The strength reduction factor ϕ is taken equal to 1.0 in Fig. 3.3a because the strength of materials is known for the experiments. In preparing Fig. 3.3a, the shear strength V_c , calculated in accordance with Section 22.6 of ACI 318-14, is used as the smallest of Eq. (3.3a) through (3.3c)

$$V_c = \left(2 + \frac{4}{\beta}\right) b_o d \lambda \sqrt{f'_c} \quad (\text{in.-lb}) \quad (3.3a)$$

$$V_c = \left(1 + \frac{2}{\beta}\right) b d \lambda \sqrt{f'_c} \quad (\text{SI}) \quad (3.3b)$$

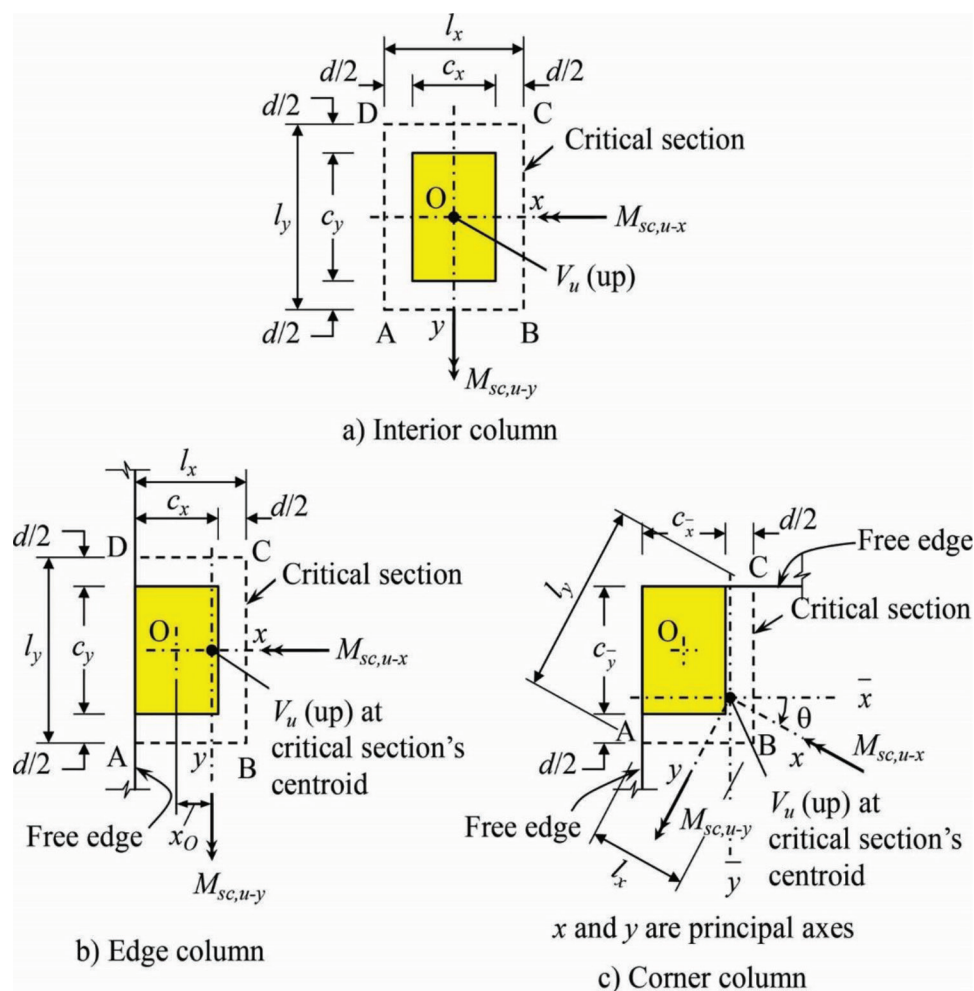


Fig. 3.3b—Critical sections for punching shear at $d/2$ from column face. The vectors for V_u and $M_{sc,u}$ represent the forces exerted by the column on the slab in their positive directions. x and y are centroidal principal axes of the critical section.

$$V_c = \left(\frac{\alpha_s d}{b_o} + 2 \right) b_o d \lambda \sqrt{f'_c} \quad (\text{in.-lb}) \quad (3.3b)$$

$$V_c = \left(\frac{\alpha_s d}{b_o} + 2 \right) \frac{b_o d \lambda \sqrt{f'_c}}{12} \quad (\text{SI})$$

$$V_c = 4b_o d \lambda \sqrt{f'_c} \quad (\text{in.-lb}) \quad (3.3c)$$

$$V_c = \frac{1}{3} b_o d \lambda \sqrt{f'_c} \quad (\text{SI})$$

where b_o is the perimeter of the critical section for shear at $(d/2)$ from the column face (Fig. 3.3b); d is the average of the distances from extreme compression fiber to the centroid of tension reinforcement in two orthogonal directions; and $\alpha_s = 40, 30$, and 20 for interior, edge, and corner flat plate-column connections, respectively (refer to Fig. 3.3b). The vectors for V_u and $M_{sc,u}$ shown in Fig. 3.3b(a) through (c) represent the forces exerted by the column on the slab in their positive directions.

Figure 3.3a indicates that the flat plate-column connection capability to experience story drift without failure decreases

with increasing magnitude of the applied gravity load. The solid horizontal line shown in Fig. 3.3a represents the design story drift ratio of 0.015 , which is frequently adopted as a minimum drift capacity. This horizontal line intersects Curve 1 at $(V_u/\phi V_c) \cong 0.40$, indicating that slabs without shear reinforcement can satisfy the required 0.015 design story drift ratio only if V_u does not exceed $0.40\phi V_c$. The same conclusion can be seen from a similar graph containing more data (Kang and Wallace 2005). The horizontal dashed line plotted in Fig. 3.3a, corresponding to the 0.025 design story drift ratio limit specified by ASCE/SEI 7 (for structures satisfying conditions specified in Section 12.12 of ASCE/SEI 7), intersects Curve 1 at approximately $(V_u/\phi V_c) \cong 0.25$. This suggests that for $DR_u = 0.025$, the slab should be designed with shear reinforcement when V_u exceeds approximately $0.25\phi V_c$. Section 3.4 conservatively recommends provision of shear reinforcement when $DR_u = 0.025$ and $(V_u/\phi V_c) > 0.20$. This complies with ACI 318-14 Section 18.14.5.1.

European codes consider V_c at different shear critical sections. The shear strength is based on the opening of a critical shear crack (Muttoni and Ruiz 2008).

Figure 3.3c is similar to Fig. 3.3a, but the two curves shown in Fig. 3.3c represent the results of experiments on edge flat plate-column connections (Megally and Ghali

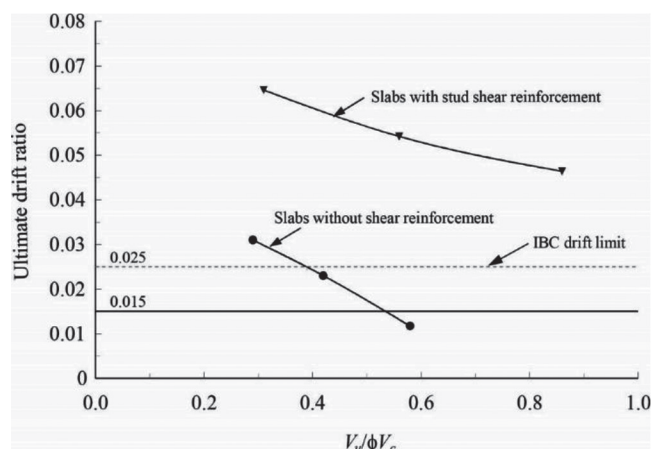


Fig. 3.3c—Effect of value of V_u on lateral drift capacity of edge flat plate-column connections (Megally and Ghali 2000a).

2000a). The graphs indicate that for $DR_u = 0.015$, the flat plate should be designed with shear reinforcement when $V_u > 0.40\phi V_c$. A comparison of Fig. 3.3a and 3.3c indicates that a somewhat higher limit on $(V_u/\phi V_c)$ can be allowed for edge connections than for interior connections for the same design story drift ratio. For simplicity, a single limit on V_u is recommended in Section 3.4. In addition to the limitation on V_u , the absence of shear reinforcement should be allowed only when the maximum shear stress is less than that given in Chapter 4.

Figures 3.3a and 3.3c show that the curves representing experiments of flat plate-column connections with shear reinforcement fall well above the horizontal lines. Curves 2 and 3 of Fig. 3.3a are not perfect fits because of the scatter of experimental data. The uncertainty of the curves, however, does not change the conclusion that no limit on $(V_u/\phi V_c)$ is needed for slabs with shear reinforcement exceeding the minimum amount given in Chapter 4 to sustain the 0.025 design story drift ratio required by ASCE/SEI 7. Figure 3.3a also shows that the drift capacity for slabs with HSSR is higher than slabs with stirrups.

3.4—Design recommendations for flat plates with and without shear reinforcement

As discussed in Section 3.1, the lateral-force-resisting structural system should have sufficient stiffness to control the story drift. Slab shear reinforcement is required when the maximum shear stress at $d/2$ from the column face exceeds ϕv_c , where v_c is the value given by Eq. (3.3a) to (3.3c) divided by $b_o d$. In addition, flat plate-column connections should have shear reinforcement equal to or exceeding the minimum amount given in Chapter 4, except when the value of V_u is less than $0.20\phi V_c$, and $\phi = 0.75$ (ACI 318-14 Section 21.2.1). This requirement ensures that the connections can sustain a design story drift ratio $DR_u = 0.025$. If it can be shown by analysis that when the maximum story drift ratio DR_u , including the inelastic deformations, is between 0.015 and 0.025, shear reinforcement is required when V_u exceeds $\phi V_c(0.70 - 20DR_u)$. This is the same as required by

ACI 318-14 Section 18.14.5.1, that shear reinforcement be provided when DR_u exceeds $0.035 - 0.05V_u/(\phi V_c)$.

Curve 1 in Fig. 3.3a shows that for slabs without shear reinforcement, DR_u can reach 0.015 only when $V_u \leq 0.40\phi V_c$. The same curve also shows conservatively that DR_u can reach 0.025 without shear reinforcement when $V_u \leq 0.20\phi V_c$. The same two conclusions can be reached if Curve 1 is replaced by the bilinear graph of Hueste and Wight (1999), plotted in the same figure. The bilinear graph represents an approximation of the same test data used in Fig. 3.3a for slabs without shear reinforcement combined with data from the literature.

CHAPTER 4—MINIMUM SHEAR AND INTEGRITY REINFORCEMENTS IN FLAT PLATES

Seismic design of nonprestressed flat plates can be without shear reinforcement only when $(V_u/\phi V_c)$ is limited, as discussed in Section 3.4. In addition, the maximum shear stress due to V_u , combined with that due to the moment $M_{sc,u}$, computed according to the linear stress distribution assumption of ACI 318, should not exceed ϕv_c , expressed in stress units (the smallest of Eq. (3.3a) through (3.3c) divided by $b_o d$), at the critical section at $d/2$ from the column face. For ductility, connections that do not satisfy both conditions should have slab shear reinforcement that satisfies Eq. (4a).

$$v_s = \frac{A_v f_{yt}}{b_o s} \geq 3\sqrt{f'_c} \quad (\text{in.-lb}) \quad (4a)$$

$$v_s = \frac{A_v f_{yt}}{b_o s} \geq \frac{1}{4}\sqrt{f'_c} \quad (\text{SI})$$

where A_v is the area of shear reinforcement in each peripheral line parallel to the column faces (typical peripheral lines are shown in Fig. 4(a) to (c)); s is the spacing between peripheral lines of shear reinforcement ($s \leq 0.5d$ for stirrups; $s \leq 0.75d$ for headed shear stud reinforcement [HSSR]); b_o is the perimeter of the critical section at $d/2$ from the column face (Fig. 3.3b); and f_{yt} is the specified yield strength of the shear reinforcement. The distance between the column face and the outermost peripheral line of shear reinforcement (Fig. 4) should not be less than $3.5d$. The case that requires the minimum reinforcement for ductility is shown in the design flow chart given in Section 7.2.

ACI 318 and ACI 421.1R provide equations to calculate the maximum shear stress at the shear-critical section at $d/2$ from the column face; when the maximum shear stress exceeds $\phi V_c/(b_o d)$, shear reinforcement is required for strength. Also for strength, the extent of the shear-reinforced zone is determined by limiting the maximum shear stress at the critical section at $d/2$ from the outermost peripheral line of shear reinforcement to $2\phi\lambda\sqrt{f'_c}$ psi ($0.17\phi\lambda\sqrt{f'_c}$ MPa) (ACI 318-14 Section 22.6.6.1). When shear reinforcement is required for strength, its extent should not be less than $3.5d$ from the column face.

The minimum shear reinforcement recommended by Eq. (4a) with the minimum extension shown in Fig. 4 was

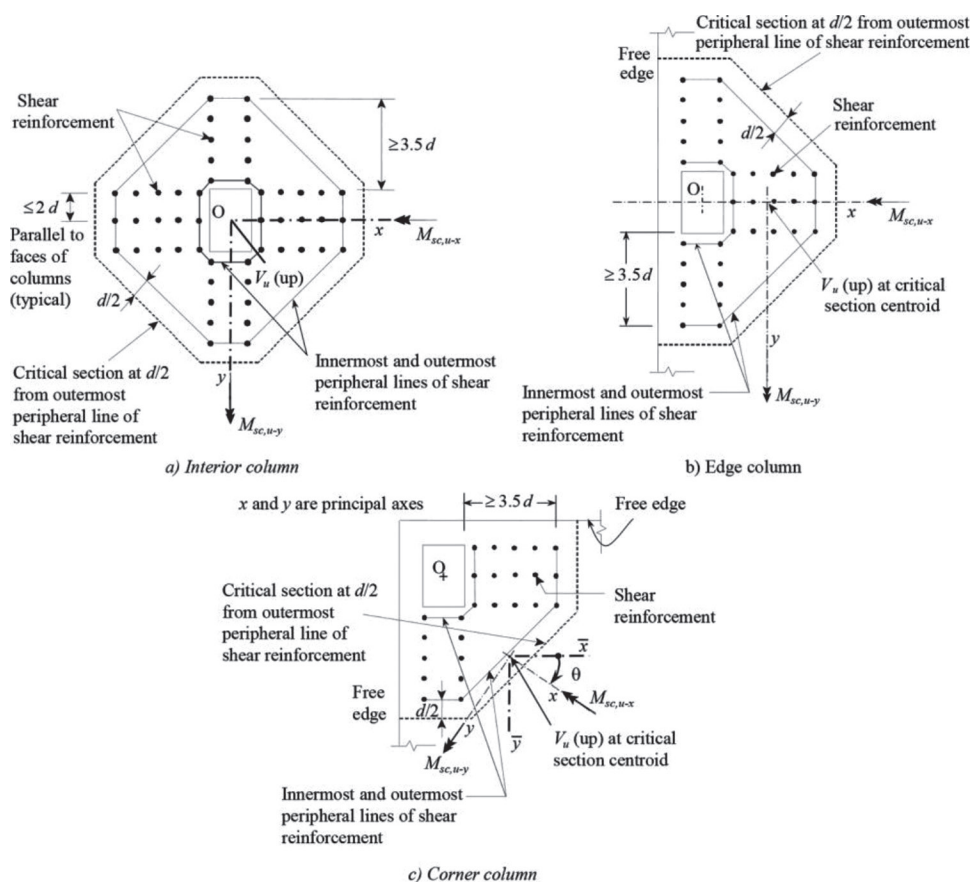


Fig. 4—Critical sections for punching shear outside the shear-reinforced zone.

based on a review of experiments by [Islam and Park \(1976\)](#); [Hawkins et al. \(1975\)](#); [Dilger and Brown \(1995\)](#); [Dilger and Cao \(1991\)](#); [Megally and Ghali \(2000a\)](#); and [Megally \(1998\)](#). In these experiments, flat plate-column connections with different amounts of stirrups or HSSR sustained drift ratios higher than 0.025 without a punching shear failure. [Appendix A](#) summarizes the results of these experiments.

The minimum shear reinforcement recommended increases the shear strength and ensures that the flat plate-column connections can support factored gravity loads after experiencing inelastic deformations due to cyclic drift during an earthquake. Experiments ([Megally 1998](#)) have shown that this can be achieved by the provision of shear reinforcement, as recommended herein. The design of shear reinforcement, other than the minimum amount, is discussed in [Chapter 7](#).

The structural integrity reinforcement defined in Section 5.5 of [ACI 421.3R-15](#), $\sum A_{sb}$ should also be provided to increase the resistance of the structural system to progressive collapse.

$$\sum A_{sb} = 2V_{se}/f_y \quad (4b)$$

where $\sum A_{sb}$ is the total area of reinforcing steel passing through the column core, summed on all four sides of the column of an interior column; V_{se} is the unfactored shear force that is not less than twice the unfactored dead load shear; f_y is the specified yield stress of reinforcement. This

reinforcement can exceed that required by the detailing rules in [ACI 318-14](#) Sections 8.7.4.2 and 8.7.5.6.

The crucifix layout of shear reinforcement in Fig. 4, the equations and the detailing in the present guide and in [ACI 421.1R](#) are based on extensive testing and analyses (see reference list). Radial layouts, requiring different set of equations and detailing rules, are deemed neither necessary nor optimum ([Dam et al. 2017](#) and its discussions).

CHAPTER 5—ASSESSMENT OF DUCTILITY

The methods for increasing punching shear strength of flat plate-column connections include increasing the flat plate thickness, improving the material properties, providing shear reinforcement, or a combination of these methods. Ductility is affected by the method used to enhance shear strength (Fig. 5).

The most common types of shear reinforcement in flat plates are the vertical legs of stirrups and headed shear stud reinforcement (HSSR). Stirrups may not be as effective as HSSR in flat plates because of their inefficient anchorage unless they are closed and detailed in accordance with [ACI 318-14](#) Section R8.7.6, which also states: “Anchorage of shear reinforcement according to the requirements of 25.7.1 is difficult in slabs thinner than 10 in. [250 mm].” With HSSR, the anchorage is provided mechanically by forged heads or by a forged head at one end and a steel strip (called a rail) welded at the other end.

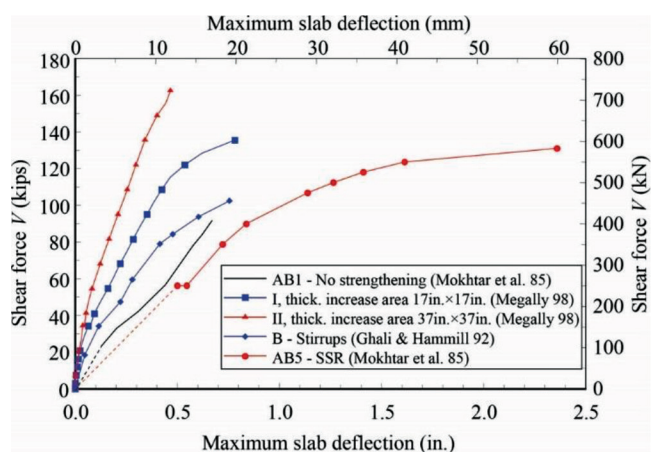


Fig. 5—Load-deflection curves of slabs with different means of increasing punching shear strength. Slabs subjected only to shear force V with no moment. The deflection is measured in the direction of V (Megally and Ghali 2000a).

Figure 5 (Megally and Ghali 2000a) shows load-deflection responses for five slabs with a thickness of 6 in. (150 mm). The tested specimens, with plan dimensions of 75 x 75 in. (1.9 x 1.9 m), represent full-size connections of slabs with interior columns 10 x 10 in. (250 x 250 mm) and square panels of spans equal to 16 ft (4.8 m). Axial load is monotonically applied on the column, transferring a shear force, but no moment M_{sc} to the slab, which is simply supported on four edges. The slabs had the same flexural reinforcement layout and properties; the flexural reinforcement ratio is equal to 0.014 in each of the two orthogonal directions. The concrete compressive strengths f'_c ranged from 4200 to 5800 psi (29 to 40 MPa). The slabs differed only in the method used to increase punching shear strength. Control Slab AB1 (Mokhtar et al. 1985) has thickness 6 in. (150 mm) and no shear reinforcement. Figure 5 includes Slabs I and II (Megally 1998) having shear caps in which slab thickness increases from 6 to 9 in. (150 to 225 mm) over areas, 17 x 17 in. (430 x 430 mm) and 37 x 37 in. (950 x 950 mm), respectively, with the column at their centers. Slab B (Ghali and Hammill 1992) has closed stirrups, detailed according to ACI 318-14 Fig. R8.7.6(c). (It is noted that since 2002, ACI 318 has permitted stirrups only in slabs with $d \geq 6$ in. [150 mm].) Slab AB5 (Mokhtar et al. 1985) has HSSR. The nominal shear strengths, expressed in stress units, provided by the shear reinforcement (Eq. (4)) in Specimens B and AB5, are: 440 psi (3.0 MPa) for Specimen B, based on measured f_{yt} , and 270 psi (1.9 MPa) for Specimen AB5, based on nominal f_{yt} . The corresponding nominal shear strengths provided by shear reinforcement, $V_s = v_s b_o d = A_s f_{yt} d / s$ are 110 and 70 kip (480 and 320 kN). Punching shear failure occurred at a section within the shear-reinforced zone in Specimen B and at a section outside the shear-reinforced zone in Specimen AB5. The extent of the shear reinforcement zones from column faces was $3.8d$ and $5.7d$ in Specimens B and AB5, respectively. The initial portions of the experimental curves shown in Fig. 5 for Slabs AB1 and AB5

are unavailable. The conclusions drawn from these curves, however, are unaffected.

Figure 5 compares the ductility and strength provided by shear reinforcement—stirrups or headed studs—without considering the effect of varying their amount or detailing. Figure 5 indicates that the punching shear strength is increased slightly with the presence of stirrups; however, increases in slab thickness and HSSR provided a more significant increase in punching shear strength. The specimens with increased slab thickness, Slabs I and II, experienced a brittle failure with small slab deflections compared with the slab with HSSR, for which the failure may be considered ductile. Stirrups are assumed more efficient when used in slabs thicker than 10 in. (250 mm), as suggested in ACI 318-14 Section R8.7.6; however, no experimental data on slabs with this thickness could be found. Figure 5 and results of experiments conducted on flat plate-column connections subjected to cyclic moment reversals (Dilger and Brown 1995; Dilger and Cao 1991; Megally and Ghali 2000a,b) indicate that HSSR effectively increases punching shear strength for earthquake-resistant flat plate-column connections even when the slab is relatively thin. Note that Fig. 5, taken from Megally and Ghali (2000a), includes Slabs I and II having drop caps; however, this guide is limited to flat plates.

CHAPTER 6—DESIGN MOMENT TRANSFERRED BETWEEN SLAB AND COLUMN

Moments are transferred between flat plates and their supporting columns as the flat plate-column connections experience the lateral displacement induced on the lateral-force-resisting system. Thus, flat plate-column connections should have a punching shear strength able to resist the factored shear force V_u and factored moment $M_{sc,u}$ due to gravity loads combined with the moments associated with the design story drift during an earthquake. The methods in Sections 6.1 and 6.2 are recommended for the calculation of the moment value of $M_{sc,u}$.

6.1—Frame analysis

A flat plate-column frame, including its lateral-force-resisting system, can be analyzed for gravity and lateral seismic loads using linear frame analysis techniques (Ghali et al. 2003). Unless the layout of columns is highly irregular, the moments of inertia of the slab and its supporting columns are determined according to the equivalent frame method of ACI 318. A static analysis can be performed using the seismic lateral forces specified by ASCE/SEI 7; alternatively, a dynamic analysis may be performed as permitted by the two references.

To account for the effect of cracking in nonprestressed flat plates, Vanderbilt and Corley (1983) recommend considering the moment of inertia of the slab as equal to one-third the value of the uncracked slab strip—from panel centerline to panel centerline—to obtain a conservative estimate of the story drift. ACI 318-14, Section R6.3.1.1 and Table 6.6.3.1.1(b), recommend the use of the uncracked moment of inertia of the slab to compute the

elastic story drift δ_e . According to [ASCE/SEI 7](#), δ_e is related to the maximum design story drift δ_u , including inelastic deformations by

$$\delta_u = \delta_e \left(\frac{C_d}{I_e} \right) \quad (6.1)$$

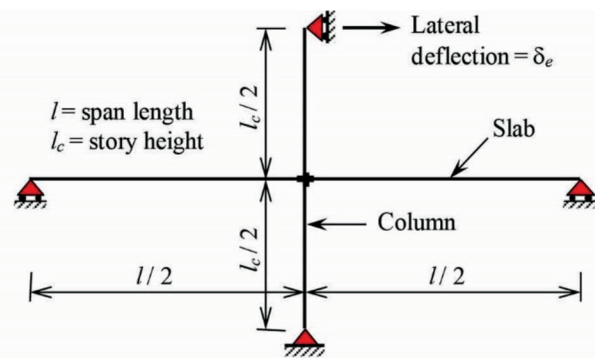
where C_d ($= 1.25$ to 6.5) and I_e ($= 1.0$ to 1.5) are dimensionless factors specified by ASCE/SEI 7, depending on the inherent inelastic deformability of the lateral-force-resisting system and the occupancy importance of the structure, respectively. Section 12.8.7 of ASCE/SEI 7 gives a procedure to include the P - Δ effect on story drifts; the procedure employs a stability coefficient, which equals the inverse of the factor inside the brackets in Eq. (6.1) multiplied by an appropriate coefficient.

Flat plate-column frames not designated as part of the lateral-force-resisting system experience the same lateral displacements as those of the lateral-force-resisting system. Thus, flat plate-column connections should be designed to transfer shears and moments associated with the δ_u values obtained from Eq. (6.1). Accurate determination of shears and moments associated with δ_u is not possible, but for design purposes, the simplified approach presented in Section 6.2 is deemed adequate. The design moments and shears for flat plate-column connections should be determined from the elastic frame analysis according to the ASCE/SEI 7 static lateral force analysis method. The moment $M_{sc,u}$ caused by the factored vertical forces that exist during an earthquake should be added to the moments obtained by the analysis discussed in Section 6.2.

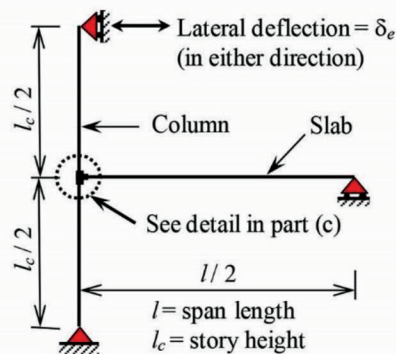
To avoid underestimation of the moments transferred between the slab and the columns for a given value of δ_e , the moments should be determined by an elastic analysis, but with the moment of inertia of the slab equal to 50 percent of the value of the uncracked slab and using the values of the moments of inertia of the uncracked columns (ACI 318-14 Section R6.3.1.1). There is no consensus on the 50 percent value in this recommendation; a one-third value has also been recommended.

6.2—Simplified elastic analysis

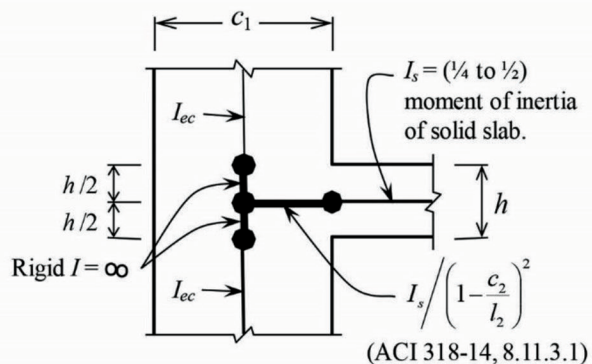
Instead of the analysis suggested in Section 6.1, the moments transferred between the slab and the columns under the elastic story drift δ_e can be determined by a linear elastic analysis of simplified equivalent frames as shown in Fig. 6.2(a) and (b) for interior and exterior columns (edge or corner), respectively. The slab is assumed to be simply supported at locations of contraflexure lines assumed at midspan. The column is assumed to have hinged supports at contraflexure points assumed approximately at midheight of the story. For the frame models shown in Fig. 6.2, it is assumed that the lengths of spans adjacent to the column are equal to l and the story heights above and below the considered level are equal to l_c . The moments of inertia of the column and the slab are calculated according to ACI 318-14 Sections 6.3.1.1 and 8.11.5 (considering torsional members



(a) Interior slab-column connection.



(b) *Edge or corner slab-column connection.*



I_{ec} = equivalent moment of inertia of uncracked column with a torsional member.

c_2 and l_2 are column and span length in direction perpendicular to the plane of the frame.

(c) Detail at slab-column connection in the equivalent plane frame.

Fig. 6.2—Plane frame idealization of flat plate-column connections.

connecting the column to the slab). Figure 6.2(c) indicates I_s and I_{ec} of members. For use in a plane-frame analysis, the modified column moment of inertia, I_{ec} , should be used. A derivation and discussion of I_{ec} are given in [Gamble \(1999\)](#).

The horizontal displacement δ_e is introduced at the upper ends of the columns as shown in the figure. The value of δ_e , representing the elastic story drift, can be estimated by any rational analysis; for example, it can be calculated using an elastic analysis of the lateral force-resisting system