

ACI 352.1R-11

Guide for Design of Slab-Column Connections in Monolithic Concrete Structures

Reported by Joint ACI-ASCE Committee 352



American Concrete Institute®

Guide for Design of Slab-Column Connections in Monolithic Concrete Structures

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Reported by Joint ACI-ASCE Committee 352

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This guide provides recommendations for determining proportions and details of monolithic reinforced and post-tensioned concrete slab-column connections.

Included are recommendations regarding appropriate uses of slab-column connections in structures resisting gravity and lateral forces; procedures for determination of connection load-carrying capacity; and reinforcement details to achieve adequate strength, ductility, and structural integrity. Recommendations are based

on a review of the literature for ultimate and serviceability limit states. A commentary is provided to clarify the recommendations and identify reference material. Design recommendations are set in standard type. Commentary is set in italics.

Keywords: connection; flat plate; flat slab; joint; lateral drift; post-tensioned; punching shear; seismic; shear reinforcement; slab-column.

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CHAPTER 1—INTRODUCTION AND SCOPE**1.1—Introduction**

The recommendations in this guide are for determining connection proportions and details to provide adequate performance of cast-in-place reinforced concrete (RC) and post-tensioned concrete (PT) slab-column connections. The recommendations are written to satisfy serviceability, strength, and ductility requirements related to the intended functions of the connection.

Design of the connection between a slab and its supporting member requires consideration of both the joint and the portion of the slab, or slab and beams, immediately adjacent to the joint. Several connection failures associated with inadequate performance of the slab adjacent to the joint have been reported (Engineering News-Record (ENR) 1956,

1971, 1973; Joint ACI-ASCE Committee 426 1974; Leyendecker and Fattal 1977; Lew et al. 1982a,b; Rosenblueth and Meli 1986; Freyermuth 1989; Moehle 1996; Hueste and Wight 1997). However, no reported cases of connection failure due to distress within the joint have been identified. Some connection failures have occurred during construction when young concrete slabs received loads from more than one floor as a consequence of shoring and reshoring (Agarwal and Gardner 1974; Lew et al. 1982a,b; Sbarounis 1984; ACI 347-05). The disastrous consequences of some failures, including total collapse of the structure, emphasize the importance of the design of the connection. These recommendations are intended to alert the designer to those aspects of behavior that should be considered in design of the connection and to suggest design procedures that will lead to adequate connection performance.

1.2—Scope

Information and design recommendations have been summarized by Joint ACI-ASCE Committee 426 (1974, 1977). This guide is an update of ACI 352.1R-89 (Joint ACI-ASCE Committee 352 1989), based on research information presented in references such as Moehle (1996); Moehle et al. (1988); Kang and Wallace (2005); ACI 318-08, Chapter 21; and Cheng et al. (2010). Modifications to the previous report include expanding the coverage to include slab-column connections with shear reinforcement, slab-column connections with post-tensioning reinforcement, and lateral drift capacity of both RC and PT slab-column connections.

These recommendations apply only to slab-column connections in monolithic concrete structures, with or without drop panels or column capitals, and using normal-weight or lightweight concrete. For strength calculation purposes, the specified concrete compressive strength should not be taken greater than 6000 psi (42 MPa). The recommendations are limited to slab-column connections with slab thickness ranging between 5 and 12 in. (125 and 300 mm); a slab span-to-thickness ratio varying from 20 to 45, except for slab-column connections with transverse beams; and a ratio of long-to-short cross-sectional column dimensions less than 4. The recommendations for PT slab-column connections are applicable only for monolithic concrete connections with unbonded post-tensioning tendons applying an average compressive stress in the concrete not less than 125 psi (0.86 MPa). Construction that combines slab-column and beam-column framing in orthogonal directions at individual connections is included, but these recommendations are limited to issues related to the transfer of loads in the direction perpendicular to the beam axis. Slab-column framing systems are considered inappropriate as seismic-force-resisting systems assigned to high seismic design categories, but they are commonly used as frames not designated as part of the seismic-force-resisting system along with a stiffer seismic-force-resisting system, such as shear walls or beam-and-column moment-resisting frames.

These recommendations are limited to slab-column connections of cast-in-place RC and PT floor-to-column, including two-way ribbed floor slab construction (Meli and

Rodriguez 1979) and slab-column connections with transverse beams. Recommendations are made elsewhere (ACI 352R-02) for connections in which framing is predominantly by action between beams and columns.

The recommendations do not consider slab-wall connections, precast connections, or slabs-on-ground. Relevant information on these subjects may be found in ACI 360R-10, Schwaighofer and Collins (1977), Paulay and Taylor (1981), and Klemencic et al. (2006). Although structures having specified concrete compressive strength exceeding 6000 psi (42 MPa) are within the realm of this guide, the recommendations limit the assumed compressive strength for design to 6000 psi (42 MPa) because of the lack of test data on slab-column connections with higher-strength concrete.

Slab-column systems are generally inadequate as the seismic-force-resisting system of multi-story buildings assigned to high seismic design categories because of problems associated with excessive lateral drift and inadequate shear and moment transfer capacity at the connections. For high seismic design categories, if designed according to these recommendations, slab-column systems may be used in multi-story construction in which earthquake-induced lateral forces are primarily carried by a stiffer seismic-force-resisting system. For low and moderate seismic design categories, slab-column systems may be adequate as the seismic-force-resisting system, provided the connection design recommendations in this guide are met.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A_b	= area of individual bar or wire, in. ² (mm ²)	b_o	= perimeter of critical section for shear in slabs, in. (mm)
A_c	= cross-sectional area of shear-critical section, in. ² (mm ²)	C_v	= product of all appropriate modification factors in Table 5.2.1.1
A_{cf}	= larger gross cross-sectional area of slab-beam strips of two orthogonal equivalent frames intersecting at a column of a two-way slab, in. ² (mm ²)	c_1	= dimension of rectangular or equivalent rectangular column or capital measured in direction of span for which moments are being determined, in. (mm)
A_s	= area of non-prestressed longitudinal tension reinforcement, in. ² (mm ²)	c_2	= dimension of rectangular or equivalent rectangular column or capital measured in direction perpendicular to c_1 , in. (mm)
A_{sm}	= minimum area of effectively continuous bottom slab bars, in each principal direction, placed over support for resistance to progressive collapse, in. ² (mm ²)	c_t	= distance from interior face of column to slab edge measured parallel to c_1 , but not exceeding c_1 , in. (mm)
$A_{s,min}$	= minimum non-post-tensioned top reinforcement, in. ² (mm ²)	DR	= maximum story-drift ratio
A_v	= cross-sectional area of all legs of reinforcement on one peripheral line that is geometrically similar to perimeter of column section, in. ² (mm ²)	d	= slab effective depth, taken as average of depths from extreme concrete compression fiber to centroid of tension steel in two orthogonal directions, in. (mm), not to be taken greater than $0.8h$
b	= width of compression face of member, in. (mm)	d_b	= nominal diameter of bar, wire, or prestressing strand, in. (mm)
b_1	= dimension of critical section b_o measured in direction of span for which moments are determined, in. (mm)	d_{beam}	= effective depth of transverse beam at connection, in. (mm)
b_2	= dimension of critical section b_o measured in direction perpendicular to b_1 , in. (mm)	f'_c	= specified compressive strength of concrete, psi (MPa)
		f_{pc}	= average precompression stress in two orthogonal directions after losses, psi (MPa)
		f_y	= specified yield strength of reinforcement, psi (MPa)
		f_{yt}	= specified yield strength of shear reinforcement, psi (MPa); not to be taken greater than 60,000 psi (420 MPa)
		h	= slab thickness, in. (mm)
		$h_{(column)}$	= column dimension parallel to bar being developed, in. (mm)
		ℓ_1	= length of span in direction that moments are being determined, measured center-to-center of supports, in. (mm)
		ℓ_2	= length of span in direction perpendicular to ℓ_1 , measured center-to-center of supports, in. (mm)
		ℓ_d	= development length of straight bar, in. (mm)
		ℓ_{dh}	= development length in tension of deformed bar or deformed wire with standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter), in. (mm)
		ℓ_{dt}	= development length in tension of headed deformed bar, measured from critical section to outside of head, in. (mm)
		ℓ_v	= length of shearhead arm from column center, in. (mm)
		M_{ub}	= moment transferred to column, in.-lb (N-mm)

M_{ub1}, M_{ub2}	= simultaneous moments transferred to column and acting about centroidal principal axes of shear-critical section, in.-lb (N-mm)
s	= spacing between peripheral lines of shear reinforcement, in. (mm)
V_c	= shear strength of concrete without modifications in Table 5.2.1.1, lb (N)
V_{max}	= limiting value of allowed shear resistance, lb (N)
V_n	= nominal shear strength, lb (N)
V_o	= design shear strength in absence of moment transfer, lb (N)
V_s	= nominal shear strength provided by shear reinforcement, lb (N)
V_u	= factored direct shear force acting on shear-critical section, lb (N)
V_{ug}	= factored shear force acting on shear-critical section for two-way action due to gravity loads, corresponding to load combination specified in ACI 318-08, Section 21.13.6, lb (N)
VR	= gravity shear ratio
w_u	= factored uniformly distributed load, but not less than two times the slab dead load, to be considered for resistance to progressive collapse, lb/ft ² (N/mm ²)
α	= coefficient in Eq. (5.2.1.2c)
α_s	= coefficient in Eq. (5.2.1.1b)
β	= ratio of long-to-short dimensions of column cross section
β_p	= coefficient in Eq. (5.2.1.1c)
ϵ_t	= net tensile strain in extreme tension steel at nominal strength
ϕ	= strength reduction factor
γ_f	= factor used to determine unbalanced moment transferred by flexure at slab-column connections (ACI 318)
γ_v	= factor used to determine unbalanced moment transferred by eccentricity of shear at slab-column connections (ACI 318)
ρ	= reinforcement ratio of top slab reinforcement in one direction at the connection
ρ'	= reinforcement ratio of bottom slab reinforcement in one direction at the connection

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, “ACI Concrete Terminology” (<http://terminology.concrete.org>). Definitions provided herein complement that resource.

column—a vertical supporting element with a ratio of height-to-least lateral dimension exceeding 3 and with a ratio of long-to-short cross-sectional dimensions not exceeding 3.

column capital—a flared portion of the column immediately below the slab and having effective plan dimensions assumed equal to the smaller of the actual dimensions and the part of the capital lying within the largest right circular

cone or pyramid with a 90-degree vertex that can be included within the outlines of the supporting column.

column strip—design strip with a width on each side of a column centerline equal to one-fourth of the span length transverse or parallel to the span, whichever is less. Column strip includes beams, if any.

connection—the joint plus adjacent regions of the slab and beams.

design story-drift ratio—the relative difference between design lateral displacements for the top and bottom of a story, divided by the story height.

direct shear—shear force transferred from slab to column.

drop panel—a projection below the slab at the connection having thickness not less than one-fourth of the adjacent slab thickness and extending from the centerline of support in each direction not less than one-sixth of the span length measured from center-to-center of supports in that direction.

joint—the part of the column within the depth of the slab, including drop panel or shear cap, and having plan dimensions equal to those of the column at the intersection between the column and the bottom surface of the slab (drop panel or shear cap).

shear cap—a projection below the slab at the connection extending a minimum horizontal distance from the column face that is equal to the thickness of the projection below the slab soffit.

shear-critical section—a slab section assumed to extend around and near a column (at a connection) and at which shear capacity must be evaluated. A critical section has a depth d perpendicular to the plane of the slab and extending around the column. A critical section should be considered around the column so that its perimeter b_o is a minimum, but it need not approach closer than lines at $d/2$ from the column face and parallel to the column boundaries. The critical section should be considered both within and outside the shear-reinforced region. For the purpose of defining the shear-critical section, a support of circular cross section may be replaced by a square support having an equal perimeter.

shear reinforcement—reinforcement provided to resist shear or diagonal tension stresses and increase the shear strength of the connection.

transfer moment—the unbalanced slab moment transferred to the supporting element at a connection.

CHAPTER 3—CONNECTION CLASSIFICATIONS

3.1—General

Connection performance, particularly during an earthquake, is affected by joint behavior, including slip of reinforcement embedded in the joint, and by the region of the slab, or slab and transverse beams, surrounding the joint. The joint definition is illustrated in Fig. 3.1a. In general, the region of slab that directly affects connection behavior extends from the joint face the greater of twice the development length of the largest slab bars or four times the slab thickness (Joint ACI-ASCE Committee 426 1977). A drop panel is used to reduce the minimum required slab thickness or the amount of top slab reinforcement over a column.

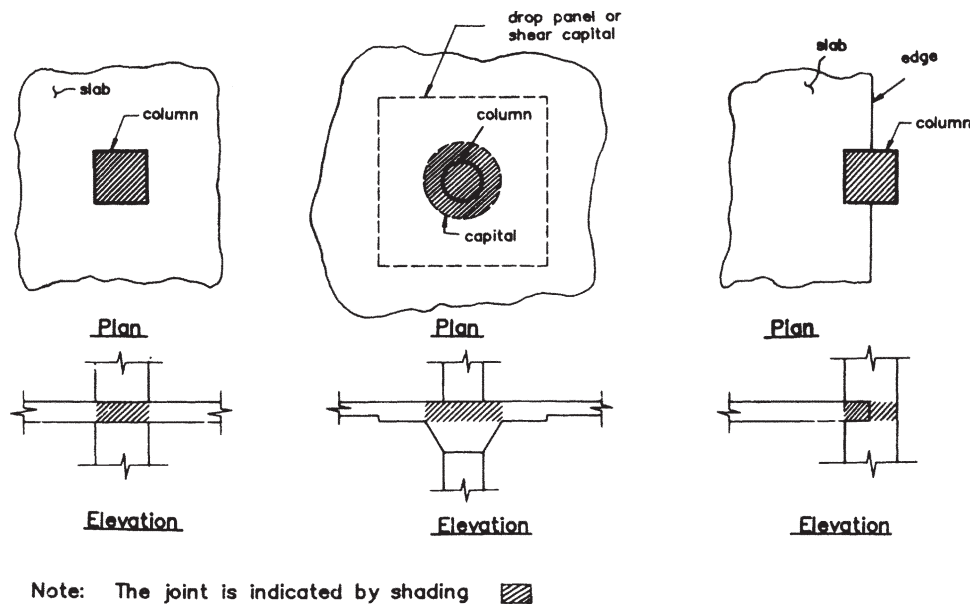


Fig. 3.1a—Joint in typical slab-column connections.

and to increase the connection shear-critical section and consequently the shear strength. A shear cap is used solely to increase the shear strength at the connection.

The shear-critical section, used for determination of connection shear strength, is the same as that specified in ACI 318, although the definition has been modified to clarify that shear-critical sections for non-rectangular supports may be assumed to have a rectangular shape. The shear-critical sections for several support geometries are shown in Fig. 3.1b, including a typical rectangular interior connection (Fig. 3.1b(a)) and an interior connection with irregular geometry (Fig. 3.1b(b)). Punching shear strengths for connections with circular columns have been observed (Vanderbilt 1972) to exceed the punching shear strengths for connections with square columns having the same perimeter; thus, it is conservative to represent circular columns by square columns having the same perimeter (Fig. 3.1b(c)). Two critical sections are defined for connections with drop panels or shear caps because failure may occur either through the thickened portion of the slab near the column or through the slab outside the drop panel or shear cap (Fig. 3.1b(d)). Shear-critical sections for exterior connections depend on the slab edge distance from the face of the column, as shown in Fig. 3.1b(e) and Fig. 3.1b(f). Alternative critical sections should be investigated at other locations that might result in reduced shear strength of a connection with shear reinforcement, as shown in Fig. 3.1b(g) and Fig. 3.1b(h) (Corley and Hawkins 1968; Hawkins and Corley 1974; Hanna et al. 1975).

The limitation on the aspect ratio of the column cross-section dimensions is illustrated in Fig. 3.1c. As the aspect ratio increases, behavior deviates from a column-slab connection behavior to a slab-wall connection behavior (Schwaighofer and Collins 1977). No recommendations for such connections are made in this guide. For more information about slab-wall connections, refer to Schwaighofer and

Collins (1977), Paulay and Taylor (1981), and Klemencic et al. (2006).

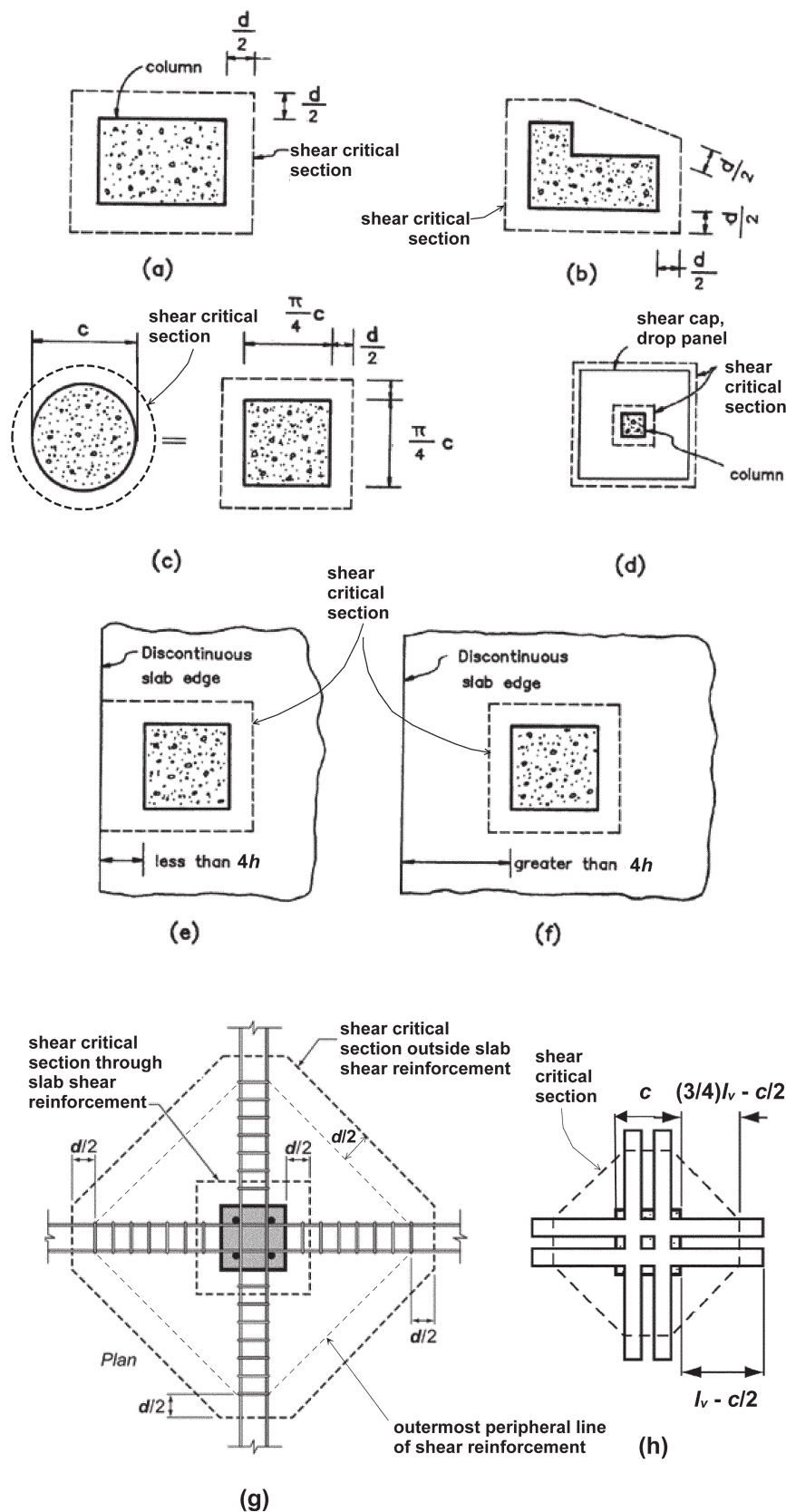
3.2—Connection classifications

Connections are classified according to geometry in Section 3.2.1 and according to anticipated performance in Section 3.2.2.

3.2.1 A slab-column connection is an exterior connection if the distance from any discontinuous edge to the nearest support face is less than four times the slab thickness. An edge connection is an exterior connection for which a discontinuous edge is located adjacent to one support face only. A corner connection is an exterior connection for which discontinuous edges are located adjacent to two adjoining support faces. A vertical slab opening located closer than four times the slab thickness to the support face should be classified as a discontinuous edge if radial lines projecting from the centroid of the support area to the boundaries of the opening enclose a length of the shear-critical section that exceeds the adjacent support dimension. A connection not defined as an exterior connection is considered an interior connection. Regardless of the connection classification, the reduced shear-critical section should be investigated for openings up to 10 times the slab thickness away from the support face.

Openings or slab edges located close to the support interrupt the shear flow in the slab, induce moment transfer to supports, reduce available anchorage lengths, and reduce the effective joint confinement. The distance of four times the slab thickness is based on considerations related to strength of the slab near the support (Joint ACI-ASCE Committee 426 1977). Examples of edge connections, corner connections, exterior connections, and interior connections with openings are shown in Fig. 3.2.1.

When classifying a connection as interior or exterior, the effect of openings on the shear-critical section should be



Note: h = slab thickness; d = slab effective depth;
 l_v = length of shearhead arm from the column center

For exterior connections, the shear critical section should extend to the slab edge as shown in (e) if the distance from any discontinuous edge to the nearest support face is less than $4h$ and if such extension will reduce the critical section perimeter. Otherwise, the shear critical section is as shown in (f).

Fig. 3.1b—Examples of shear-critical sections.

Note: The recommendations apply only if $1/3 \leq c_1 / c_2 \leq 3$

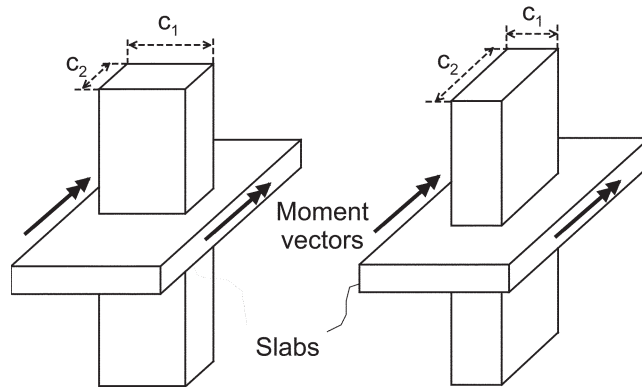


Fig. 3.1c—Limitation on column aspect ratio.

considered. Where large openings are located closer than four times the slab thickness to the support face, the connection may behave as an exterior connection, depending on the size and proximity of the opening. To approximately gauge the effect of the opening, radial lines are drawn from the centroid of the support area to the boundaries of the opening (Fig. 3.2.1(e) and Fig. 3.2.1(f)). If the length of the shear-critical section enclosed within the radial lines exceeds the adjacent support dimension, the connection is classified as an exterior connection. Regardless of connection classification, however, it is conservatively recommended (ACI 318-08, Section 11.11.6) that openings within 10 times the slab thickness to a support face be evaluated using a reduced shear-critical section, even though this scenario would seldom control when considered using traditional approaches for slab openings (Joint ACI-ASCE Committee 326 (later 426) 1962). The aforementioned method of classification for an exterior connection should not be used when applying Section 5.2.1.2(b). In the preceding, if there are no shear caps, a support should be interpreted as being the column plus the column capital, if present. If there are shear caps or drop panels, the effect of the opening should first be checked considering the column as the support, and then secondly considering the shear cap or drop panel as the support.

Where distances from the support face to openings and free edges exceed the aforementioned limits, the connection may be defined as interior. In such cases, the diameter of the longitudinal bars should be limited so that adequate development is available between the column and the opening or edge. Recommendations given elsewhere (Joint ACI-ASCE Committee 426 1977) suggest that bars should be selected so that the development length for a straight bar in tension from the column face is less than half the distance from the column face to the edge or opening.

3.2.2 A connection is classified as either Type 1 or Type 2 depending on the loading conditions of the connection as follows:

(a) **Type 1:** A connection between elements that are designed to satisfy ACI 318 strength, ductility, and service-

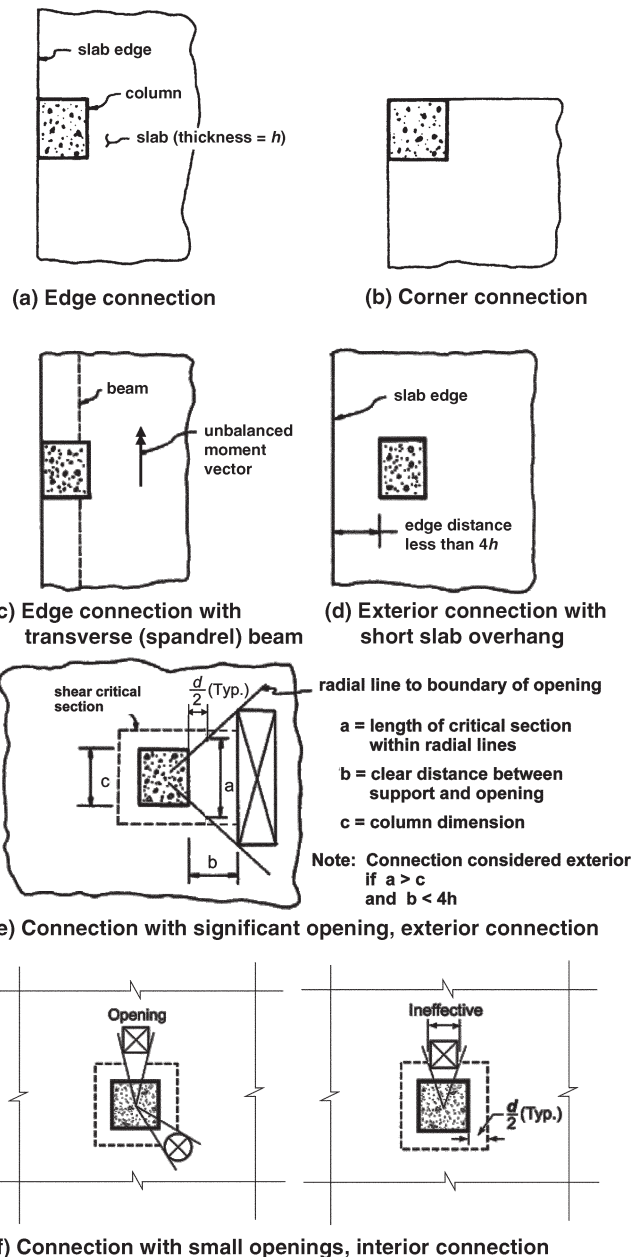


Fig. 3.2.1—Examples of exterior connections and connections with openings.

ability requirements, excluding Chapter 21, and that are not subjected to earthquake-induced inelastic deformations.

(b) **Type 2:** A connection between elements that are designed to satisfy ACI 318 strength, ductility, and serviceability requirements, including Chapter 21. Type 2 connections that are not part of the seismic-force-resisting system are required to maintain gravity-load-carrying capacity under moderate-to-significant inelastic deformations. Type 2 connections that are part of the seismic-force-resisting system are required to maintain sufficient strength to resist earthquake-induced force demands in addition to gravity loads under the presence of inelastic deformations.

The design recommendations for connections depend on the deformations implied for the design loading conditions. A Type 1 connection is a connection in a struc-

ture designed to resist gravity and lateral forces without deformations into the inelastic range for expected forces. Type 1 connections are typically defined as being part of an “ordinary” structural system or as not being part of the lateral-force-resisting system in an “intermediate” structural system. Some local yielding of slab reinforcement may be acceptable for Type 1 connections. Slabs designed by conventional yield-line methods may be included in this category, except if required to resist loads as described for Type 2 connections.

A Type 2 connection is a connection between members that may undergo significant deformations into the inelastic range without punching failure. Typical examples of Type 2 connections are connections in intermediate moment-resisting frames or frames not designated as part of the seismic-force-resisting system designed according to ACI 318-08, Chapter 21. For structures assigned to high seismic design categories, a slab-column connection should be classified as Type 2 even though it may not be considered during design as part of the seismic-force-resisting system.

CHAPTER 4—DESIGN CONSIDERATIONS

4.1—Connection performance

The connection should be proportioned for serviceability and ultimate limit states to resist the actions and deformations specified in this chapter.

4.2—Types of actions on the connection

4.2.1 The design of the connection should account for simultaneous effects of axial forces, shears, bending moments, and torsion applied to the connection as a consequence of self-weight, externally applied loads, ground motions, creep, shrinkage, temperature, and foundation movements. Loads occurring during construction, including post-tensioning, and during the service life should be considered.

The connection should be designed for the forces and deformations due to self-weight, externally applied loads, ground motions, post-tensioning, and time-dependent and temperature effects where they are significant. Unexpected foundation movements and differential settlements may affect the connection’s structural integrity. Effects of construction loads and early concrete strengths are of particular importance for slabs without beams, as demonstrated by several catastrophic failures during construction (ENR 1956, 1971, 1973; Leyendecker and Fattal 1977; ACI 347-05). Effects of heavy construction equipment and of shoring and reshoring (Grundy and Kabaila 1963; Agarwal and Gardner 1974; Liu et al. 1985) should be considered. Special loading conditions should be evaluated during the construction stage, by a licensed design professional retained by the contractor, to avoid exceeding the load-carrying capacity of the slab-column connection as designed by the engineer-of-record. Effects of simultaneous bidirectional moment transfer should be considered in design of the connection, except that seismic lateral forces are generally not considered to act

simultaneously along both axes of the slab-column connection for punching shear design.

4.2.2 Moment transfer about any of the two principal axes should be included in evaluating connection resistance. The moment should be taken about the centroidal principal axes of the shear-critical section defined in Section 3.2.

Moment transfer at a connection can increase the shear demand placed on a slab-column connection.

4.3—Determination of connection forces

4.3.1 Forces on the connection may be determined by any method satisfying requirements of equilibrium and geometric compatibility for the structure. Time-dependent effects should be evaluated.

4.3.2 For normal gravity loads, Section 4.3.1 may be satisfied using the Direct Design Method, the Equivalent Frame Method of ACI 318-08, the Effective Slab Width Method, or the Finite Element Method of ASCE/SEI 41-06. For uniformly loaded slabs with nearly equal spans, with no more than 20 percent difference, slab shears at the connection may be determined for loads within a tributary area bounded by panel centerlines; slab shears at first interior supports should not be taken less than 1.2 times the tributary area values unless a compatibility analysis shows lower values are appropriate.

The design should account for the worst combinations of actions at the connection. Analysis for connection forces should consider at least (a) loads producing the maximum slab shear on the shear-critical section; and (b) loads producing the maximum transfer moment at the shear-critical section.

Factored slab shear at the connection can be determined by procedures such as yield line and strip design methods (Johansen 1962; Park and Gamble 1980) and the equivalent frame method. However, in typical designs, simpler procedures such as the use of tributary areas are acceptable. The designer is cautioned, when using simplified procedures, that the actual shear at first interior supports is likely to be as much as 20 percent higher than the shears calculated using the tributary area (Hatcher et al. 1965; Criswell 1972) because of continuity effects. For cases with unequal spans, this increase may exceed 20 percent.

4.3.3 For lateral loads, effects of cracking, compatibility, and $P-\Delta$ effects should be considered. The lateral displacements may be determined in accordance with ACI 318-08, Section 8.8, and ASCE/SEI 41-06. The design story-drift ratio is due to the design lateral forces, wind or seismic, and must consider realistic lateral forces, $P-\Delta$ effects, inelastic deformations in members, and foundation movements, which can significantly affect lateral drift.

Cracking in the connection has been shown (Vanderbilt and Corley 1983; Mulcahy and Rotter 1983; Moehle and Diebold 1985) to reduce lateral-load stiffness below the stiffness calculated by elastic plate theory (Vanderbilt and Corley 1983; Darvall and Allen 1984). The reduction in stiffness can result in calculated lateral drift exceeding that anticipated by an uncracked elastic analysis. The effects of gravity loads on the deformed structure ($P-\Delta$ effects) are