CHAPTER 18—EARTHQUAKE-RESISTANT STRUCTURES

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18.1—Scope

R18.1—Scope

Chapter 18 does not apply to structures assigned to Seismic Design Category (SDC) A. For structures assigned to SDC B and C, Chapter 18 applies to structural systems designated as part of the seismic-force-resisting system. For structures assigned to SDC D through F, Chapter 18 applies to both structural systems designated as part of the seismic-force-resisting system and structural systems not designated as part of the seismic-force-resisting system.

Chapter 18 contains provisions considered to be the minimum requirements for a cast-in-place or precast concrete structure capable of sustaining a series of oscillations into the inelastic range of response without critical deterioration in strength. The integrity of the structure in the inelastic range of response should be maintained because the design earthquake forces defined in documents such as ASCE/SEI 7, the 2021 IBC, the UBC (ICBO 1997), and the NEHRP (FEMA P-749) provisions are considered less than those corresponding to linear response at the anticipated earthquake intensity (FEMA P-749; Blume et al. 1961; Clough 1960; Gulkan and Sozen 1974).

The design philosophy in Chapter 18 is for cast-in-place concrete structures to respond in the nonlinear range when subjected to design-level ground motions, with decreased stiffness and increased energy dissipation but without critical strength decay. Precast concrete structures designed in accordance with Chapter 18 are intended to emulate cast-in-place construction, except 18.5, 18.9.2.3, and 18.11.2.2, which permit precast construction with alternative yielding mechanisms. The combination of reduced stiffness and increased energy dissipation tends to reduce the response accelerations and lateral inertia forces relative to values that would occur were the structure to remain linearly elastic and lightly damped (Gulkan and Sozen 1974). Thus, the use of design forces representing earthquake effects such as those in ASCE/SEI 7 requires that the seismic-force-resisting system retain a substantial portion of its strength into the inelastic range under displacement reversals.

The provisions of Chapter 18 relate detailing requirements to type of structural framing and SDC. Seismic design categories are adopted directly from ASCE/SEI 7, and relate to considerations of seismic hazard level, soil type, occupancy, and use. Before the 2008 Code, low, intermediate, and high seismic risk designations were used to delineate detailing requirements. For a qualitative comparison of seismic design categories and seismic risk designations, refer to Table R5.2.2. The assignment of a structure to a SDC is regulated by the general building code (refer to 4.4.6.1).

18.1.1 This chapter shall apply to the design of nonprestressed and prestressed concrete structures assigned to Seismic Design Categories (SDC) B through F, including, where applicable:

(a) Structural systems designated as part of the seismic-force-resisting system, including diaphragms, moment frames, structural walls, and foundations

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(b) Members not designated as part of the seismic-force-resisting system but required to support other loads while undergoing deformations associated with earthquake effects

18.1.2 Structures designed according to the provisions of this chapter are intended to resist earthquake motions through ductile inelastic response of selected members.

18.2—General

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R18.2—General

Structures assigned to SDC A need not satisfy requirements of Chapter 18 but must satisfy all other applicable requirements of the Code. Structures assigned to Seismic Design Categories B through F must satisfy requirements of Chapter 18 in addition to all other applicable requirements of the Code.

Sections 18.2.1.3 through 18.2.1.5 identify those parts of Chapter 18 that apply to the building based on its assigned SDC, regardless of the vertical elements of the seismic-force-resisting system. ASCE/SEI 7 defines the permissible vertical elements of the seismic-force-resisting system and applies where adopted. The remaining commentary of R18.2 summarizes the intent of ACI CODE-318 regarding which vertical elements should be permissible in a building considering its SDC. Section 18.2.1.6 defines the requirements for the vertical elements of the seismic-force-resisting system.

The design and detailing requirements should be compatible with the level of inelastic response assumed in the calculation of the design earthquake forces. The terms “ordinary,” “intermediate,” and “special” are used to facilitate this compatibility. For any given structural element or system, the terms “ordinary,” “intermediate,” and “special,” refer to increasing requirements for detailing and proportioning, with expectations of increased deformation capacity. Structures assigned to SDC B are not expected to be subjected to strong ground motion, but instead are expected to experience low levels of ground motion at long time intervals. The Code provides some requirements for beam-column ordinary moment frames to improve deformation capacity.

Structures assigned to SDC C may be subjected to moderately strong ground motion. The designated seismic-force-resisting system typically comprises some combination of ordinary cast-in-place structural walls, intermediate precast structural walls, and intermediate moment frames. The general building code also may contain provisions for use of other seismic-force-resisting systems in SDC C. Provision 18.2.1.6 defines requirements for whatever system is selected.

Structures assigned to SDC D, E, or F may be subjected to strong ground motion. It is the intent of ACI Committee 318 that the seismic-force-resisting system of structural concrete buildings assigned to SDC D, E, or F be provided by special moment frames, special structural walls, and, in limited conditions, intermediate precast walls. In addition to 18.2.2 through 18.2.8, these structures also are required to satisfy applicable requirements for continuous inspection (26.13.1.3), diaphragms and trusses (18.12), foundations (18.13), gravity-

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load-resisting elements that are not designated as part of the seismic-force-resisting system (18.14), and regions of the seismic-force-resisting system designed with the strut-and-tie method (23.11). These provisions have been developed to provide the structure with adequate deformation capacity for the high demands expected for these seismic design categories.

The general building code may also permit the use of intermediate moment frames as part of dual systems for some buildings assigned to SDC D, E, or F. It is not the intent of ACI Committee 318 to recommend the use of intermediate moment frames as part of moment-resisting frame or dual systems in SDC D, E, or F. The general building code may also permit substantiated alternative or nonprescriptive designs or, with various supplementary provisions, the use of ordinary or intermediate systems for nonbuilding structures in the higher seismic design categories. These are not the typical applications that were considered in the writing of this chapter, but wherever the term "ordinary or intermediate moment frame" is used in reference to reinforced concrete, 18.3 or 18.4 apply.

Table R18.2 summarizes the applicability of the provisions of Chapter 18 as they are typically applied when using the minimum requirements in the various seismic design categories. Where special systems are used for structures in SDC B or C, it is not required to satisfy the requirements of 18.14, although it should be verified that members not designated as part of the seismic-force-resisting system will be stable under design displacements.

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Table R18.2—Sections of Chapter 18 to be satisfied in typical applications[1]

Component resisting earthquake effect, unless otherwise noted	SDC			
	A (None)	B (18.2.1.3)	C (18.2.1.4)	D, E, F (18.2.1.5)
Analysis and design requirements	None	18.2.2	18.2.2	18.2.2, 18.2.4
Materials		None	None	18.2.5 through 18.2.8
Frame members		18.3	18.4	18.6 through 18.9
Structural walls and coupling beams		None	None	18.10
Precast structural walls		None	18.5	18.5[2], 18.11
Diaphragms and trusses		None	18.12.1.2	18.12
Foundations		None	18.13	18.13
Frame members not designated as part of the seismic-force-resisting system		None	None	18.14
Anchors		None	18.2.3	18.2.3

[1]In addition to requirements of Chapters 1 through 17, 19 through 26, and ACI CODE-318.2, except as modified by Chapter 18. Section 14.1.3 also applies in SDC D, E, and F.
[2]As permitted by the general building code.

The proportioning and detailing requirements in Chapter 18 are based predominantly on field and laboratory experience with monolithic reinforced concrete building structures and precast concrete building structures designed and detailed to behave like monolithic building structures. Extrapolation of these requirements to other types of cast-in-place or precast concrete structures should be based on evidence provided by field experience, tests, or analysis. The acceptance criteria for moment frames given in ACI CODE-374.1 can be used in conjunction with Chapter 18 to demonstrate that the strength, energy dissipation capacity, and deformation capacity of a proposed frame system equals or exceeds that provided by a comparable monolithic concrete system. ACI CODE-550.6 provides similar information for precast wall systems.

The toughness requirement in 18.2.1.7 refers to the requirement to maintain structural integrity of the entire seismic-force-resisting system at lateral displacements anticipated for the maximum considered earthquake motion. Depending on the energy-dissipation characteristics of the structural system used, such displacements may be larger than for a monolithic reinforced concrete structure satisfying the prescriptive provisions of other parts of the Code.

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18.2.1 *Structural systems*

18.2.1.1 All structures shall be assigned to a SDC in accordance with 4.4.6.1.

18.2.1.2 All members shall satisfy Chapters 1 to 17 and 19 to 26. Structures assigned to SDC B, C, D, E, or F also shall satisfy 18.2.1.3 through 18.2.1.7, as applicable. Where Chapter 18 conflicts with other chapters of this Code, Chapter 18 shall govern.

18.2.1.3 Structures assigned to SDC B shall satisfy 18.2.2.

18.2.1.4 Structures assigned to SDC C shall satisfy 18.2.2, 18.2.3, 18.12.1.2, and 18.13.

18.2.1.5 Structures assigned to SDC D, E, or F shall satisfy 18.2.2 through 18.2.8, 18.12 through 18.14, and 23.11.

18.2.1.6 Structural systems designated as part of the seismic-force-resisting system shall be restricted to those designated by the general building code, or determined by other authority having jurisdiction in areas without a legally adopted building code. Except for SDC A, for which Chapter 18 does not apply, (a) through (h) shall be satisfied for each structural system designated as part of the seismic-force-resisting system, in addition to 18.2.1.3 through 18.2.1.5:

(a) Ordinary moment frames shall satisfy 18.3
(b) Ordinary reinforced concrete structural walls need not satisfy any detailing provisions in Chapter 18, unless required by 18.2.1.3 or 18.2.1.4
(c) Intermediate moment frames shall satisfy 18.4
(d) Intermediate precast walls shall satisfy 18.5
(e) Special moment frames shall satisfy 18.2.3 through 18.2.8 and 18.6 through 18.8
(f) Special moment frames constructed using precast concrete shall satisfy 18.2.3 through 18.2.8 and 18.9
(g) Special structural walls shall satisfy 18.2.3 through 18.2.8 and 18.10
(h) Special structural walls constructed using precast concrete shall satisfy 18.2.3 through 18.2.8 and 18.11

18.2.1.7 A reinforced concrete structural system not satisfying this chapter shall be permitted if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable reinforced concrete structure satisfying this chapter.

18.2.2 *Analysis and proportioning of structural members*

COMMENTARY

R18.2.2 *Analysis and proportioning of structural members*

It is assumed that the distribution of required strength to the various components of a seismic-force-resisting system will be determined from the analysis of a linearly elastic model of the system acted upon by the factored forces, as

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required by the general building code. If nonlinear response history analyses are to be used, base motions should be selected after a detailed study of the site conditions and local seismic history.

Because the basis for earthquake-resistant design admits nonlinear response, it is necessary to investigate the stability of the seismic-force-resisting system, as well as its interaction with other structural and nonstructural members, under expected lateral displacements corresponding to maximum considered earthquake ground motion. For lateral displacement calculations, assuming all the structural members to be fully cracked is likely to lead to better estimates of the possible drift than using uncracked stiffness for all members. The analysis assumptions described in 6.6.3.1.2 may be used to estimate lateral deflections of reinforced concrete building systems.

The main objective of Chapter 18 is the safety of the structure. The intent of 18.2.2.1 and 18.2.2.2 is to draw attention to the influence of nonstructural members on structural response and to hazards from falling objects.

Section 18.2.2.3 serves as an alert that the base of structure as defined in analysis may not necessarily correspond to the foundation or ground level. Details of columns and walls extending below the base of structure to the foundation are required to be consistent with those above the base of structure.

In selecting member sizes for earthquake-resistant structures, it is important to consider constructibility problems related to congestion of reinforcement. The design should be such that all reinforcement can be assembled and placed in the proper location and that concrete can be cast and consolidated properly. Using the upper limits of permitted reinforcement ratios may lead to construction problems.

18.2.2.1 The interaction of all structural and nonstructural members that affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

18.2.2.2 Rigid members assumed not to be a part of the seismic-force-resisting system shall be permitted provided their effect on the response of the system is considered in the structural design. Consequences of failure of structural and nonstructural members that are not a part of the seismic-force-resisting system shall be considered.

18.2.2.3 Structural members extending below the base of structure that are required to transmit forces resulting from earthquake effects to the foundation shall comply with the requirements of Chapter 18 that are consistent with the seismic-force-resisting system above the base of structure.

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18.2.3 *Anchoring to concrete*

18.2.3.1 Anchors resisting earthquake-induced forces in structures assigned to SDC C, D, E, or F shall be in accordance with 17.10.

18.2.4 *Strength reduction factors*

18.2.4.1 Strength reduction factors shall be in accordance with Chapter 21.

18.2.5 *Concrete in special moment frames and special structural walls*

18.2.5.1 Specified compressive strength of concrete in special moment frames and special structural walls shall be in accordance with the special seismic systems requirements of 19.2.1.

18.2.6 *Reinforcement in special moment frames and special structural walls*

18.2.6.1 Reinforcement in special moment frames and special structural walls shall be in accordance with the special seismic systems requirements of 20.2.2.

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R18.2.4 *Strength reduction factors*

R18.2.4.1 Chapter 21 contains strength reduction factors for all members, joints, and connections of earthquake-resistant structures, including specific provisions in 21.2.4 for buildings that use special moment frames, special structural walls, and intermediate precast walls.

R18.2.5 *Concrete in special moment frames and special structural walls*

Requirements of this section refer to concrete quality in frames and walls that resist earthquake-induced forces. The maximum specified compressive strength of lightweight concrete to be used in structural design calculations is limited to 5000 psi, primarily because of paucity of experimental and field data on the behavior of members made with lightweight concrete subjected to displacement reversals in the nonlinear range. If convincing evidence is developed for a specific application, the limit on maximum specified compressive strength of lightweight concrete may be increased to a level justified by the evidence.

R18.2.6 *Reinforcement in special moment frames and special structural walls*

R18.2.6.1 Nonprestressed reinforcement for seismic systems is required to meet 20.2.2.4 and 20.2.2.5. ASTM A706 Grades 60, 80 and 100 reinforcement is permitted to resist moments, axial, and shear forces in special structural walls and all components of special structural walls, including coupling beams and wall piers. ASTM A706 Grades 60 and 80 reinforcement is permitted in special moment frames. Results of tests and analytical studies presented in NIST (2014) and Sokoli and Ghannoum (2016) indicate that properly detailed beams and columns of special moment frames with ASTM A706 Grade 80 reinforcement exhibit strength and deformation capacities similar to those of members reinforced with Grade 60 reinforcement. The use of Grade 100 reinforcement is not allowed in special moment frames because there is insufficient data to demonstrate satisfactory seismic performance.

To allow the use of ASTM A706 Grade 80 and 100 reinforcement, the 2019 Code includes limits for spacing of transverse reinforcement to provide adequate longitudinal bar support to control longitudinal bar buckling. In special

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moment frames, the use of Grade 80 reinforcement requires increased joint depths to prevent excessive slip of beam bars passing through beam-column joints (18.8.2.3).

The requirement in 20.2.1.3(b) for a tensile strength greater than the yield strength of the reinforcement is based on laboratory tests demonstrating adequate deformation capacities in structural components with reinforcement satisfying this requirement.

The restrictions on the value of f_{yt} apply to all types of transverse reinforcement, including spirals, circular hoops, rectilinear hoops, and crossties. Research results (Budek et al. 2002; Muguruma and Watanabe 1990; Sugano et al. 1990) indicate that higher yield strengths can be used effectively as confinement reinforcement as specified in 18.7.5.4. The increases to 80,000 psi and 100,000 psi for shear design of some special seismic system members is based on research indicating the design shear strength can be developed (Wallace 1998; Aoyama 2001; Budek et al. 2002; Sokoli and Ghannoum 2016; Cheng et al. 2016; Huq et al. 2018; Weber-Kamin et al. 2020). The 60,000 psi restriction on the value of f_{yt} for deformed bar in 20.2.2.4 for calculating nominal shear strength is intended to limit the width of shear cracks at service-level loads. Service-level cracking is not a concern in members of the seismic-force-resisting system subjected to design-level earthquake forces.

18.2.7 *Mechanical splices in special moment frames and special structural walls*

18.2.7.1 Mechanical splices shall conform to 25.5.7 and the requirements of this section.

18.2.7.2 Mechanical splices shall satisfy (a) through (d):

(a) Mechanical splices shall be Class G or Class S.

(b) Class S mechanical splices shall be permitted at any location, except as noted in 18.9.2.1(c)

(c) Class G mechanical splices in special moment frames are prohibited within joints, within a distance equal to twice the member depth from the column or beam face for special moment frames, and within a distance equal to twice the member depth from critical sections where yielding of the reinforcement is likely to occur as a result of lateral displacements beyond the linear range of behavior.

R18.2.7 *Mechanical splices in special moment frames and special structural walls*

R18.2.7.1 The 2025 Code consolidated the requirements for all classes of mechanical splices into 25.5.7, with mechanical splice classifications designated as Class L, Class G, and Class S replacing the classifications and requirements used in prior Code editions. The requirements for each class of mechanical splice are described in R25.5.7.2. In a structure undergoing inelastic deformations during an earthquake, tensile strains in reinforcing bars may approach the specified minimum uniform elongation. The requirements specified in Table 25.5.7.2 for Class S mechanical splices are intended to avoid premature failure under inelastic cyclic effects on the mechanical splicing device and the bars being mechanically spliced.

R18.2.7.2 Class L and Class G mechanical splices are not required to be capable of resisting the considerable strain levels that may occur in yielding regions due to earthquake loading, nor are these particular classes of mechanical splices required to resist inelastic cyclic loading. As a result, in special seismic systems, Class L mechanical splices are not permitted and locations where Class G mechanical splices may be used are restricted. These restrictions apply to all reinforcement resisting earthquake effects, including transverse reinforcement.

Regarding the use of Class L mechanical splices in ordinary and intermediate moment frames and structural walls,

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(d) Class G mechanical splices in special structural walls are prohibited where lap splices of longitudinal reinforcement in boundary regions are prohibited by 18.10.2.3(c), within coupling beams, and within a distance equal to twice the member depth from critical sections where yielding of the reinforcement is likely to occur as a result of lateral displacements beyond the linear range of behavior.

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including coupling beams, the requirements of 25.5.7.2(a) preclude the use of Class L mechanical splices in yielding regions of these systems, resulting in location restrictions similar to those of 18.2.7.2(c) and (d).

Recommended detailing practice precludes use of mechanical or lapped bar splices in regions of potential yielding in members resisting earthquake effects. If use of mechanical splices in regions of potential yielding cannot be avoided, Class S mechanical splices are required. Documentation required by 26.6.1.2(b) should be provided to confirm the mechanical splice used in construction meets the Class S requirements as specified in 25.5.7.2.

For staggering of mechanical splices, refer to R25.5.7.4.

18.2.8 *Welded splices in special moment frames and special structural walls*

18.2.8.1 Welded splices are not permitted in special moment frames or in special structural walls, including coupling beams.

18.2.8.2 Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement required by design shall not be permitted.

R18.2.8 *Welded splices in special moment frames and special structural walls*

R18.2.8.1 Welded splices are similar to Class L mechanical splices. Therefore, welded splices are not permitted in special seismic systems because reinforcement tension strains and stresses in yielding regions can exceed the requirements of 25.5.7.3(a), which does not establish minimum tensile strain or inelastic cyclic endurance requirements for welded splices. The restriction on welded splices applies to all reinforcement resisting earthquake effects, including transverse reinforcement.

Regarding the use of welded splices in ordinary and intermediate moment frames and structural walls, including coupling beams, the requirement of 25.5.7.3(c) precludes the use of welded splices in yielding regions of these systems, resulting in location restrictions similar to those of Class G mechanical splices in 18.2.7.2(c) and 18.2.7.2(d).

R18.2.8.2 Welding of crossing reinforcing bars can lead to local embrittlement of the steel. If welding of crossing bars is used to facilitate fabrication or placement of reinforcement, it should be done only on bars added for such purposes. The prohibition of welding crossing reinforcing bars does not apply to bars that are welded with automated welding operations under continuous, competent control, as in the manufacture of welded-wire reinforcement.

18.3—Ordinary moment frames

R18.3—Ordinary moment frames

This section applies only to ordinary moment frames assigned to SDC B. The provisions for beam reinforcement are intended to improve continuity in the framing members and thereby improve lateral force resistance and structural integrity; these provisions do not apply to slab-column moment frames. The provisions for columns are intended to provide additional capacity to resist shear for columns with proportions that would otherwise make them more susceptible to shear failure under earthquake loading.

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18.3.1 *Scope*

18.3.1.1 This section shall apply to ordinary moment frames forming part of the seismic-force-resisting system.

18.3.2 Beams shall have at least two continuous bars at both top and bottom faces. Continuous bottom bars shall have area not less than one-fourth the maximum area of bottom bars along the span. These bars shall be developed in tension in accordance with 25.4 by substituting a bar stress of $1.25f_y$ for f_y at the face of support.

18.3.3 Columns having unsupported length $\ell_u \leq 5c_1$ shall have ϕV_n at least the lesser of (a) and (b):

(a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.

(b) The maximum shear obtained from design load combinations that include E, with $\Omega_o E$ substituted for E.

18.3.4 Beam-column joints shall satisfy Chapter 15 with joint shear V_u calculated on a plane at mid-height of the joint using tensile and compressive beam forces and column shear consistent with beam nominal moment strengths M_n.

18.4—Intermediate moment frames

18.4.1 *Scope*

18.4.1.1 This section shall apply to intermediate moment frames including two-way slabs without beams forming part of the seismic-force-resisting system.

18.4.2 *Beams*

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R18.4—Intermediate moment frames

The objective of the requirements in 18.4.2.3 and 18.4.3.1 is to reduce the risk of failure in shear in beams and columns during an earthquake. Two options are provided to determine the factored shear force.

R18.4.2 *Beams*

According to 18.4.2.3(a), the factored shear force is determined from a free-body diagram obtained by cutting through the beam ends, with end moments assumed equal to the nominal moment strengths acting in reverse curvature bending, both clockwise and counterclockwise. Figure R18.4.2 demonstrates only one of the two options that are to be considered for every beam. To determine the maximum beam shear, it is assumed that its nominal moment strengths (ϕ = 1.0 for moment) are developed simultaneously at both ends of its clear span. As indicated in Fig. R18.4.2, the shear associated with this condition $[(M_{n\ell} + M_{nr})/\ell_n]$ is added algebraically to the shear due to the factored gravity loads and vertical earthquake effects to obtain the design shear for the

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beam. For the example shown, dead load, live load, and snow load have been assumed to be uniformly distributed. The figure also shows that vertical earthquake effects are to be included, as is typically required by the general building code. For example, ASCE/SEI 7 requires vertical earthquake effects, $0.2S_{DS}$, to be included.

Provision 18.4.2.3(b) bases V_u on the load combination including the earthquake effect E, which should be doubled. For example, the load combination defined by Eq. (5.3.1.e) would be

$$U = 1.2D + 2.0E + 1.0L + 0.2S$$

where E is the value specified by the general building code. The factor of 1.0 applied to L is allowed to be reduced to 0.5 in accordance with 5.3.3.

Transverse reinforcement at the ends of the beam is required to be hoops. In most cases, transverse reinforcement required by 18.4.2.3 for the design shear force will be more than those required by 18.4.2.4.

Beams may be subjected to axial compressive force due to prestressing or applied loads. The additional requirements in 18.4.2.6 are intended to provide lateral support for beam longitudinal reinforcement.

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18.4.2.1 Beams shall have at least two continuous bars at both top and bottom faces. Continuous bottom bars shall have area not less than one-fourth the maximum area of bottom bars along the span. These bars shall be developed in tension in accordance with 25.4 by substituting a bar stress of $1.25f_y$ for f_y at the face of support.

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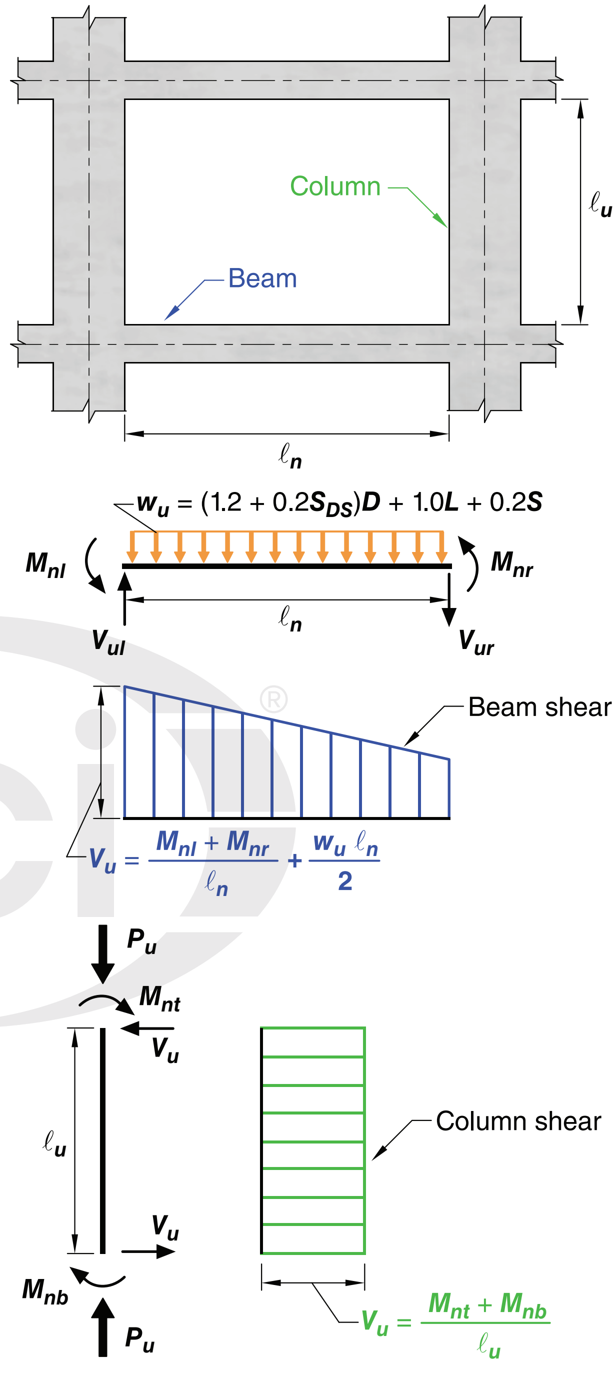

Fig. R18.4.2—Design shears for intermediate moment frames.

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18.4.2.2 The positive moment strength at the face of the joint shall be at least one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

18.4.2.3 ϕV_n shall be at least the lesser of (a) and (b):

(a) The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity and vertical earthquake loads

(b) The maximum shear obtained from design load combinations that include E, with E taken as twice that prescribed by the general building code

18.4.2.4 At both ends of the beam, hoops or closed stirrups in accordance with 18.6.4.3 shall be provided over a length of at least twice the beam depth measured from the face of the supporting member toward midspan. The first hoop or closed stirrup shall be located not more than 2 in. from the face of the supporting member. Spacing of hoops or closed stirrups shall not exceed the smallest of (a) through (d):

(a) $d/4$

(b) Eight times the diameter of the smallest longitudinal bar enclosed

(c) 24 times the diameter of the transverse reinforcing bar

(d) 12 in.

18.4.2.5 Transverse reinforcement spacing shall not exceed $d/2$ throughout the length of the beam.

18.4.2.6 In beams having factored axial compressive force exceeding $A_g f_c'/10$, transverse reinforcement required by 18.4.2.5 shall conform to 25.7.2.2 and either 25.7.2.3 or 25.7.2.4.

18.4.3 *Columns*

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R18.4.3 *Columns*

According to 18.4.3.1(a), the factored shear force is determined from a free-body diagram obtained by cutting through the column ends, with end moments assumed equal to the nominal moment strengths acting in reverse curvature bending, both clockwise and counterclockwise. Figure R18.4.2 demonstrates only one of the two options that are to be considered for every column. The factored axial force P_u should be chosen to develop the largest moment strength of the column within the range of design axial forces. Provision 18.4.3.1(b) for columns is similar to 18.4.2.3(b) for beams except it bases V_u on load combinations including the earthquake effect E, with E increased by the overstrength factor Ω_o rather than the factor 2.0. In ASCE/SEI 7, $\Omega_o = 3.0$ for intermediate moment frames. The higher factor for columns relative to beams is because of greater concerns about shear failures in columns.

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18.4.3.1 ϕV_n shall be at least the lesser of (a) and (b):

(a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength

(b) The maximum shear obtained from factored load combinations that include E, with $\Omega_o E$ substituted for E

18.4.3.2 Columns shall be spirally reinforced in accordance with Chapter 10 or shall be in accordance with 18.4.3.3 through 18.4.3.5. Provision 18.4.3.6 shall apply to all columns supporting discontinuous stiff members.

18.4.3.3 At both ends of the column, hoops shall be provided at spacing s_o over a length ℓ_o measured from the joint face. Spacing s_o shall not exceed the least of (a) through (c):

(a) For Grade 60, the smaller of $8d_b$ of the smallest longitudinal bar enclosed and 8 in.

(b) For Grade 80, the smaller of $6d_b$ of the smallest longitudinal bar enclosed and 6 in.

(c) One-half of the smallest cross-sectional dimension of the column

Length ℓ_o shall not be less than the longest of (d), (e), and (f):

(d) One-sixth of the clear span of the column

(e) Maximum cross-sectional dimension of the column

(f) 18 in.

18.4.3.4 The first hoop shall be located not more than $s_o/2$ from the joint face.

18.4.3.5 Outside of length ℓ_o, spacing of transverse reinforcement shall be in accordance with 10.7.6.5.2.

18.4.3.6 Columns supporting reactions from discontinuous stiff members, such as walls, shall be provided with transverse reinforcement at the spacing s_o in accordance with 18.4.3.3 over the full height beneath the level at which the discontinuity occurs if the portion of factored axial compressive force in these members related to earthquake effects exceeds $A_g f_c'/10$. If design forces have been magnified to

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Transverse reinforcement at the ends of columns is required to be spirals or hoops. The amount of transverse reinforcement at the ends must satisfy both 18.4.3.1 and 18.4.3.2. Note that hoops require seismic hooks at both ends. The maximum spacing allowed for hoops is intended to inhibit or delay buckling of longitudinal reinforcement.

Discontinuous structural walls and other stiff members can impose large axial forces on supporting columns during earthquakes. The required transverse reinforcement in 18.4.3.6 is to improve column toughness under anticipated demands. The factored axial compressive force related to earthquake effect should include the factor Ω_o if required by the general building code.

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account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $A_g f_c'/10$ shall be increased to $A_g f_c'/4$. Transverse reinforcement shall extend above and below the column in accordance with 18.7.5.6(b).

18.4.4 *Joints*

18.4.4.1 Beam-column joints shall satisfy the detailing requirements of 15.7.1.2, 15.7.1.3, and 18.4.4.2 through 18.4.4.5.

18.4.4.2 If a beam framing into the joint and generating joint shear has depth exceeding twice the column depth, analysis and design of the joint shall be based on the strut-and-tie method in accordance with Chapter 23 and (a) and (b) shall be satisfied:

(a) Design joint shear strength determined in accordance with Chapter 23 shall not exceed ϕV_n calculated in accordance with 15.5.

(b) Detailing requirements of 18.4.4.3 through 18.4.4.5 shall be satisfied.

18.4.4.3 Longitudinal reinforcement terminated in a joint shall extend to the far face of the joint core and shall be developed in tension in accordance with 18.8.5.

18.4.4.4 Spacing of joint transverse reinforcement s shall not exceed the lesser of 18.4.3.3(a) through (c) within the height of the deepest beam framing into the joint.

18.4.4.5 Where the top beam longitudinal reinforcement consists of headed deformed bars that terminate in the joint, the column shall extend above the top of the joint a distance at least the depth h of the joint. Alternatively, the beam reinforcement shall be enclosed by additional vertical joint reinforcement providing equivalent confinement to the top face of the joint.

18.4.4.6 Slab-column joints shall satisfy transverse reinforcement requirements of 15.7.2. Where slab-column joint transverse reinforcement is required, at least one layer of joint transverse reinforcement shall be placed between the top and bottom slab reinforcement.

18.4.4.7 *Shear strength requirements for beam-column joints*

18.4.4.7.1 Design shear strength of cast-in-place beam-column joints shall satisfy:

$$\phi V_n \geq V_u$$

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R18.4.4 *Joints*

R18.4.4.2 For joints in which the beam depth is significantly greater than the column depth, a diagonal strut between the joint corners may not be effective. Therefore, the Code requires that joints in which the beam depth exceeds twice the column depth be designed using the strut-and-tie method of Chapter 23.

R18.4.4.3 Refer to R18.8.2.2.

R18.4.4.4 The maximum spacing of transverse reinforcement within a joint is consistent with the spacing limits for reinforcement in columns of intermediate moment frames.

R18.4.4.5 Refer to R25.4.4.6.

R18.4.4.7 *Shear strength requirements for beam-column joints*

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18.4.4.7.2 V_u of the joint shall be determined in accordance with 18.3.4.

18.4.4.7.3 ϕ shall be in accordance with 21.2.1 for shear.

18.4.4.7.4 V_n of the joint shall be in accordance with 18.8.4.3.

18.4.5 *Two-way slabs without beams*

18.4.5.1 Factored slab moment at the support including earthquake effects, E, shall be calculated for load combinations given in Eq. (5.3.1e) and (5.3.1g). Reinforcement to resist M_{sc} shall be placed within the column strip defined in 8.4.1.5.

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R18.4.4.7.2 Factored joint shear force is determined assuming that beams framing into the joint develop end moments equal to their nominal moment strengths. Consequently, joint shear force generated by the flexural reinforcement is calculated for a stress of f_y in the reinforcement. This is consistent with 18.4.2 and 18.4.3 for determination of minimum design shear strength in beams and columns of intermediate moment frames.

R18.4.5 *Two-way slabs without beams*

Section 18.4.5 applies to two-way slabs without beams, such as flat plates.

Using load combinations of Eq. (5.3.1e) and (5.3.1g) may result in moments requiring top and bottom reinforcement at the supports.

The moment M_{sc} refers, for a given design load combination with E acting in one horizontal direction, to that portion of the factored slab moment that is balanced by the supporting members at a joint. It is not necessarily equal to the total design moment at the support for a load combination including earthquake effect. In accordance with 8.4.2.2.3, only a fraction of the moment M_{sc} is assigned to the slab effective width. For edge and corner connections, flexural reinforcement perpendicular to the edge is not considered fully effective unless it is placed within the effective slab width (ACI PRC-352.1; Pan and Moehle 1989). Refer to Fig. R18.4.5.1.

Application of the provisions of 18.4.5 is illustrated in Fig. R18.4.5.2 and R18.4.5.3.

R18.4.5.1

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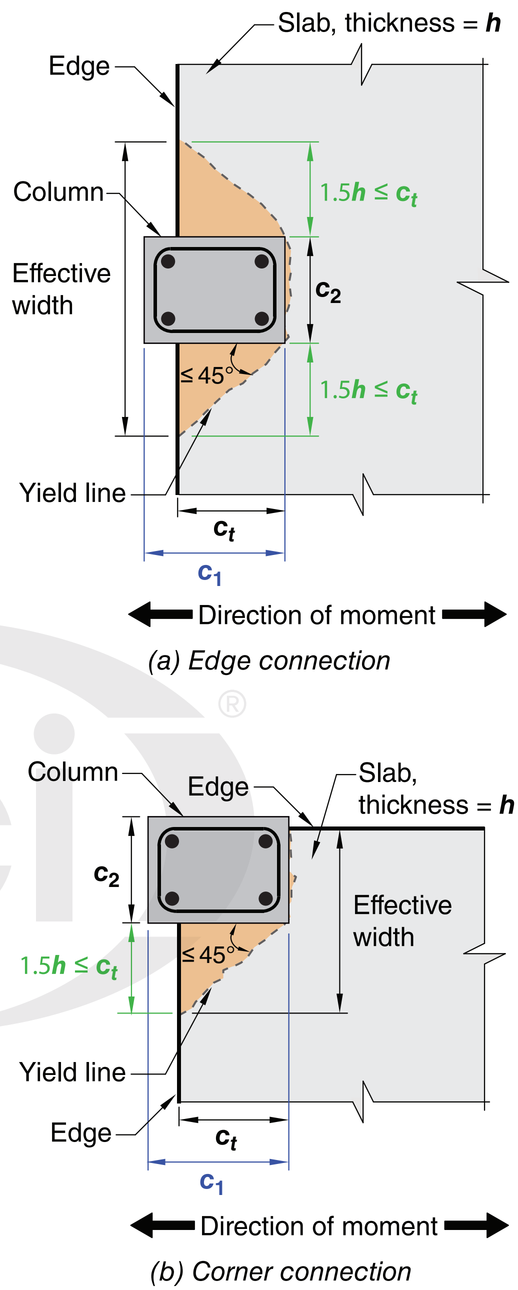

Fig. R18.4.5.1—Effective width for reinforcement placement in edge and corner connections.

18.4.5.2 Reinforcement placed within the effective width given in 8.4.2.2.3 shall be designed to resist $\gamma_f M_{sc}$. Effective slab width for exterior and corner connections shall not extend beyond the column face a distance greater than c_t measured perpendicular to the slab span.

R18.4.5.2

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18.4.5.3 At least one-half of the reinforcement in the column strip at the support shall be placed within the effective slab width given in 8.4.2.2.3.

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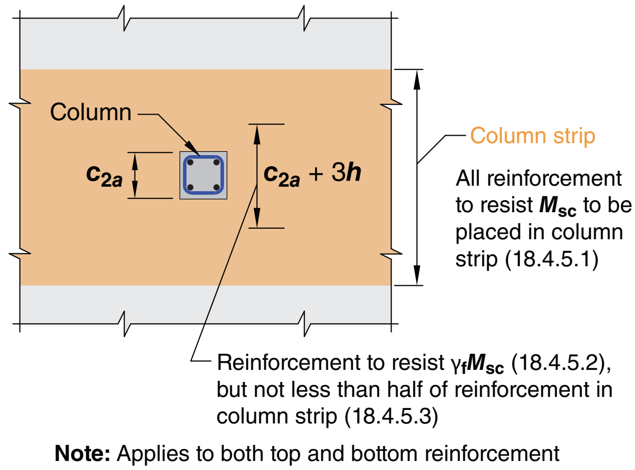

Fig. R18.4.5.2—Location of reinforcement in slabs.

R18.4.5.3

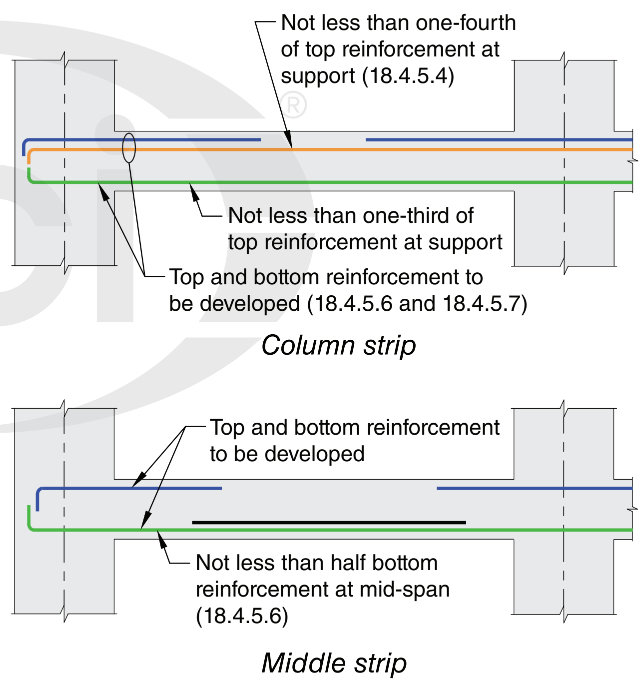

Fig. R18.4.5.3—Arrangement of reinforcement in slabs.

18.4.5.4 At least one-fourth of the top reinforcement at the support in the column strip shall be continuous throughout the span.

18.4.5.5 Continuous bottom reinforcement in the column strip shall be at least one-third of the top reinforcement at the support in the column strip.

18.4.5.6 At least one-half of all bottom middle strip reinforcement and all bottom column strip reinforcement at midspan shall be continuous and shall develop f_y at the face of columns, capitals, brackets, or walls.

18.4.5.7 At discontinuous edges of the slab, all top and bottom reinforcement at the support shall be developed at the face of columns, capitals, brackets, or walls.

18.4.5.8 At the critical sections for columns defined in 22.6.4.1, two-way shear stress caused by factored gravity loads without moment transfer shall not exceed $0.4\phi v_c$ for nonprestressed slab-column connections and $0.5\phi v_c$ for unbonded post-tensioned slab-column connections with f_{pc} in each direction meeting the requirements of 8.6.2.1, where v_c shall be calculated in accordance with 22.6.5. This requirement need not be satisfied if the slab-column connection satisfies 18.14.5.

R18.4.5.8 The requirements apply to two-way slabs that are designated part of the seismic-force-resisting system. Nonprestressed slab-column connections in laboratory tests (Pan and Moehle 1989) exhibited reduced lateral displacement ductility when the shear stress at the column connection exceeded the recommended limit of $0.4\phi v_c$. Based on laboratory test data (Kang and Wallace 2006; Kang et al. 2007), a higher maximum factored gravity shear stress of $0.5\phi v_c$ is allowed for unbonded post-tensioned slab-column connections with f_{pc} in each direction meeting the requirements of 8.6.2.1. Post-tensioned slab-column connections with f_{pc} in each direction not meeting the requirements of 8.6.2.1 can be designed as nonprestressed slab-column connections in accordance with 8.2.3. Slab-column connections also must satisfy shear and moment strength requirements of Chapter 8 under load combinations including earthquake effect.

18.5—Intermediate precast structural walls

R18.5—Intermediate precast structural walls

Connections between precast wall panels or between wall panels and the foundation are required to resist forces induced by earthquake motions and to provide for yielding in the vicinity of connections.

18.5.1 *Scope*

18.5.1.1 This section shall apply to intermediate precast structural walls forming part of the seismic-force-resisting system.

18.5.2 *General*

18.5.2.1 In connections between wall panels, or between wall panels and the foundation, yielding shall be restricted to steel elements or reinforcement. Mechanical splices used as components of the connection shall be Class S.

18.5.2.2 For elements of the connection that are not designed to yield, the required strength shall be based on $1.5S_y$ of the yielding portion of the connection, but need not exceed the strength required from applying factored load combinations that include E_{mh}.

R18.5.2.2 Connection design forces need not exceed the maximum forces that the structural system can deliver to the connection.

18.5.2.3 In structures assigned to SDC D, E, or F, wall piers shall be designed in accordance with 18.10.8 or 18.14.

18.6—Beams of special moment frames

18.6.1 *Scope*

R18.6—Beams of special moment frames

R18.6.1 *Scope*

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COMMENTARY

This section applies to beams of special moment frames resisting lateral loads induced by earthquake motions. In previous Codes, any frame member subjected to a factored axial compressive force exceeding ($A_g f_c'/10$) under any load combination was to be proportioned and detailed as described in 18.7. In the 2014 Code, all requirements for beams are contained in 18.6 regardless of the magnitude of axial compressive force.

The Code is written with the assumption that special moment frames comprise horizontal beams and vertical columns interconnected by beam-column joints. It is acceptable for beams and columns to be inclined provided the resulting system behaves as a frame—that is, lateral resistance is provided primarily by moment transfer between beams and columns rather than by strut or brace action. In special moment frames, it is acceptable to design beams to resist combined moment and axial force as occurs in beams that act both as moment frame members and as chords or collectors of a diaphragm. It is acceptable for beams of special moment frames to cantilever beyond columns, but such cantilevers are not part of the special moment frame that forms part of the seismic-force-resisting system. It is acceptable for beams of a special moment frame to connect into a wall boundary if the boundary is reinforced as a special moment frame column in accordance with 18.7. A concrete braced frame, in which lateral resistance is provided primarily by axial forces in beams and columns, is not a recognized seismic-force-resisting system.

18.6.1.1 This section shall apply to beams of special moment frames that form part of the seismic-force-resisting system and are proportioned primarily to resist flexure and shear.

18.6.1.2 Beams of special moment frames shall frame into columns of special moment frames satisfying 18.7.

18.6.2 *Dimensional limits*

R18.6.2 *Dimensional limits*

Experimental evidence (Hirosawa 1977) indicates that, under reversals of displacement into the nonlinear range, behavior of continuous members having length-to-depth ratios of less than 4 is significantly different from the behavior of relatively slender members. Design rules derived from experience with relatively slender members do not apply directly to members with length-to-depth ratios less than 4, especially with respect to shear strength.

Geometric constraints indicated in 18.6.2.1(b) and (c) were derived from practice and research (ACI PRC-352) on reinforced concrete frames resisting earthquake-induced forces. The limits in 18.6.2.1(c) define the maximum beam width that can effectively transfer forces into the beam-column joint. An example of maximum effective beam width is shown in Fig. R18.6.2.

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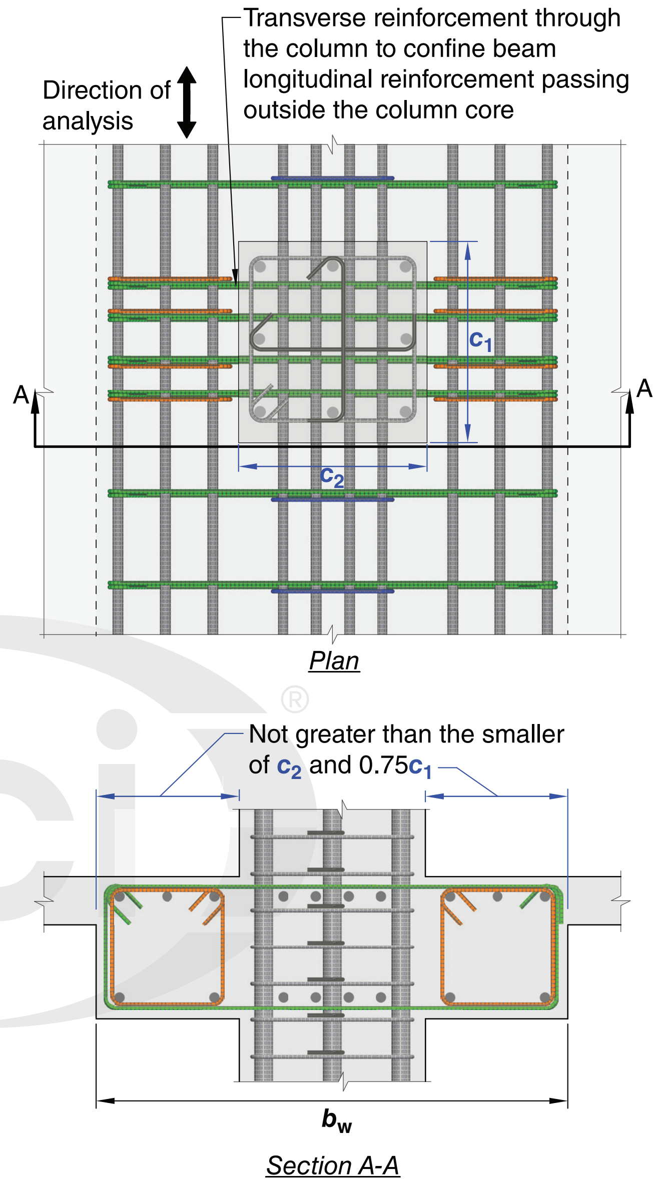

Fig. R18.6.2—Maximum effective width of wide beam and required transverse reinforcement.

18.6.2.1 Beams shall satisfy (a) through (c):

(a) Clear span ℓ_n shall be at least $4d$

(b) Width b_w shall be at least the larger of $0.3h$ and 10 in.

(c) Projection of the beam width beyond the width of the supporting column on each side shall not exceed the smaller of c_2 and $0.7c_1$.

R18.6.2.1 Experimental evidence (Hirosawa 1977) indicates that, under reversals of displacement into the nonlinear range, behavior of continuous members having length-to-depth ratios less than 4 is significantly different from the behavior of relatively slender members. Design rules derived from experience with relatively slender members do not apply directly to members with length-to-depth ratios less than 4, especially with respect to shear strength.

Geometric constraints indicated in 18.6.2.1(b) and (c) were derived from practice and research (ACI PRC-352) on reinforced concrete frames resisting earthquake-induced forces. The limits in 18.6.2.1(c) define the maximum beam width that can effectively transfer forces into the beam-column joint. An example of maximum effective beam width is shown in Fig. R18.6.2.

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18.6.3 *Longitudinal reinforcement*

18.6.3.1 Beams shall have at least two continuous bars at both top and bottom faces. At any section, for top as well as for bottom reinforcement, the amount of reinforcement shall be at least that required by 9.6.1.2, and the reinforcement ratio ρ shall not exceed 0.025 for Grade 60 reinforcement and 0.02 for Grade 80 reinforcement.

18.6.3.2 Positive moment strength at joint face shall be at least one-half the negative moment strength provided at that face of the joint. Both the negative and the positive moment strength at any section along member length shall be at least one-fourth the maximum moment strength provided at face of either joint.

18.6.3.3 Lap splices of deformed longitudinal reinforcement shall be permitted if hoop or spiral reinforcement is provided over the lap length. Spacing of the transverse reinforcement enclosing the lap-spliced bars shall not exceed the lesser of $d/4$ and 4 in. Lap splices shall not be used in locations (a) through (c):

(a) Within the joints

(b) Within a distance of twice the beam depth from the face of the joint

(c) Within a distance of twice the beam depth from critical sections where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior

18.6.3.4 Mechanical splices shall conform to 18.2.7 and welded splices shall conform to 18.2.8.

18.6.3.5 Unless used in a special moment frame as permitted by 18.9.2.3, prestressing shall satisfy (a) through (d):

(a) The average prestress f_{pc} calculated for an area equal to the least cross-sectional dimension of the beam multiplied by the perpendicular cross-sectional dimension shall not exceed the lesser of 500 psi and $f_c'/10$.

(b) Prestressed reinforcement shall be unbonded in potential plastic hinge regions, and the calculated strains in prestressed reinforcement under the design displacement shall be less than 0.01.

(c) Prestressed reinforcement shall not contribute more than one-fourth of the positive or negative flexural strength at the critical section in a plastic hinge region and shall be anchored at or beyond the exterior face of the joint.

(d) Anchorages of post-tensioning tendons resisting earthquake-induced forces shall be capable of allowing tendons to withstand 50 cycles of loading, with prestressed reinforcement forces bounded by 40 and 85% of the specified tensile strength of the prestressing reinforcement.

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R18.6.3 *Longitudinal reinforcement*

R18.6.3.1 The limiting reinforcement ratios of 0.025 and 0.02 are based primarily on considerations of providing adequate deformation capacity, avoiding reinforcement congestion, and, indirectly, on limiting shear stresses in beams of typical proportions.

R18.6.3.3 Lap splices of reinforcement are prohibited along lengths where flexural yielding is anticipated because such splices are not reliable under conditions of cyclic loading into the inelastic range. Transverse reinforcement for lap splices at any location is mandatory because of the potential of concrete cover spalling and the need to confine the splice.

R18.6.3.5 These provisions were developed, in part, based on observations of building performance in earthquakes (ACI PRC-423.3). For calculating the average prestress, the least cross-sectional dimension in a beam normally is the web dimension, and is not intended to refer to the flange thickness. In a potential plastic hinge region, the limitation on strain and the requirement for unbonded tendons are intended to prevent fracture of tendons under inelastic earthquake deformation. Calculation of strain in the prestressed reinforcement is required considering the anticipated inelastic mechanism of the structure. For prestressed reinforcement unbonded along the full beam span, strains generally will be well below the specified limit. For prestressed reinforcement with short unbonded length through or adjacent to the joint, the additional strain due to earthquake deformation is calculated as the product of the depth to the neutral axis and the sum of plastic hinge rotations at the joint, divided by the unbonded length.

The restrictions on the flexural strength provided by the tendons are based on the results of analytical and experimental studies (Ishizuka and Hawkins 1987; Park and Thompson 1977). Although satisfactory seismic perfor-

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mance can be obtained with greater amounts of prestressed reinforcement, this restriction is needed to allow the use of the same response modification and deflection amplification factors as those specified in model codes for special moment frames without prestressed reinforcement. Prestressed special moment frames will generally contain continuous prestressed reinforcement that is anchored with adequate cover at or beyond the exterior face of each beam-column connection located at the ends of the moment frame.

Fatigue testing for 50 cycles of loading between 40 and 80% of the specified tensile strength of the prestressed reinforcement has been a long-standing industry practice (ACI PRC-423.3; ACI SPEC-423.7). The 80% limit was increased to 85% to correspond to the 1% limit on the strain in prestressed reinforcement. Testing over this range of stress is intended to conservatively simulate the effect of a severe earthquake. Additional details on testing procedures are provided in ACI SPEC-423.7.

18.6.4 *Transverse reinforcement*

R18.6.4 *Transverse reinforcement*

Transverse reinforcement is required primarily to confine the concrete and provide lateral support for the reinforcing bars in regions where yielding is expected. Examples of transverse reinforcement suitable for beams are shown in Fig. R18.6.4.

In earlier Code editions, the upper limit on hoop spacing was the least of $d/4$, eight longitudinal bar diameters, 24 tie bar diameters, and 12 in. The upper limits were changed in the 2011 edition because of concerns about adequacy of longitudinal bar buckling restraint and confinement in large beams.

In the case of members with varying strength along the span or members for which the permanent load represents a large proportion of the total design load, concentrations of inelastic rotation may occur within the span. If such a condition is anticipated, transverse reinforcement is also required in regions where yielding is expected. Because spalling of the concrete shell might occur, especially at and near regions of flexural yielding, all web reinforcement is required to be provided in the form of closed hoops.

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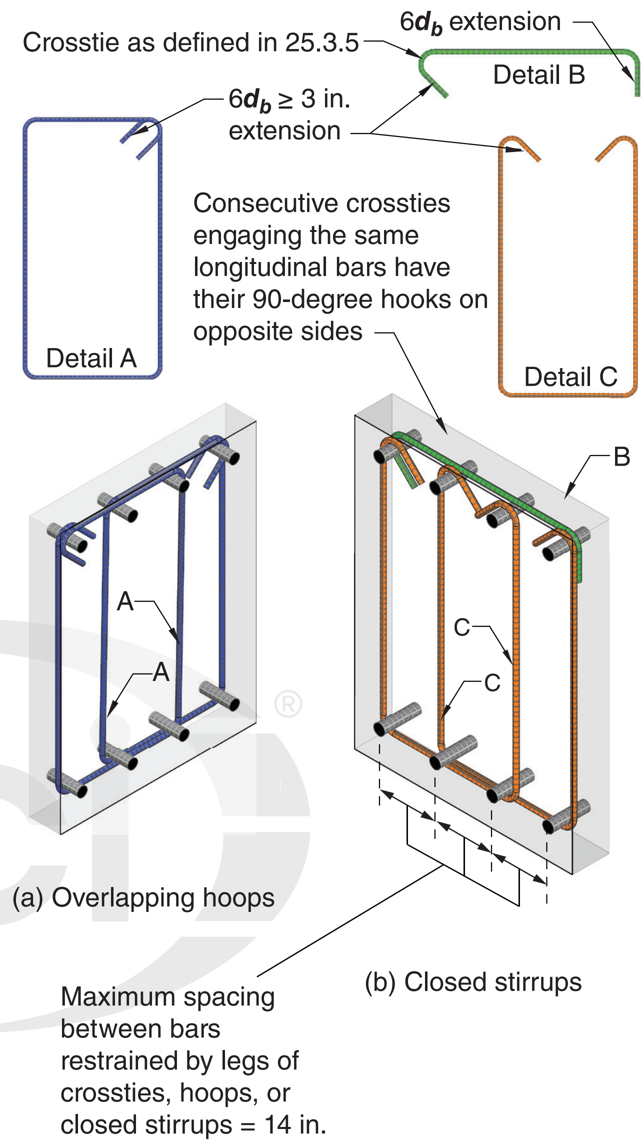

Fig. R18.6.4—Examples of beam transverse reinforcement and illustration of limit on maximum horizontal spacing of supported longitudinal bars.

18.6.4.1 Hoops or closed stirrups in accordance with 18.6.4.3 shall be provided in the following regions of a beam:

(a) Over a length equal to twice the beam depth measured from the face of the supporting column toward midspan, at both ends of the beam

(b) Over lengths equal to twice the beam depth on both sides of a section where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior.

18.6.4.2 In regions of the beam defined in 18.6.4.1, primary longitudinal reinforcing bars closest to the tension and compression faces shall have lateral support in accor-

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dance with 25.7.2.3 and 25.7.2.4. The transverse spacing of supported flexural reinforcing bars shall not exceed 14 in. Skin reinforcement required by 9.7.2.3 need not be laterally supported.

18.6.4.3 Closed stirrups in beams shall be permitted to be made up of one or more U-stirrups having seismic hooks at both ends, closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90-degree hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the crossties are confined by a slab on only one side of the beam, the 90-degree hooks of the crossties shall be placed on that side.

18.6.4.4 The first hoop or closed stirrup shall be located not more than 2 in. from the face of a supporting column. Spacing of the hoops or closed stirrups shall not exceed the least of (a) through (d):

(a) $d/4$

(b) 6 in.

(c) For Grade 60, $6d_b$ of the smallest primary flexural reinforcing bar excluding longitudinal skin reinforcement required by 9.7.2.3

(d) For Grade 80, $5d_b$ of the smallest primary flexural reinforcing bar excluding longitudinal skin reinforcement required by 9.7.2.3

18.6.4.5 Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than $d/2$ throughout the length of the beam.

18.6.4.6 In beams having factored axial compressive force exceeding $A_g f_c'/10$, hoops satisfying 18.7.5.2 through 18.7.5.4 shall be provided along lengths given in 18.6.4.1. Along the remaining length, hoops satisfying 18.7.5.2 shall have spacing s not exceeding the least of 6 in., $6d_b$ of the smallest Grade 60 enclosed longitudinal beam bar, and $5d_b$ of the smallest Grade 80 enclosed longitudinal beam bar. Where concrete cover over transverse reinforcement exceeds 4 in., additional transverse reinforcement having cover not exceeding 4 in. and spacing not exceeding 12 in. shall be provided.

18.6.5 *Shear strength*

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R18.6.5 *Shear strength*

Unless a beam possesses a moment strength that is on the order of 3 or 4 times the design moment, it should be assumed that it will yield in flexure in the event of a major earthquake. The design shear force should be selected so as to be a good approximation of the maximum shear that may develop in a member. Therefore, required shear strength for frame members is related to flexural strengths of the designed member rather than to factored shear forces indicated by lateral load analysis. The conditions described by 18.6.5.1 are illustrated in Fig. R18.6.5. The figure also shows that vertical earthquake effects are to be included, as is typi-

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cally required by the general building code. For example, ASCE/SEI 7 requires vertical earthquake effects, $0.2S_{DS}$, to be included.

Because the actual yield strength of the longitudinal reinforcement may exceed the specified yield strength and because strain hardening of the reinforcement is likely to take place at a joint subjected to large rotations, required shear strengths are determined using a stress of at least $1.25f_y$ in the longitudinal reinforcement.

Experimental studies (Popov et al. 1972) of reinforced concrete members subjected to cyclic loading have demonstrated that more shear reinforcement is required to ensure a flexural failure if the member is subjected to alternating nonlinear displacements than if the member is loaded in only one direction: the necessary increase of shear reinforcement being higher in the case of no axial load. This observation is reflected in the Code (refer to 18.6.5.2) by eliminating the term representing the contribution of concrete to shear strength. The added conservatism on shear is deemed necessary in locations where potential flexural hinging may occur. However, this stratagem, chosen for its relative simplicity, should not be interpreted to mean that no concrete is required to resist shear. On the contrary, it may be argued that the concrete core resists all the shear with the shear (transverse) reinforcement confining and strengthening the concrete. The confined concrete core plays an important role in the behavior of the beam and should not be reduced to a minimum just because the design expression does not explicitly recognize it.

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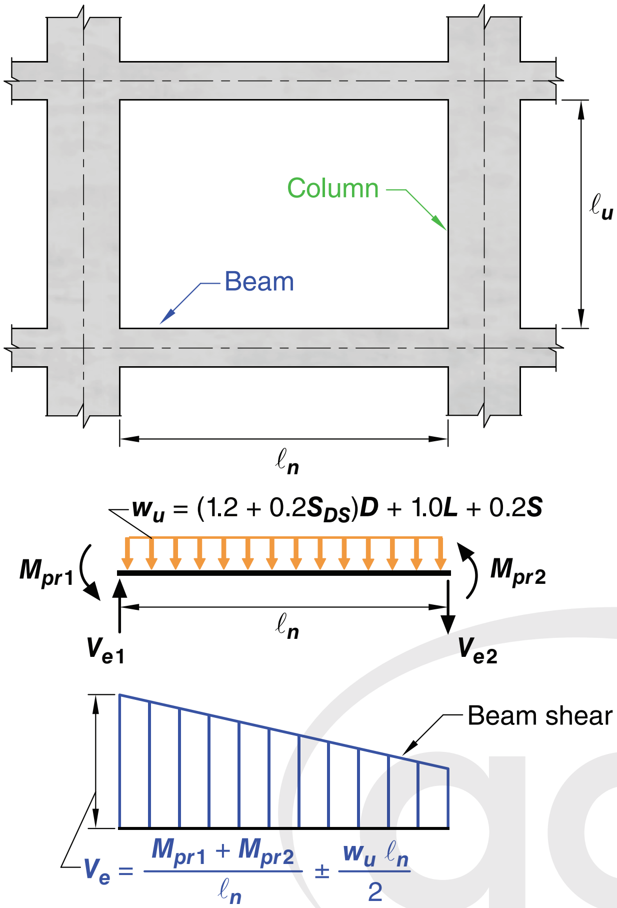

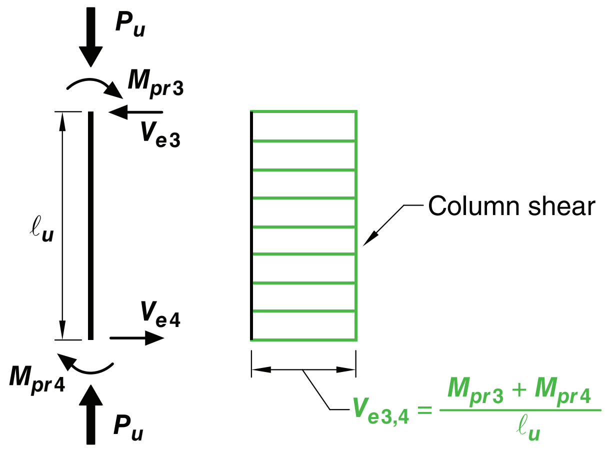

Notes on Fig. R18.6.5:

1. Direction of shear force V_e depends on relative magnitudes of gravity loads and shear generated by end moments.
2. End moments M_{pr} based on steel tensile stress of $1.25f_y$, where f_y is specified yield strength. (Both end moments should be considered in both directions, clockwise and counter-clockwise).
3. End moment M_{pr} for columns need not be greater than moments generated by the M_{pr} of the beams framing into the beam-column joints. V_e should not be less than that required by analysis of the structure.

Fig. R18.6.5—Design shears for beams and columns.

18.6.5.1 *Design forces*

The design shear force V_e shall be calculated from consideration of the forces on the portion of the beam between faces of the joints. It shall be assumed that moments of opposite sign corresponding to probable flexural strength, M_{pr}, act at the joint faces and that the beam is loaded with the factored gravity and vertical earthquake loads along its span.

18.6.5.2 *Transverse reinforcement*

Transverse reinforcement over the lengths identified in 18.6.4.1 shall be designed to resist shear assuming $V_c = 0$ when both (a) and (b) occur:

(a) The earthquake-induced shear force calculated in accordance with 18.6.5.1 represents at least one-half of the maximum required shear strength within those lengths.

(b) The factored axial compressive force P_u including earthquake effects is less than $A_g f_c'/20$.

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18.7—Columns of special moment frames

18.7.1 *Scope*

18.7.1.1 This section shall apply to columns of special moment frames that form part of the seismic-force-resisting system and are proportioned primarily to resist flexure, shear, and axial forces.

18.7.2 *Dimensional limits*

18.7.2.1 Columns shall satisfy (a) and (b):

(a) The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall be at least 12 in.

(b) The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall be at least 0.4.

18.7.3 *Minimum flexural strength of columns*

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R18.7—Columns of special moment frames

R18.7.1 *Scope*

This section applies to columns of special moment frames regardless of the magnitude of axial force. Before 2014, the Code permitted columns with low levels of axial stress to be detailed as beams.

R18.7.2 *Dimensional limits*

The geometric constraints in this provision follow from previous practice (Seismology Committee of SEAOC [1996]).

R18.7.3 *Minimum flexural strength of columns*

The intent of 18.7.3.2 is to reduce the likelihood of yielding in columns that are considered as part of the seismic-force-resisting system. If columns are not stronger than beams framing into a joint, there is increased likelihood of inelastic action. In the worst case of weak columns, flexural yielding can occur at both ends of all columns in a given story, resulting in a column failure mechanism that can lead to collapse. Connections with discontinuous columns above the connection, such as roof-level connections, are exempted if the column axial load is low, because special moment frame columns with low axial stress are inherently ductile and column yielding at such levels is unlikely to create a column failure mechanism that can lead to collapse.

In 18.7.3.2, the nominal strengths of the beams and columns are calculated at the joint faces, and those strengths are compared directly using Eq. (18.7.3.2). The 1995 and earlier Codes required design strengths to be compared at the center of the joint, which typically produced similar results but with added calculation effort.

In determining the nominal moment strength of a beam section in negative bending (top in tension), longitudinal reinforcement contained within an effective flange width of a top slab that acts monolithically with the beam increases the beam strength. French and Moehle (1991), on beam-column subassemblies under lateral loading, indicates that using the effective flange widths defined in 6.3.2 gives reasonable estimates of beam negative moment strengths of interior connections at story displacements approaching 2 percent of

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story height. This effective width is conservative where the slab terminates in a weak spandrel.

If 18.7.3.2 cannot be satisfied at a joint, 18.7.3.3 requires that any positive contribution of the column or columns involved to the lateral strength and stiffness of the structure is to be ignored. Negative contributions of the column or columns should not be ignored. For example, ignoring the stiffness of the columns ought not to be used as a justification for reducing the design base shear. If inclusion of those columns in the analytical model of the building results in an increase in torsional effects, the increase should be considered as required by the general building code. Furthermore, the column must be provided with transverse reinforcement to increase its resistance to shear and axial forces.

18.7.3.1 Columns shall satisfy 18.7.3.2 or 18.7.3.3, except at connections where the column is discontinuous above the connection and the column factored axial compressive force P_u under load combinations including earthquake effect, E, are less than $A_g f_c'/10$.

18.7.3.2 The flexural strengths of the columns shall satisfy

$$\sum M_{nc} \geq (6/5)\sum M_{nb} \quad (18.7.3.2)$$

where

$\sum M_{nc}$ is sum of nominal flexural strengths of columns framing into the joint, evaluated at the faces of the joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.

$\sum M_{nb}$ is sum of nominal flexural strengths of the beams framing into the joint, evaluated at the faces of the joint. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width defined in accordance with 6.3.2 shall be assumed to contribute to M_{nb} if the slab reinforcement is developed at the critical section for flexure.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Equation (18.7.3.2) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

18.7.3.3 If 18.7.3.2 is not satisfied at a joint, the lateral strength and stiffness of the columns framing into that joint shall be ignored when calculating strength and stiffness of the structure. These columns shall conform to 18.14.

18.7.4 *Longitudinal reinforcement*

R18.7.4 *Longitudinal reinforcement*

The lower limit of the area of longitudinal reinforcement is to control time-dependent deformations and to have the yield moment exceed the cracking moment. The upper limit of the area reflects concern for reinforcement congestion, load transfer from floor elements to column (especially in low-rise construction) and the development of high shear stresses.

Spalling of the shell concrete, which is likely to occur near the ends of the column in frames of typical configuration, makes lap splices in these locations vulnerable. If lap splices are to be used at all, they should be located near the midheight where stress reversal is likely to be limited to a smaller stress range than at locations near the joints. Transverse reinforcement is required along the lap-splice length because of the uncertainty in moment distributions along the height and the need for confinement of lap splices subjected to stress reversals (Sivakumar et al. 1983).

18.7.4.1 Area of longitudinal reinforcement, A_{st}, shall be at least $0.01A_g$ and shall not exceed $0.06A_g$.

18.7.4.2 In columns with circular hoops, there shall be at least six longitudinal bars.

18.7.4.3 Over column clear height, either (a) or (b) shall be satisfied:

(a) Longitudinal reinforcement shall be selected such that $1.25\ell_d \leq \ell_u/2$.

(b) Transverse reinforcement shall be selected such that $K_{tr} \geq 1.2d_b$.

R18.7.4.3 Bond splitting failure along longitudinal bars within the clear column height may occur under earthquake demands (Ichinose 1995; Sokoli and Ghannoum 2016).

Splitting can be controlled by restricting longitudinal bar size, increasing the amount of transverse reinforcement, or increasing concrete strength, all of which reduce the development length of longitudinal bars (ℓ_d) over column clear height (ℓ_u).

18.7.4.4 Mechanical splices shall conform to 18.2.7 and welded splices shall conform to 18.2.8. Lap splices shall be permitted only within the center half of the member length, shall be designed as tension lap splices, and shall be enclosed within transverse reinforcement in accordance with 18.7.5.2 and 18.7.5.3.

18.7.5 *Transverse reinforcement*

R18.7.5 *Transverse reinforcement*

This section is concerned with confining the concrete and providing lateral support to the longitudinal reinforcement.

18.7.5.1 Transverse reinforcement required in 18.7.5.2 through 18.7.5.4 shall be provided over a length ℓ_o from each joint face and on both sides of any section where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior. Length ℓ_o shall be at least the greatest of (a) through (c):

(a) The depth of the column at the joint face or at the section where flexural yielding is likely to occur

(b) One-sixth of the clear span of the column

(c) 18 in.

R18.7.5.1 This section stipulates a minimum length over which to provide closely-spaced transverse reinforcement at the column ends, where flexural yielding normally occurs. Research results indicate that the length should be increased by 50% or more in locations, such as the base of a building, where axial loads and flexural demands may be especially high (Watson et al. 1994)

18.7.5.2 Transverse reinforcement shall be in accordance with (a) through (f):

R18.7.5.2 Sections 18.7.5.2 and 18.7.5.3 provide requirements for configuration of transverse reinforcement for columns and joints of special moment frames. Figure R18.7.5.2 shows an example of transverse reinforcement

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(a) Transverse reinforcement shall comprise either single or overlapping spirals, circular hoops, or single or overlapping rectilinear hoops with or without crossties.

(b) Bends of rectilinear hoops and crossties shall engage peripheral longitudinal reinforcing bars.

(c) Crossties of the same or smaller bar size as the hoops shall be permitted, subject to the limitation of 25.7.2.2. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement and around the perimeter of the cross section.

(d) Where rectilinear hoops or crossties are used, they shall provide lateral support to longitudinal reinforcement in accordance with 25.7.2.2 and 25.7.2.3.

(e) Reinforcement shall be arranged such that the spacing h_x of longitudinal bars laterally supported by the corner of a crosstie or hoop leg shall not exceed 14 in. around the perimeter of the column.

(f) Where $P_u > 0.3A_g f_c'$ or $f_c' > 10{,}000$ psi in columns with rectilinear hoops, every longitudinal bar or bundle of bars around the perimeter of the column core shall have lateral support provided by the corner of a hoop or by a seismic hook, and the value of h_x shall not exceed 8 in. P_u shall be the largest value in compression consistent with factored load combinations including E.

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provided by one hoop and three crossties. Crossties with a 90-degree hook are not as effective as either crossties with 135-degree hooks or hoops in providing confinement. For lower values of $P_u/A_g f_c'$ and lower concrete compressive strengths, crossties with 90-degree hooks are adequate if the ends are alternated along the length and around the perimeter of the column. For higher values of $P_u/A_g f_c'$, for which compression-controlled behavior is expected, and for higher compressive strengths, for which behavior tends to be more brittle, the improved confinement provided by having corners of hoops or seismic hooks supporting all longitudinal bars is important to achieving intended performance. Where these conditions apply, crossties with seismic hooks at both ends are required. The 8 in. limit on h_x is also intended to improve performance under these critical conditions. For bundled bars, bends or hooks of hoops and crossties need to enclose the bundle, and longer extensions on hooks should be considered. Column axial load P_u should reflect factored compressive demands from both earthquake and gravity loads.

In past editions of the Code, the requirements for transverse reinforcement in columns, walls, beam-column joints, and diagonally reinforced coupling beams referred to the same equations. In the 2014 edition of the Code, the equations and detailing requirements differ among the member types based on consideration of their loadings, deformations, and performance requirements. Additionally, h_x previously referred to the distance between legs of hoops or crossties. In the 2014 edition of the Code, h_x refers to the distance between longitudinal bars supported by those hoops or crossties.

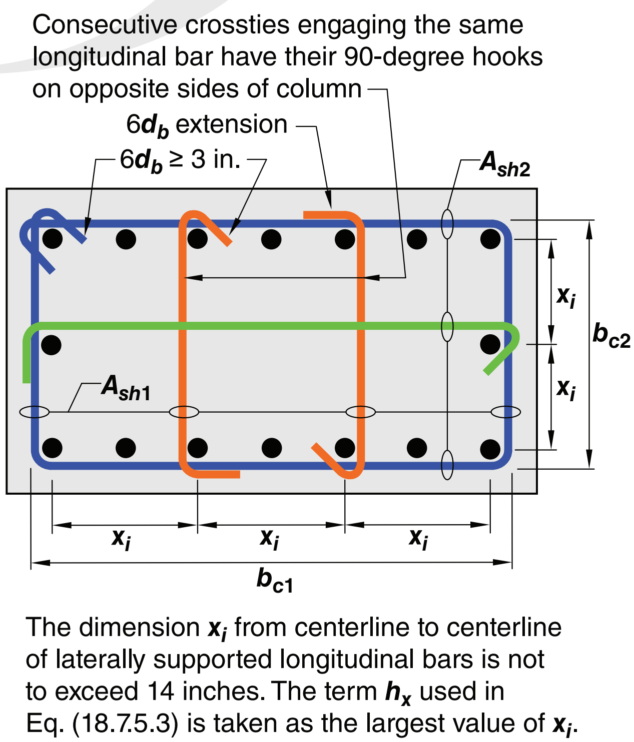

Fig. R18.7.5.2—Example of transverse reinforcement in columns.

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18.7.5.3 Spacing of transverse reinforcement shall not exceed the least of (a) through (d):

(a) One-fourth of the minimum column dimension

(b) For Grade 60, $6d_b$ of the smallest longitudinal bar

(c) For Grade 80, $5d_b$ of the smallest longitudinal bar

(d) s_o, as calculated by:

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right) \quad (18.7.5.3)$$

The value of s_o from Eq. (18.7.5.3) shall not exceed 6 in. and need not be taken less than 4 in.

18.7.5.4 Amount of transverse reinforcement shall be in accordance with Table 18.7.5.4.

The concrete strength factor k_f and confinement effectiveness factor k_n are calculated according to Eq. (18.7.5.4a) and (18.7.5.4b).

$$\text{(a) } k_f = \frac{f_c'}{25{,}000} + 0.6 \geq 1.0 \quad (18.7.5.4a)$$

$$\text{(b) } k_n = \frac{n_l}{n_l - 2} \quad (18.7.5.4b)$$

where n_l is the number of longitudinal bars or bar bundles around the perimeter of a column core with rectilinear hoops that are laterally supported by the corner of hoops or by seismic hooks.

Table 18.7.5.4—Transverse reinforcement for columns of special moment frames

Transverse reinforcement	Conditions	Applicable expressions	
A_{sh}/sb_c for rectilinear hoop	$P_u \leq 0.3A_gf_c'$ and $f_c' \leq 10{,}000$ psi	Greater of (a) and (b)	$0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$ (a)
			$0.09\frac{f_c'}{f_{yt}}$ (b)
	$P_u > 0.3A_gf_c'$ or $f_c' > 10{,}000$ psi	Greatest of (a), (b), and (c)	$0.2k_fk_n\frac{P_u}{f_{yt}A_{ch}}$ (c)
ρ_s for spiral or circular hoop	$P_u \leq 0.3A_gf_c'$ and $f_c' \leq 10{,}000$ psi	Greater of (d) and (e)	$0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$ (d)
			$0.12\frac{f_c'}{f_{yt}}$ (e)
	$P_u > 0.3A_gf_c'$ or $f_c' > 10{,}000$ psi	Greatest of (d), (e), and (f)	$0.35k_f\frac{P_u}{f_{yt}A_{ch}}$ (f)

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R18.7.5.3 The requirement that spacing not exceed one-fourth of the minimum member dimension or 6 in. is for concrete confinement. If the maximum spacing of crossties or legs of overlapping hoops within the section is less than 14 in., then the 4 in. limit can be increased as permitted by Eq. (18.7.5.3). The spacing limit as a function of the longitudinal bar diameter is intended to provide adequate longitudinal bar restraint to control buckling after spalling.

R18.7.5.4 The effect of helical (spiral) reinforcement and adequately configured rectilinear hoop reinforcement on deformation capacity of columns is well established (Sakai and Sheikh 1989). Expressions (a), (b), (d), and (e) in Table 18.7.5.4 have historically been used in ACI 318 to calculate the required confinement reinforcement to ensure that spalling of shell concrete does not result in a loss of column axial load strength. Expressions (c) and (f) were developed from a review of column test data (Elwood et al. 2009) and are intended to result in columns capable of sustaining a drift ratio of 0.03 with limited strength degradation. Expressions (c) and (f) are triggered for axial load greater than $0.3A_gf_c'$, which corresponds approximately to the onset of compression-controlled behavior for symmetrically reinforced columns. The k_n term (Paultre and Légeron 2008) decreases the required confinement for columns with closely spaced, laterally supported longitudinal reinforcement because such columns are more effectively confined than columns with more widely spaced longitudinal reinforcement. The k_f term increases the required confinement for columns with $f_c' > 10{,}000$ psi because such columns can experience brittle failure if not well confined. Concrete strengths greater than 15,000 psi should be used with caution given the limited test data for such columns. The concrete strength used to determine the confinement reinforcement is required to be the same as that specified in the construction documents.

Expressions (a), (b), and (c) in Table 18.7.5.4 are to be satisfied in both cross-sectional directions of the rectangular core. For each direction, b_c is the core dimension perpendicular to the tie legs that constitute A_{sh}, as shown in Fig. R18.7.5.2.

Research results indicate that high strength reinforcement can be used effectively as confinement reinforcement. Section 20.2.2.4 permits a value of f_{yt} as high as 100,000 psi to be used in Table 18.7.5.4.

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18.7.5.5 Beyond the length ℓ_o given in 18.7.5.1, the column shall contain spiral reinforcement satisfying 25.7.3 or hoop and crosstie reinforcement satisfying 25.7.2 and 25.7.4 with spacing s not exceeding the least of 6 in., $6d_b$ of the smallest Grade 60 longitudinal column bar, and $5d_b$ of the smallest Grade 80 longitudinal column bar, unless a greater amount of transverse reinforcement is required by 18.7.4.4 or 18.7.6.

18.7.5.6 Columns supporting reactions from discontinued stiff members, such as walls, shall satisfy (a) and (b):

(a) Transverse reinforcement required by 18.7.5.2 through 18.7.5.4 shall be provided over the full height at all levels beneath the discontinuity if the factored axial compressive force in these columns, related to earthquake effect, exceeds $A_g f_c'/10$. Where design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $A_g f_c'/10$ shall be increased to $A_g f_c'/4$.

(b) Transverse reinforcement shall extend into the discontinued member at least ℓ_d of the largest longitudinal column bar, where ℓ_d is in accordance with 18.8.5. Where the lower end of the column terminates on a wall, the required transverse reinforcement shall extend into the wall at least ℓ_d of the largest longitudinal column bar at the point of termination. Where the column terminates on a footing or mat, the required transverse reinforcement shall extend at least 12 in. into the footing or mat.

18.7.5.7 If the concrete cover outside the confining transverse reinforcement required by 18.7.5.1, 18.7.5.5, and 18.7.5.6 exceeds 4 in., additional transverse reinforcement having cover not exceeding 4 in. and spacing not exceeding 12 in. shall be provided.

18.7.6 *Shear strength*
18.7.6.1 *Design forces*

18.7.6.1.1 The design shear force V_e shall be calculated from considering the maximum forces that can be generated at the faces of the joints at each end of the column. These joint forces shall be calculated using the maximum probable flexural strengths, M_{pr}, at each end of the column associated with the range of factored axial forces, P_u, acting on the column. The column shears need not exceed those calculated from joint strengths based on M_{pr} of the beams framing into the joint. In no case shall V_e be less than the factored shear calculated by analysis of the structure.

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R18.7.5.5 This provision is intended to provide reasonable protection to the midheight of columns outside the length ℓ_o. Observations after earthquakes have shown significant damage to columns in this region, and the minimum hoops or spirals required should provide more uniform strength of the column along its length.

R18.7.5.6 Columns supporting discontinued stiff members, such as walls or trusses, may develop considerable inelastic response. Therefore, it is required that these columns have the specified reinforcement throughout their length. This covers all columns beneath the level at which the stiff member has been discontinued, unless the factored forces corresponding to earthquake effect are low. Refer to R18.12.7.6 for discussion of the overstrength factor Ω_o.

R18.7.5.7 The unreinforced shell may spall as the column deforms to resist earthquake effects. Separation of portions of the shell from the core caused by local spalling creates a falling hazard. The additional reinforcement is required to reduce the risk of portions of the shell falling away from the column.

R18.7.6 *Shear strength*
R18.7.6.1 *Design forces*

R18.7.6.1.1 The procedures of 18.6.5.1 also apply to columns. Above the ground floor, the moment at a joint may be limited by the flexural strength of the beams framing into the joint. Where beams frame into opposite sides of a joint, the combined strength is the sum of the negative moment strength of the beam on one side of the joint and the positive moment strength of the beam on the other side of the joint. Moment strengths are to be determined using a strength reduction factor of 1.0 and reinforcement with an effective yield stress equal to at least $1.25f_y$. Distribution of the combined moment strength of the beams to the columns above and below the joint should be based on analysis.

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18.7.6.2 *Transverse reinforcement*

18.7.6.2.1 Transverse reinforcement over the lengths ℓ_o, given in 18.7.5.1, shall be designed to resist shear assuming $V_c = 0$ when both (a) and (b) occur:

(a) The earthquake-induced shear force, calculated in accordance with 18.7.6.1, is at least one-half of the maximum required shear strength within ℓ_o.

(b) The factored axial compressive force P_u including earthquake effects is less than $A_g f_c'/20$.

18.8—Joints of special moment frames

18.8.1 *Scope*

18.8.1.1 This section shall apply to beam-column joints of special moment frames forming part of the seismic-force-resisting system.

18.8.2 *General*

18.8.2.1 Forces in longitudinal beam reinforcement at the joint face shall be calculated assuming that the stress in the flexural tensile reinforcement is $1.25f_y$.

18.8.2.2 Longitudinal reinforcement terminated in a joint shall extend to the far face of the joint core and shall be developed in tension in accordance with 18.8.5.

18.8.2.3 Where longitudinal beam reinforcement extends through a beam-column joint, the depth h of the joint parallel to the beam longitudinal reinforcement shall be at least the greatest of (a) through (c):

(a) $(20/\lambda)d_b$ of the largest Grade 60 longitudinal bar, where $\lambda = 0.75$ for lightweight concrete and 1.0 for all other cases

(b) $26d_b$ of the largest Grade 80 longitudinal bar

(c) $h/2$ of any beam framing into the joint and generating joint shear as part of the seismic-force-resisting system in the direction under consideration

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R18.8—Joints of special moment frames

R18.8.2 *General*

Development of inelastic rotations at the faces of joints of reinforced concrete frames is associated with strains in the flexural reinforcement well in excess of the yield strain. Consequently, joint shear force generated by the flexural reinforcement is calculated for a stress of $1.2f_y$ in the reinforcement (refer to 18.8.2.1). A detailed explanation of the reasons for the possible development of stresses in excess of the yield strength in beam tensile reinforcement is provided in ACI PRC-352.

R18.8.2.2 The design provisions for hooked bars in special moment frames are based mainly on research and experience for joints with standard 90-degree hooks. Therefore, standard 90-degree hooks generally are preferred to standard 180-degree hooks unless unusual considerations dictate use of 180-degree hooks. Prior to the 2025 edition of the Code, it was required to check compression development length of longitudinal reinforcement. Assessment of experimental data indicates this check is unnecessary to achieve satisfactory joint behavior (Uzumeri and Seckin 1974; Kang et al. 2009).

R18.8.2.3 Depth h of the joint is defined in Fig. R15.5.2.2. The column dimension parallel to the beam reinforcement in joints with circular columns may be taken as that of a square section of equivalent area. Research (Meinheit and Jirsa 1977; Briss et al. 1978; Ehsani 1982; Durrani and Wight 1982; Leon 1989; Aoyama 2001; Lin et al. 2000) has shown that straight longitudinal beam bars may slip within the beam-column joint during a series of large moment reversals. The bond stresses on these straight bars may be very large. To reduce slip substantially during the formation of

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adjacent beam hinging, it would be necessary to have a ratio of column dimension to bar diameter of approximately 32 for Grade 60 bars, which would result in very large joints. Tests demonstrate adequate behavior if the ratio of joint depth to maximum beam longitudinal bar diameter for Grade 60 reinforcement is at least 20 for normalweight concrete and 26 for lightweight concrete. A joint depth of $26d_b$ for Grade 80 reinforcement is intended to achieve similar performance to that of a joint depth of $20d_b$ for Grade 60 reinforcement and normalweight concrete. The limits on joint depth provide reasonable control on the amount of slip of the beam bars in a beam-column joint, considering the number of anticipated inelastic excursions of the building frame during a major earthquake. A thorough treatment of this topic is given in Zhu and Jirsa (1983).

Requirement (c) on joint aspect ratio applies only to beams that are designated as part of the seismic-force-resisting system. Joints having depth less than half the beam depth require a steep diagonal compression strut across the joint, which may be less effective in resisting joint shear. Tests to demonstrate performance of such joints have not been reported in the literature.

18.8.2.3.1 Concrete used in joints with Grade 80 longitudinal reinforcement shall be normalweight concrete.

R18.8.2.3.1 Test data justifying the combination of lightweight concrete and Grade 80 longitudinal reinforcement in joints are not available.

18.8.3 *Transverse reinforcement*

R18.8.3 *Transverse reinforcement*

The Code requires transverse reinforcement in a joint regardless of the magnitude of the calculated shear force.

18.8.3.1 Joint transverse reinforcement shall satisfy 18.7.5.2, 18.7.5.3, 18.7.5.4, and 18.7.5.7, except as permitted in 18.8.3.2.

18.8.3.2 Where beams frame into all four sides of the joint and where each beam width is at least three-fourths the column width, the amount of reinforcement required by 18.7.5.4 shall be permitted to be reduced by one-half, and the spacing required by 18.7.5.3 shall be permitted to be increased to 6 in. within the overall depth h of the shallowest framing beam.

R18.8.3.2 The amount of confining reinforcement may be reduced and the spacing may be increased if beams of adequate dimensions frame into all four sides of the joint.

18.8.3.3 Longitudinal beam reinforcement outside the column core shall be confined by transverse reinforcement passing through the column that satisfies spacing requirements of 18.6.4.4, and requirements of 18.6.4.2, and 18.6.4.3, if such confinement is not provided by a beam framing into the joint.

R18.8.3.3 The required transverse reinforcement, or transverse beam if present, is intended to confine the beam longitudinal reinforcement and improve force transfer to the beam-column joint.

An example of transverse reinforcement through the column provided to confine the beam reinforcement passing outside the column core is shown in Fig. R18.6.2. Additional detailing guidance and design recommendations for both interior and exterior wide-beam connections with beam reinforcement passing outside the column core may be found in ACI PRC-352.

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18.8.4 *Shear strength*

18.8.4.1 Joint shear force V_u shall be calculated on a plane at mid-height of the joint from calculated forces at the joint faces using tensile and compressive beam forces determined in accordance with 18.8.2.1 and column shear consistent with beam probable flexural strengths M_{pr}.

18.8.4.2 ϕ shall be in accordance with 21.2.4.4.

18.8.4.3 V_n of the joint shall be in accordance with Table 18.8.4.3.

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R18.8.4 *Shear strength*

The shear strength values given in 18.8.4.3 are based on the recommendation in ACI PRC-352 for joints with members that are expected to undergo reversals of deformation into the inelastic range, although the ACI PRC-352 definition of effective cross-sectional joint area is sometimes different. The given nominal joint shear strengths do not explicitly consider transverse reinforcement in the joint because tests of joints (Meinheit and Jirsa 1977) and deep beams (Hirosawa 1977) have indicated that joint shear strength is not sensitive to transverse reinforcement if at least the required minimum amount is provided in the joint.

Cyclic loading tests of joints with extensions of beams with lengths at least equal to their depths have indicated similar joint shear strengths to those of joints with continuous beams. These findings suggest that extensions of beams and columns, when properly dimensioned and reinforced with longitudinal and transverse bars, provide effective confinement to the joint faces, thus delaying joint strength deterioration at large deformations (Meinheit and Jirsa 1981).

Table 18.8.4.3—Nominal joint shear strength V_n

Column	Beam in direction of V_u	Confinement by transverse beams according to 15.5.2.5	V_n, lb[1]
Continuous or meets 15.5.2.3	Continuous or meets 15.5.2.4	Confined	$20\lambda\sqrt{f_c'}A_j$
		Not confined	$15\lambda\sqrt{f_c'}A_j$
	Other	Confined	$15\lambda\sqrt{f_c'}A_j$
		Not confined	$12\lambda\sqrt{f_c'}A_j$
Other	Continuous or meets 15.5.2.4	Confined	$15\lambda\sqrt{f_c'}A_j$
		Not confined	$12\lambda\sqrt{f_c'}A_j$
	Other	Confined	$12\lambda\sqrt{f_c'}A_j$
		Not confined	$8\lambda\sqrt{f_c'}A_j$

[1] λ shall be 0.75 for lightweight concrete and 1.0 for normalweight concrete. A_j shall be calculated in accordance with 15.5.2.2 .

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18.8.5 *Development length of bars in tension*

18.8.5.1 For bar sizes No. 3 through No. 11 terminating in a standard hook, ℓ_{dh} shall be calculated by Eq. (18.8.5.1), but ℓ_{dh} shall be at least the greater of $8d_b$ and 6 in. for normal-

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R18.8.5 *Development length of bars in tension*

R18.8.5.1 Minimum embedment length in tension for deformed bars with standard hooks is determined using Eq. (18.8.5.1), which is based on the requirements of 25.4.3 of ACI 318-14. The embedment length of a bar with a stan-

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weight concrete and at least the greater of $10d_b$ and 7-1/2 in. for lightweight concrete.

$$\ell_{dh} = f_y d_b/(65\lambda\sqrt{f_c'}) \quad (18.8.5.1)$$

The value of λ shall be 0.75 for concrete containing lightweight aggregate and 1.0 otherwise.

The hook shall be located within the confined core of a column or of a boundary element, with the hook bent into the joint.

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dard hook is the distance, parallel to the bar, from the critical section (where the bar is to be developed) to a tangent drawn to the outside edge of the hook. The tangent is to be drawn perpendicular to the axis of the bar (refer to Table 25.3.1).

Equation (18.8.5.1) was derived from the equation for development length ℓ_{dh} in 25.4.3 of ACI 318-14 using coefficients of 1.0 (no epoxy coating), 0.7 (cover) and 0.8 (confinement reinforcement) because Chapter 18 stipulates that the hook is to be embedded in confined concrete. The development length that would be derived directly from 25.4.3 in ACI 318-14 was increased to reflect the effect of load reversals. Factors such as the actual stress in the reinforcement being more than the yield strength and the effective development length not necessarily starting at the face of the joint were implicitly considered in the formulation of the expression for basic development length that has been used as the basis for Eq. (18.8.5.1).

The requirement for the hook to project into the joint is to improve development of a diagonal compression strut across the joint. The requirement applies to beam and column bars terminated at a joint with a standard hook, preferably a standard 90-degree hook.

18.8.5.2 Headed deformed bars satisfying 20.2.1.6 shall develop $1.25f_y$ in tension in accordance with 25.4.4 by substituting a bar stress of $1.25f_y$ for f_y.

R18.8.5.2 The factor 1.25 is intended to represent the potential increase in stresses due to inelastic response, including strain hardening that may occur in beams of special moment frames.

18.8.5.3 For bar sizes No. 3 through No. 11, ℓ_d, the development length in tension for a straight bar, shall be at least the greater of (a) and (b):

(a) 2.5 times the length in accordance with 18.8.5.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 12 in.

(b) 3.25 times the length in accordance with 18.8.5.1 if the depth of the concrete cast in one lift beneath the bar exceeds 12 in.

R18.8.5.3 Minimum development length in tension for straight bars is a multiple of the length indicated by 18.8.5.1. Section 18.8.5.3(b) refers to top bars. Lack of reference to No. 14 and No. 18 bars in 18.8.5 is due to the paucity of information on anchorage of such bars subjected to load reversals simulating earthquake effects.

18.8.5.4 Straight bars terminated at a joint shall pass through the confined core of a column or a boundary element. Any portion of ℓ_d not within the confined core shall be increased by a factor of 1.6.

R18.8.5.4 If the required straight embedment length of a reinforcing bar extends beyond the confined volume of concrete (as defined in 18.6.4, 18.7.5, or 18.8.3), the required development length is increased on the premise that the limiting bond stress outside the confined region is less than that inside.

$$\ell = 1.6(\ell_d - \ell_{dc}) + \ell_{dc}$$

or

$$\ell_{dm} = 1.6\ell_d - 0.6\ell_{dc}$$

where ℓ_{dm} is the required development length if bar is not entirely embedded in confined concrete; ℓ_d is the required development length in tension for straight bar as defined in 18.8.5.3; and ℓ_{dc} is the length of bar embedded in confined concrete.

18.8.5.5 If epoxy-coated reinforcement is used, the development lengths in 18.8.5.1, 18.8.5.3, and 18.8.5.4 shall be multiplied by applicable factors in 25.4.2.5 or 25.4.3.2.

18.9—Special moment frames constructed using precast concrete

R18.9—Special moment frames constructed using precast concrete

The detailing provisions in 18.9.2.1 and 18.9.2.2 are intended to produce frames that respond to design displacements essentially like monolithic special moment frames.

Precast frame systems composed of concrete elements with ductile connections are expected to experience flexural yielding in connection regions (Yoshioka and Sekine 1991; Kurose et al. 1991; Restrepo et al. 1995a,b). The restriction on location of mechanical splices is intended to avoid strain concentrations over a short length of reinforcement adjacent to a splice device. Additional requirements for shear strength are provided in 18.9.2.1 to prevent sliding on connection faces. Precast frames composed of elements with ductile connections may be designed to promote yielding at locations not adjacent to the joints. Therefore, design shear V_e, as calculated according to 18.6.5.1 or 18.7.6.1, may not be conservative.

Precast concrete frame systems composed of elements joined using strong connections are intended to experience flexural yielding outside the connections. Strong connections include the length of the mechanical splice hardware as shown in Fig. R18.9.2.2. Capacity-design techniques are used in 18.9.2.2(c) to ensure the strong connection remains elastic following formation of plastic hinges. Additional column requirements are provided to avoid hinging and strength deterioration of column-to-column connections.

Strain concentrations have been observed to cause brittle fracture of reinforcing bars at the face of mechanical splices in laboratory tests of precast beam-column connections (Palmieri et al. 1996). Locations of strong connections should be selected carefully or other measures should be taken, such as debonding of reinforcing bars in highly stressed regions, to avoid strain concentrations that can result in premature fracture of reinforcement.

18.9.1 *Scope*

18.9.1.1 This section shall apply to special moment frames constructed using precast concrete forming part of the seismic-force-resisting system.

18.9.2 *General*

R18.9.2 *General*

18.9.2.1 Special moment frames with ductile connections constructed using precast concrete shall satisfy (a) through (c):

(a) Requirements of 18.6 through 18.8 for special moment frames constructed with cast-in-place concrete

(b) V_n for connections calculated according to 22.9 shall be at least $2V_e$, where V_e is in accordance with 18.6.5.1 or 18.7.6.1

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(c) Mechanical splices of beam reinforcement shall be located not closer than $h/2$ from the joint face and shall be Class S.

18.9.2.2 Special moment frames with strong connections constructed using precast concrete shall satisfy (a) through (e):

(a) Requirements of 18.6 through 18.8 for special moment frames constructed with cast-in-place concrete

(b) Provision 18.6.2.1(a) shall apply to segments between locations where flexural yielding is intended to occur due to design displacements

(c) Design strength of the strong connection, ϕS_n, shall be at least S_e

(d) Primary longitudinal reinforcement shall be made continuous across connections and shall be developed outside both the strong connection and the plastic hinge region

(e) For column-to-column connections, ϕS_n shall be at least $1.4S_e$, ϕM_n shall be at least $0.4M_{pr}$ for the column within the story height, and ϕV_n shall be at least V_e in accordance with 18.7.6.1

18.9.2.3 Special moment frames constructed using precast concrete and not satisfying 18.9.2.1 or 18.9.2.2 shall satisfy (a) through (c):

(a) ACI CODE-374.1

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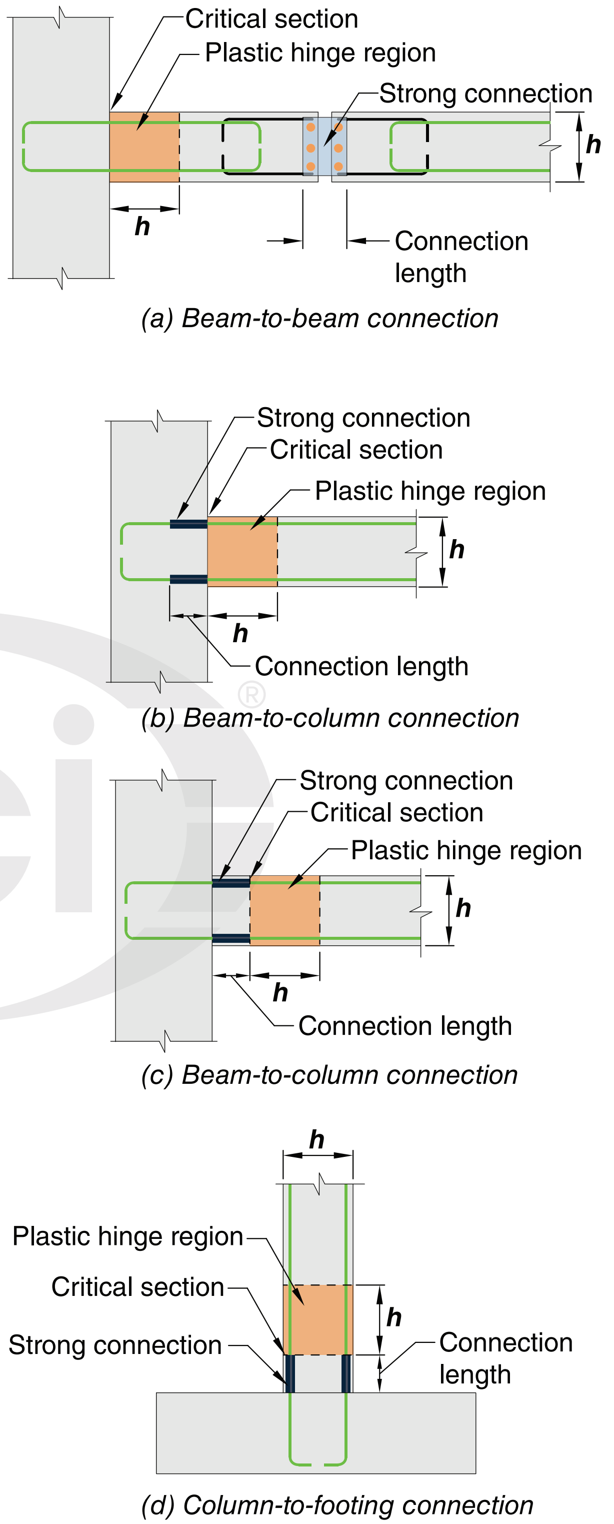

Fig. R18.9.2.2—Strong connection examples.

R18.9.2.3 Precast frame systems not satisfying the prescriptive requirements of Chapter 18 have been demonstrated in experimental studies to provide satisfactory seismic performance characteristics (Stone et al. 1995; Nakaki et al. 1999). ACI CODE-374.1 defines a protocol for establishing

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(b) Details and materials used in the test specimens shall be representative of those used in the structure

(c) The design procedure used to proportion the test specimens shall define the mechanism by which the frame resists gravity and earthquake effects, and shall establish acceptance values for sustaining that mechanism. Portions of the mechanism that deviate from Code requirements shall be contained in the test specimens and shall be tested to determine upper bounds for acceptance values.

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a design procedure, validated by analysis and laboratory tests, for such frames. The design procedure should identify the load path or mechanism by which the frame resists gravity and earthquake effects. The tests should be configured to investigate critical behaviors, and the measured quantities should establish upper-bound acceptance values for components of the load path, which may be in terms of limiting stresses, forces, strains, or other quantities. The design procedure used for the structure should not deviate from that used to design the test specimens, and acceptance values should not exceed values that were demonstrated by the tests to be acceptable. Materials and components used in the structure should be similar to those used in the tests. Deviations may be acceptable if the licensed design professional can demonstrate that those deviations do not adversely affect the behavior of the framing system.

ACI CODE-550.3 defines design requirements for one type of special precast concrete moment frame for use in accordance with 18.9.2.3.

18.10—Special structural walls

18.10.1 *Scope*

R18.10—Special structural walls

R18.10.1 *Scope*

This section contains requirements for the dimensions and details of special structural walls and all components including coupling beams and wall piers. Wall piers are defined in Chapter 2. Design provisions for vertical wall segments depend on the aspect ratio of the wall segment in the plane of the wall (h_w/ℓ_w), and the aspect ratio of the horizontal cross section (ℓ_w/b_w), and generally follow the descriptions in Table R18.10.1. The limiting aspect ratios for wall piers are based on engineering judgment. It is intended that flexural yielding of the vertical reinforcement in the pier should limit shear demand on the pier.

Bearing wall systems utilizing special structural walls with $h_{wcs}/\ell_w \geq 2.0$ that are designed and detailed according to these provisions are expected to meet the seismic performance objectives of Building Frame Systems: Special reinforced concrete shear walls as defined by ASCE/SEI 7.

Table R18.10.1—Governing design provisions for vertical wall segments[1]

Clear height of vertical wall segment/length of vertical wall segment (h_w/ℓ_w)	Length of vertical wall segment/wall thickness (ℓ_w/b_w)		
	$(\ell_w/b_w) \leq 2.5$	$2.5 < (\ell_w/b_w) \leq 6.0$	$(\ell_w/b_w) > 6.0$
$h_w/\ell_w < 2.0$	Wall	Wall	Wall
$h_w/\ell_w \geq 2.0$	Wall pier required to satisfy specified column design requirements; refer to 18.10.8.1	Wall pier required to satisfy specified column design requirements or alternative requirements; refer to 18.10.8.1	Wall

[1]h_w is the clear height, ℓ_w is the horizontal length, and b_w is the width of the web of the wall segment.

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18.10.1.1 This section shall apply to special structural walls, including ductile coupled walls, and all components of special structural walls including coupling beams and wall piers forming part of the seismic-force-resisting system.

18.10.1.2 Special structural walls constructed using precast concrete shall be in accordance with 18.11 in addition to 18.10.

18.10.2 *Reinforcement*

18.10.2.1 The distributed web reinforcement ratios, ρ_ℓ and ρ_t, for structural walls shall be at least 0.0025, except that if V_u does not exceed $\lambda\sqrt{f_c'}A_{cv}$, ρ_t shall be permitted to be reduced to the values in 11.6. Reinforcement spacing each way in structural walls shall not exceed 18 in. Reinforcement contributing to V_n shall be continuous and shall be distributed across the shear plane.

18.10.2.2 At least two curtains of reinforcement shall be used in a wall if $V_u > 2\lambda\sqrt{f_c'}A_{cv}$ or $h_w/\ell_w \geq 2.0$, in which h_w and ℓ_w refer to height and length of entire wall, respectively.

18.10.2.3 Reinforcement in structural walls shall be developed or spliced for f_y in tension in accordance with 25.4, 25.5, and (a) through (d):

(a) Except at the top of a wall, longitudinal reinforcement shall extend at least 12 ft above the point at which it is no longer required to resist flexure but need not extend more than ℓ_d above the next floor level.

(b) At locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements, longitudinal reinforcement shall develop 1.25 f_y in tension in accordance with 25.4 by substituting a bar stress of 1.25f_y for f_y.

(c) Lap splices of longitudinal reinforcement within boundary regions shall not be permitted over a height equal to h_{sx} above, and ℓ_d below, critical sections where yielding of longitudinal reinforcement is likely to occur

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R18.10.2 *Reinforcement*

Minimum reinforcement requirements in 18.10.2.1 follow from preceding Codes. The requirement for distributed shear reinforcement is related to the intent to control the width of inclined cracks. The requirement for two layers of reinforcement in walls resisting substantial design shears in 18.10.2.2 is based on the observation that, under ordinary construction conditions, the probability of maintaining a single layer of reinforcement near the middle of the wall section is quite low. Furthermore, presence of reinforcement close to the surface tends to inhibit fragmentation of the concrete in the event of severe cracking during an earthquake. The requirement for two layers of vertical reinforcement in more slender walls is to improve lateral stability of the compression zone under cyclic loads following yielding of vertical reinforcement in tension.

R18.10.2.3 Requirements are based on provisions in Chapter 25, with modifications to address issues specific to structural walls, as well as to the use of high-strength reinforcement. Because actual forces in longitudinal reinforcement of structural walls may exceed calculated forces, reinforcement should be developed or spliced to reach the yield strength of the bar in tension. Termination of longitudinal (vertical) reinforcement in structural walls should be specified so that bars extend above elevations where they are no longer required to resist design flexure and axial force; extending bars ℓ_d above the next floor level is a practical approach to achieving this requirement. A limit of 12 ft is included for cases with large story heights. Bar terminations should be accomplished gradually over a wall height and should not be located close to critical sections where yielding of longitudinal reinforcement is expected,

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as a result of lateral displacements. The value of h_{sx} need not exceed 20 ft. Boundary regions include those within lengths specified in 18.10.6.4(a) and within a length equal to the wall thickness measured beyond the intersecting region(s) of connected walls.

(d) Mechanical splices of reinforcement shall conform to 18.2.7 and welded splices of reinforcement shall conform to 18.2.8.

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which typically occurs at the base of a wall with a uniform, or nearly uniform, cross section over the building height. Strain hardening of reinforcement results in spread of plasticity away from critical sections as lateral deformations increase. Research (Aaleti et al. 2013; Hardisty et al. 2015) shows that lap splices should be avoided in walls where flexural yielding is anticipated, for example at the base of walls, because they may lead to large localized strains and bar fractures. Figure R18.10.2.3 illustrates boundary regions where lap splices are not permitted.

At locations where yielding of longitudinal reinforcement is expected, the reinforcement is developed for $1.25f_y$ to account for the likelihood that the actual yield strength exceeds the specified yield strength of the bar, as well as the influence of strain hardening and cyclic load reversals. Where transverse reinforcement is used, development lengths for straight and hooked bars may be reduced as permitted in 25.4.2 and 25.4.3, respectively, because closely spaced transverse reinforcement improves the performance of splices and hooks subjected to repeated inelastic demands (ACI PRC-408.2).

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COMMENTARY

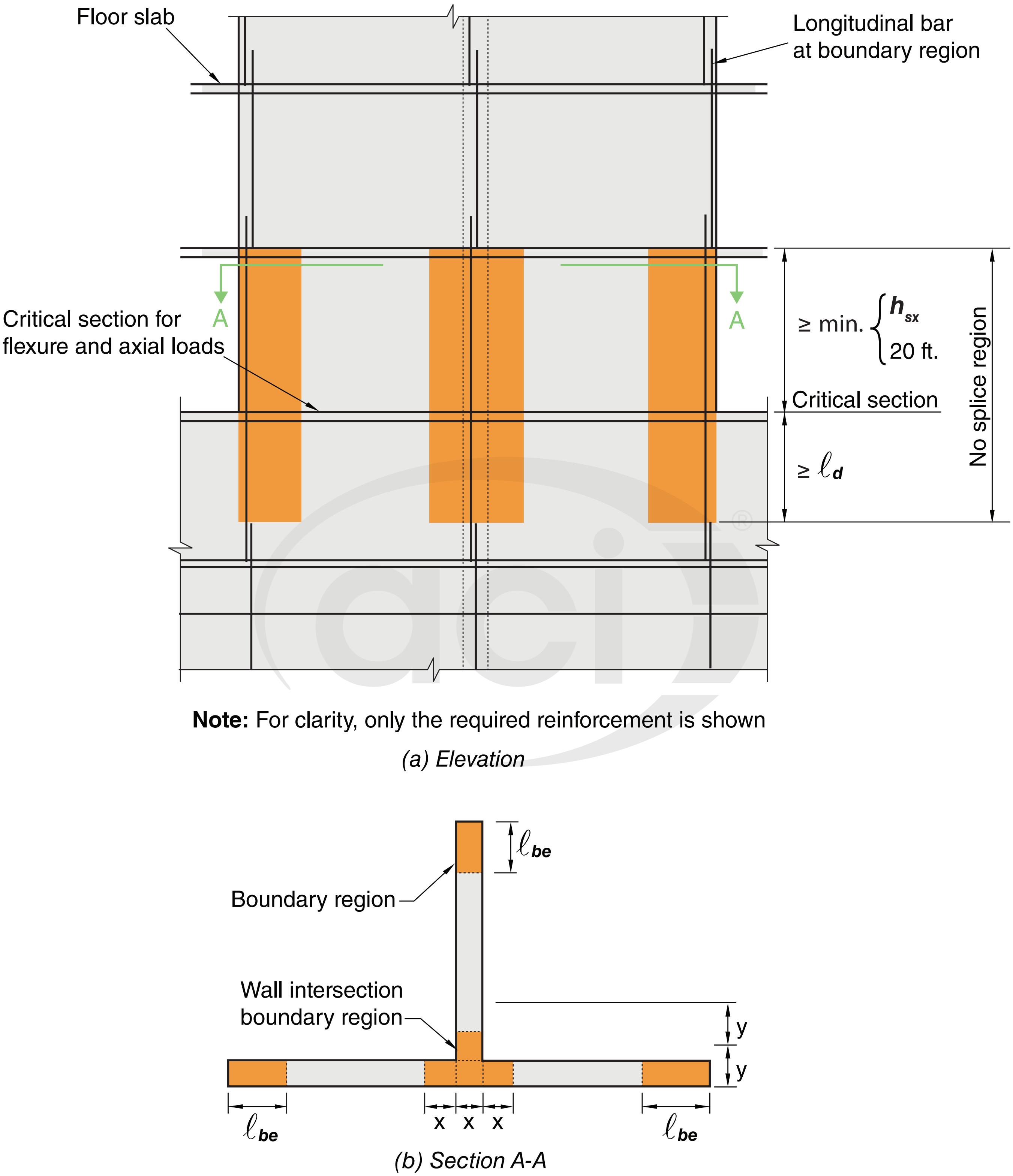

Fig. R18.10.2.3—Wall boundary regions within heights where lap splices are not permitted.

18.10.2.4 Walls or wall piers with $h_w/\ell_w \geq 2.0$ that are effectively continuous from the base of structure to top of wall and are designed to have a single critical section for

R18.10.2.4 This provision is based on the assumption that inelastic response of the wall is dominated by flexural action at a critical, yielding section. The wall should be propor-

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flexure and axial loads shall have longitudinal reinforcement at the ends of a vertical wall segment that satisfies (a) through (c).

(a) Longitudinal reinforcement ratio within $0.15\ell_w$ from the end of a vertical wall segment, and over a width equal to the wall thickness, shall be at least $6\sqrt{f_c'}/f_y$.

(b) The longitudinal reinforcement required by 18.10.2.4(a) shall extend vertically above and below the critical section at least the greater of ℓ_w and $M_u/3V_u$.

(c) No more than 50% of the reinforcement required by 18.10.2.4(a) shall be terminated at any one section.

COMMENTARY

tioned so that the critical section occurs where intended. If there is potential for more than one critical section, it is prudent to provide the minimum boundary reinforcement at all such sections.

The requirement for minimum longitudinal reinforcement in the ends of the wall is to promote the formation of well-distributed secondary flexural cracks in the wall plastic hinge region to achieve the required deformation capacity during earthquakes (Lu et al. 2017; Sritharan et al. 2014). Furthermore, significantly higher in-place concrete strengths than used in design calculations may be detrimental to the distribution of cracking. 18.10.2.4(a) specifies the required reinforcement ratio in the end tension zones, as shown for different wall sections in Fig. R18.10.2.4.

The longitudinal reinforcement required by 18.10.2.4(a) should be located at a critical section where concentrated yielding of longitudinal reinforcement is expected (typically the base of a cantilever wall) and must continue to a sufficient elevation of the wall to avoid a weak section adjacent to the intended plastic hinge region. A height above or below the critical section of $M_u/3V_u$ is used to identify the length over which yielding is expected.

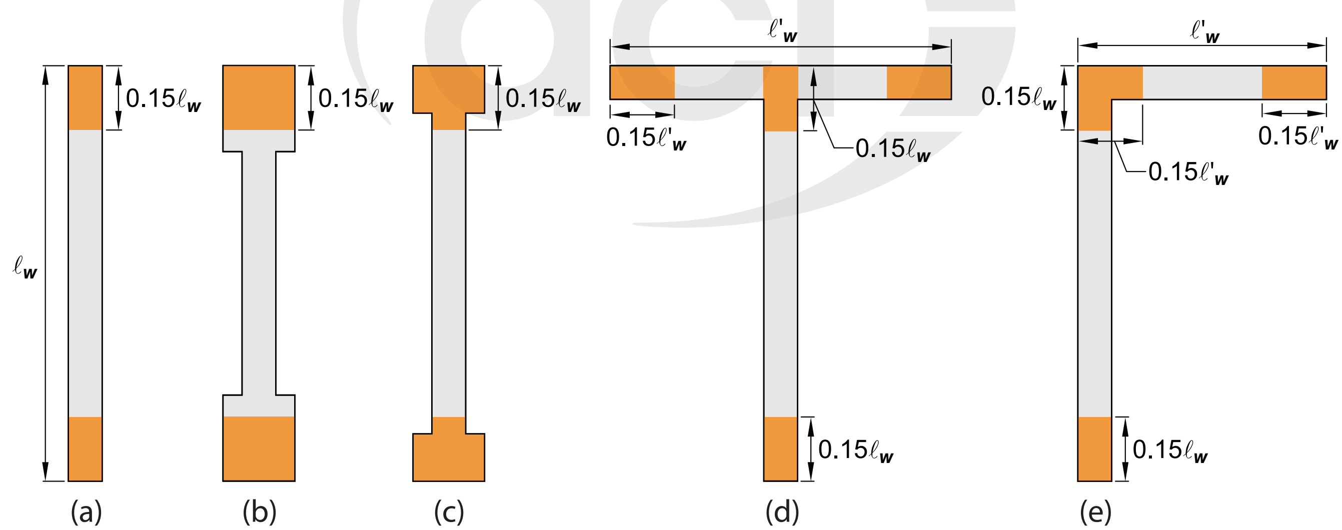

Fig. R18.10.2.4—Locations of longitudinal reinforcement required by 18.10.2.4(a) in different configurations of wall sections.

18.10.2.5 Reinforcement in coupling beams shall develop f_y in tension in accordance with 25.4, 25.5, and (a) and (b):

(a) If coupling beams are reinforced according to 18.6.3.1, longitudinal reinforcement shall develop $1.25f_y$ in tension in accordance with 25.4.

(b) If coupling beams are reinforced according to 18.10.7.4, diagonal reinforcement shall develop $1.25f_y$ in tension in accordance with 25.4.

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18.10.3 *Design forces*

18.10.3.1 Design shear forces for horizontal wall segments, including coupling beams, shall be in accordance with 18.10.7.

18.10.3.2 Design shear forces for wall piers shall be in accordance with 18.10.8.

18.10.3.3 Design shear forces for parts of walls not covered by 18.10.3.1 or 18.10.3.2 shall be in accordance with the requirements of 18.10.3.3.1 through 18.10.3.3.5.

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R18.10.3 *Design forces*

Numerous studies (Priestley et al. 2007; Pugh et al. 2017; Rodriguez et al. 2002) have shown that the actual shear experienced by a structural wall subjected to a design-basis earthquake may be greater than the shear obtained from linear analysis of the structure under code-prescribed earthquake-induced forces. The procedures of 18.10.3 may amplify wall shear for some walls designed by linear analysis methods. The amplification factors do not apply to wall piers (refer to 2.3 and Table R18.10.1) or horizontal wall segments including coupling beams because alternative approaches to determine design shears for those components are specified in 18.10.7, 18.10.8, and 21.2.4.1. Design shears determined by linear analysis procedures of the general building code are increased to account for (i) flexural overstrength at critical sections where yielding of longitudinal reinforcement is anticipated, as represented by the factor Ω_v, and (ii) dynamic amplification due to higher-mode effects, as represented by the factor ω_v (refer to Fig. R18.10.3.3). The factors apply only to the portion of wall shear V_{uEh} due to the horizontal seismic load effect E_h specified in the general building code. Design shear generally will be controlled by load combinations 5.3.1(e) or 5.3.1(g) in Table 5.3.1, whichever produces the greater value of design shear V_e.

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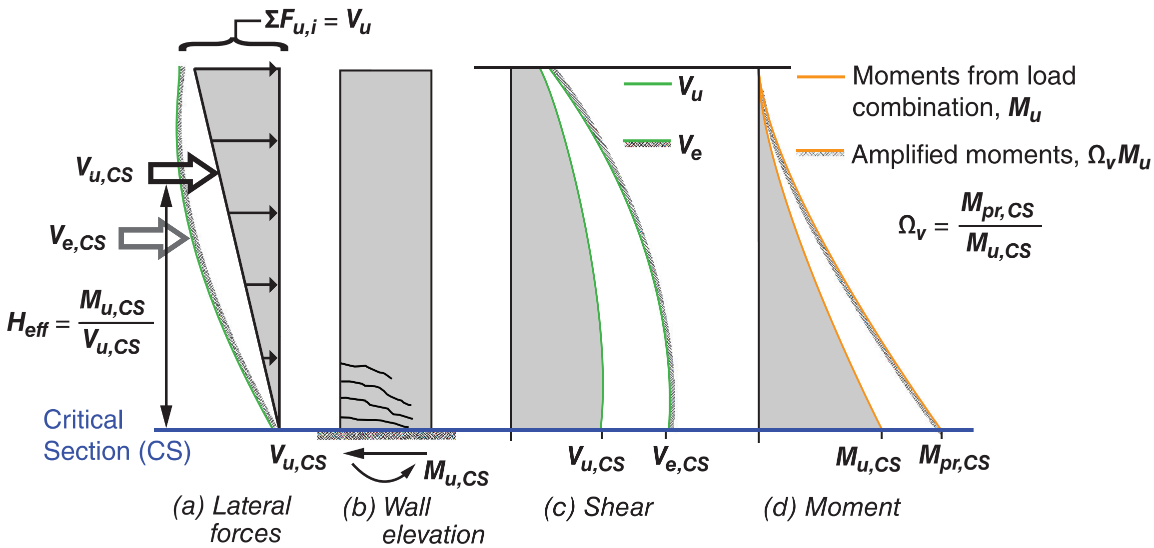

Fig. R18.10.3.3—Determination of shear demand for walls with $h_w/\ell_w \geq 2.0$ *(Moehle et al. 2011).*

18.10.3.3.1 If the wall design actions are determined in accordance with nonlinear dynamic analysis procedures satisfying Appendix A, design shear forces shall be as determined in Appendix A.

18.10.3.3.2 If V_{uEh} is determined by linear analysis procedures of the general building code, it shall be amplified by the product $\Omega_v\omega_v$, where Ω_v and ω_v are defined in 18.10.3.3.3 through 18.10.3.3.5.

18.10.3.3.3 Ω_v and ω_v shall be in accordance with Table 18.10.3.3.3. Alternatively, it shall be permitted to calculate $\Omega_v = M_{pr}/M_u$ at the critical section for flexure, where M_{pr} is calculated for axial force that includes the effects of E and the expected gravity loads, with expected gravity loads in accordance with ASCE/SEI 7 Section 16.3.2.

Table 18.10.3.3.3—Factors Ω_v and ω_v

Condition	Ω_v	ω_v
$h_{wcs}/\ell_w \leq 1.0$	1.0	1.0
$1.0 < h_{wcs}/\ell_w < 2.0$	Linear interpolation permitted between 1.0 and 1.5	
$h_{wcs}/\ell_w \geq 2.0$	1.5	$0.8 + 0.09\,h_n^{1/3} \geq 1.0$

R18.10.3.3 The factor Ω_v is intended to approximate the flexural overstrength ratio M_{pr}/M_u of the wall critical section, with M_{pr} based on axial forces due to E and expected gravity loads as specified in ASCE/SEI 7. While it is permitted to calculate this ratio directly from analysis of the wall critical section, Table 18.10.3.3.3 provides a simpler alternative. For walls with $h_{wcs}/\ell_w \leq 1.0$, a value of $\Omega_v = 1$ is permitted because low-aspect-ratio walls are unlikely to develop extensive flexural yielding. For walls with $h_{wcs}/\ell_w \geq 2.0$, yielding of the wall critical section is likely to produce flexural overstrength. The value of $\Omega_v = 1.5$ assumes that the wall is proportioned for moment strength using a strength reduction factor $\phi = 0.9$, that the provided moment strength ϕM_n closely matches the required moment strength M_u, and that longitudinal reinforcement reaches a tensile stress of $1.25f_y$ under earthquake shaking.

The dynamic amplification factor ω_v is derived from the similar factor in New Zealand Standard 3101 (2006). Dynamic amplification is not significant in walls with $h_{wcs}/\ell_w < 2$.

Design shear forces are amplified over the entire wall height, including portions of the wall below the critical section.

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18.10.3.3.4 If the general building code includes provisions to account for overstrength of the seismic-force-resisting system, it shall be permitted to take $\Omega_v \omega_v$ equal to Ω_o.

18.10.3.3.5 If $\Omega_v \omega_v = \Omega_o$, it shall be permitted to take the redundancy factor contained in the general building code equal to 1.0 for determination of V_{uEh}.

18.10.4 *Shear strength*

COMMENTARY

R18.10.3.3.5 Consistent with ASCE/SEI 7, it is permitted to take the redundancy factor as 1.0 where member design is for seismic load effects including the overstrength factor Ω_o.

R18.10.4 *Shear strength*

Equation (18.10.4.1) recognizes the higher shear strength of walls with lower moment-to-shear ratios (Hirosawa 1977; Joint ACI-ASCE Committee 326 [1962]; Barda et al. 1977; Rojas-Leon et al. 2024). The nominal shear strength is given in terms of the gross area of the section resisting shear, A_{cv}. For a rectangular section without openings, the term A_{cv} refers to the gross area of the cross section rather than to the product of the width and the effective depth.

In ACI CODE-318-19 Section 18.10.4, no limit was specified on the value of $\sqrt{f_c'}$. The limit on f_c' of 12,000 psi is based on review of test data for walls subjected to cyclic loading by Rojas-Leon et al. (2024).

A vertical wall segment refers to a part of a wall bounded horizontally by openings or by an opening and an edge. For an isolated wall or a vertical wall segment, ρ_t refers to horizontal reinforcement and ρ_ℓ refers to vertical reinforcement.

The ratio h_w/ℓ_w may refer to overall dimensions of a wall, or of a segment of the wall bounded by two openings, or an opening and an edge. The intent of 18.10.4.2 is to make certain that any segment of a wall is not assigned a unit strength greater than that for the entire wall. However, a wall segment with a ratio of h_w/ℓ_w higher than that of the entire wall should be proportioned for the unit strength associated with the ratio h_w/ℓ_w based on the dimensions for that segment.

To restrain the inclined cracks effectively, reinforcement included in ρ_t and ρ_ℓ should be appropriately distributed along the length and height of the wall (refer to 18.10.4.3). Chord reinforcement provided near wall edges in concentrated amounts for resisting bending moment is not to be included in determining ρ_t and ρ_ℓ. Within practical limits, shear reinforcement distribution should be uniform and at a small spacing.

If the factored shear force at a given level in a structure is resisted by several walls or several vertical wall segments of a perforated wall, the average unit shear strength assumed for the total available cross-sectional area is limited to the sum of $\alpha_{sh} 8\sqrt{f_c'}$ for those walls or wall segments with the additional requirement that the unit shear strength assigned to any single wall or vertical wall segment does not exceed $\alpha_{sh} 10\sqrt{f_c'}$ (refer to 18.10.4.4). The upper limit of strength to be assigned to any one member is imposed to limit the degree of redistribution of shear force. The term α_{sh} accounts for the higher unit shear stress that develops prior to diagonal compression failure in a wall with a compression flange

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18.10.4.1 V_n shall be calculated by:

$$V_n = (\alpha_c \lambda \sqrt{f_c'} + \rho_t f_{yt})A_{cv} \qquad (18.10.4.1)$$

where

$\alpha_c = 3$ for $h_w/\ell_w \leq 1.5$

$\alpha_c = 2$ for $h_w/\ell_w \geq 2.0$

It shall be permitted to linearly interpolate the value of α_c between 3 and 2 for $1.5 < h_w/\ell_w < 2.0$. The value of f_c' used in Eq. (18.10.4.1) and in 18.10.4.4 and 18.10.4.5 shall not exceed 12,000 psi.

18.10.4.2 In 18.10.4.1, the value of ratio h_w/ℓ_w used to calculate V_n for segments of a wall shall be the greater of the ratios for the entire wall and the segment of wall considered.

18.10.4.3 Walls shall have distributed shear reinforcement in two orthogonal directions in the plane of the wall. If h_w/ℓ_w does not exceed 2.0, reinforcement ratio ρ_ℓ shall be at least the reinforcement ratio ρ_t.

18.10.4.4 V_n shall not be taken greater than the sum of $\alpha_{sh}8\sqrt{f_c'}A_{cv}$ for all vertical wall segments sharing a common lateral force. For any one of the individual vertical wall segments, V_n shall not be taken greater than $\alpha_{sh}10\sqrt{f_c'}A_{cw}$, where A_{cw} is the area of concrete section of the individual vertical wall segment considered. The term α_{sh} is determined as

$$0.7\left(1 + \frac{(b_w + b_{cf})t_{cf}}{A_{cs}}\right)^2 \leq 1.2 \qquad (18.10.4.4)$$

where b_{cf} is determined according to 18.10.5.2 and A_{cx} shall be taken as A_{cv} or A_{cw}, as applicable. The value of α_{sh} need not be taken less than 1.0. It shall be permitted to take α_{sh} = 1.0.

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(Rojas-Leon et al. 2024). If the term $b_{cf}t_{cf}$ is different at each edge (boundary) of a wall or if a flange does not exist at one end (for example, a T-shaped wall cross section), then the wall shear stress limit is evaluated independently for each load combination depending on the direction of the shear demand or the wall shear stress limit may be based on the smaller value of $b_{cf}t_{cf}$. For a barbell-shaped wall cross section, b_{cf} is the width of the boundary column minus the web width b_w and t_{cf} is the depth of the boundary column.

Horizontal wall segments in 18.10.4.5 refer to wall sections between two vertically aligned openings (refer to Fig. R18.10.4.5). It is, in effect, a vertical wall segment rotated through 90 degrees. A horizontal wall segment is also referred to as a coupling beam when the openings are aligned vertically over the building height. When designing a horizontal wall segment or coupling beam, ρ_t refers to vertical reinforcement and ρ_ℓ refers to horizontal reinforcement.

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18.10.4.5 For horizontal wall segments and coupling beams, V_n shall not be taken greater than $10\sqrt{f_c'}A_{cw}$, where A_{cw} is the area of concrete section of a horizontal wall segment or coupling beam.

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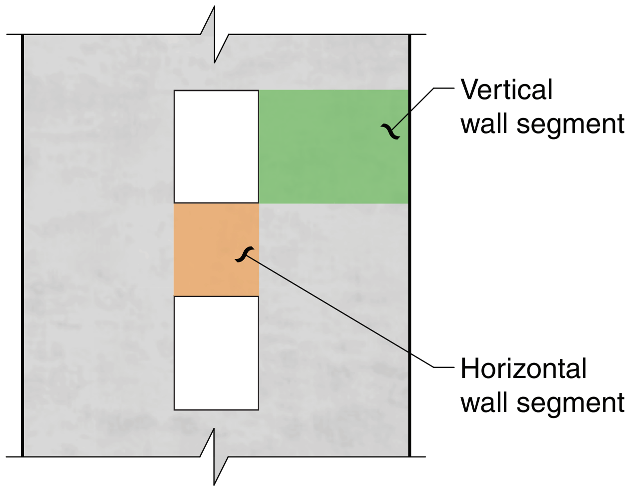

Fig. R18.10.4.5—Wall with openings.

18.10.5 *Design for flexure and axial force*

R18.10.5 *Design for flexure and axial force*

18.10.5.1 Structural walls and portions of such walls subject to combined flexure and axial loads shall be designed in accordance with 22.4. Concrete and developed longitudinal reinforcement within effective flange widths, boundary elements, and the wall web shall be considered effective. The effects of openings shall be considered.

R18.10.5.1 Flexural strength of a wall or wall segment is determined according to procedures commonly used for columns. Strength should be determined considering the applied axial and lateral forces. Reinforcement concentrated in boundary elements and distributed in flanges and webs should be included in the strength calculations based on a strain compatibility analysis. The foundation supporting the wall should be designed to resist the wall boundary and web forces. For walls with openings, the influence of the opening or openings on flexural and shear strengths is to be considered and a load path around the opening or openings should be verified. Capacity-design concepts and the strut-and-tie method may be useful for this purpose (Taylor et al. 1998).

18.10.5.2 Unless a more detailed analysis is performed, effective flange widths of flanged sections shall extend from the face of the web a distance equal to the lesser of one-half the distance to an adjacent wall web and 25% of the total wall height above the section under consideration.

R18.10.5.2 Where wall sections intersect to form L-, T-, C-, or other cross-sectional shapes, the influence of the flange on the behavior of the wall should be considered by selecting appropriate flange widths. Tests (Wallace 1996) show that effective flange width increases with increasing drift level and the effectiveness of a flange in compression differs from that for a flange in tension. The value used for the effective compression flange width has little effect on the strength and deformation capacity of the wall; therefore, to simplify design, a single value of effective flange width based on an estimate of the effective tension flange width is used in both tension and compression.

18.10.6 *Boundary elements of special structural walls*

R18.10.6 *Boundary elements of special structural walls*

18.10.6.1 The need for special boundary elements at the edges of structural walls shall be evaluated in accordance with 18.10.6.2 or 18.10.6.3. The requirements of 18.10.6.4 and 18.10.6.5 shall also be satisfied.

R18.10.6.1 Two design approaches for evaluating detailing requirements at wall boundaries are included in 18.10.6.1. Provision 18.10.6.2 allows the use of displacement-based design of walls, in which the structural details are determined directly on the basis of the expected lateral

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18.10.6.2 Walls or wall piers with $h_{wcs}/\ell_w \geq 2.0$ that are effectively continuous from the base of structure to top of wall and are designed to have a single critical section for flexure and axial loads shall satisfy (a) and (b):

(a) Compression zones shall be reinforced with special boundary elements where

$$\frac{1.5\delta_u}{h_{wcs}} \geq \frac{\ell_w}{600_c} \qquad (18.10.6.2a)$$

and c corresponds to the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the direction of the design displacement δ_u. Ratio δ_u/h_{wcs} shall not be taken less than 0.005.

(b) If special boundary elements are required by (a), then (i) and either (ii) or (iii) shall be satisfied.

(i) Special boundary element transverse reinforcement shall extend vertically above and below the critical section at least the greater of ℓ_w and $M_u/4V_u$, except as permitted in 18.10.6.4(j).

(ii) $b \geq \sqrt{c\,\ell_w/40}$

(iii) $\delta_c/h_{wcs} \geq 1.5\delta_u/h_{wcs}$, where:

$$\frac{\delta}{h_{wcs}} = \frac{1}{100}\left(4 - \frac{1}{50} - \left(\frac{\ell_w}{b}\right)\left(\frac{c}{b}\right) - \frac{V_e}{8\sqrt{f_c'}A_{cv}}\right) \qquad (18.10.6.2b)$$

The value of $\delta_c h_{wcs}$ in Eq. (18.10.6.2b) need not be taken less than 0.015.

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displacements of the wall. The provisions of 18.10.6.3 are similar to those of the 1995 Code, and have been retained because they are conservative for assessing required transverse reinforcement at wall boundaries for many walls. Provisions 18.10.6.4 and 18.10.6.5 apply to structural walls designed by either 18.10.6.2 or 18.10.6.3.

R18.10.6.2 This section is based on the assumption that inelastic response of the wall is dominated by flexural action at a critical, yielding section. The wall should be proportioned and reinforced so that the critical section occurs where intended.

Equation (18.10.6.2a) follows from a displacement-based approach (Moehle 1992; Wallace and Orakcal 2002). The approach assumes that special boundary elements are required to confine the concrete where the strain at the extreme compression fiber of the wall exceeds a critical value when the wall is displaced to 1.5 times the design displacement. Consistent with a displacement-based design approach, the design displacement in Eq. (18.10.6.2a) is taken at the top of the wall, and the wall height is taken as the height above the critical section. The multiplier of 1.5 on design displacement was added to Eq. (18.10.6.2) in the 2014 Code to produce detailing requirements more consistent with the building code performance intent of a low probability of collapse in Maximum Considered Earthquake level shaking. The lower limit of 0.005 on the quantity δ_u/h_{wcs} requires special boundary elements if wall boundary longitudinal reinforcement tensile strain does not reach approximately twice the limit used to define tension-controlled beam sections according to 21.2.2. The lower limit of 0.005 on the quantity δ_u/h_{wcs} requires moderate wall deformation capacity for stiff buildings.

The neutral axis depth c in Eq. (18.10.6.2) is the depth calculated according to 22.2 corresponding to development of nominal flexural strength of the wall when displaced in the same direction as δ_u. The axial load is the factored axial load that is consistent with the design load combination that produces the design displacement δ_u.

The height of the special boundary element is based on estimates of plastic hinge length and extends beyond the zone over which yielding of tension reinforcement and spalling of concrete are likely to occur.

Equation (18.10.6.2b) is based on the mean top-of-wall drift capacity at 20% loss of lateral strength proposed by Abdullah and Wallace (2019). The requirement that drift capacity exceed 1.5 times the drift demand results in a low probability of strength loss for the design earthquake. The expression for b in (ii) is derived from Eq. (18.10.6.2b), assuming values of $V_u/(8A_{cv}\sqrt{f_c'})$ and δ_u/h_{wcs} of approximately 1.0 and 0.015, respectively. If b varies over c, an average or representative value of b should be used. For example, at the flanged end of a wall, b should be taken equal to the effective flange width defined in 18.10.5.2, unless c extends into the web, then a weighted average should be used for b. At the

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end of a wall without a flange, b should be taken equal to the wall thickness. If the drift capacity does not exceed the drift demand for a trial design, then changes to the design are required to increase wall drift capacity, reduces wall drift demand, or both, such that drift capacity exceeds drift demand for each wall in a given building.

18.10.6.3 Structural walls not designed in accordance with 18.10.6.2 shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to load combinations including earthquake effects E, exceeds $0.2f_c'$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f_c'$. Stresses shall be calculated for the factored loads using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as given in 18.10.5.2 shall be used.

R18.10.6.3 By this procedure, the wall is considered to be acted on by gravity loads and the maximum shear and moment induced by earthquake in a given direction. Under this loading, the compressed boundary at the critical section resists the tributary gravity load plus the compressive resultant associated with the bending moment.

Recognizing that this loading condition may be repeated many times during the strong motion, the concrete is to be confined where the calculated compressive stresses exceed a nominal critical value equal to $0.2f_c'$. The stress is to be calculated for the factored forces on the section assuming linear response of the gross concrete section. The compressive stress of $0.2f_c'$ is used as an index value and does not necessarily describe the actual state of stress that may develop at the critical section under the influence of the actual inertia forces for the anticipated earthquake intensity.

18.10.6.4 If special boundary elements are required by 18.10.6.2 or 18.10.6.3, (a) through (k) shall be satisfied:

(a) The boundary element shall extend horizontally from the extreme compression fiber a distance at least the greater of $c - 0.1\ell_w$ and $c/2$, where c is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with δ_u.

(b) Width of the flexural compression zone, b, over the horizontal distance calculated by 18.10.6.4(a), including flange if present, shall be at least $h_u/16$.

(c) For walls or wall piers with $h_w/\ell_w \geq 2.0$ that are effectively continuous from the base of structure to top of wall, designed to have a single critical section for flexure and axial loads, and with $c/\ell_w \geq 3/8$, width of the flexural compression zone b over the length calculated in 18.10.6.4(a) shall be greater than or equal to 12 in.

(d) In flanged sections, the boundary element shall include the effective flange width in compression and shall extend at least 12 in. into the web.

(e) The boundary element transverse reinforcement shall satisfy 18.7.5.2(a) through (d) and 18.7.5.3, except the transverse reinforcement spacing limit of 18.7.5.3(a) shall be one-third of the least dimension of the boundary element. The maximum vertical spacing of transverse reinforcement in the boundary element shall also not exceed that in Table 18.10.6.5(b).

(f) Spacing h_x between laterally supported longitudinal bars around the perimeter of the boundary element shall not exceed the lesser of 14 in. and $(2/3)b$. Lateral support shall be provided by a seismic hook of a crosstie or corner of a hoop. Unless (i) or (ii) is satisfied, the length of the

R18.10.6.4 The horizontal dimension of the special boundary element is intended to extend at least over the length where the concrete compressive strain exceeds the critical value. For flanged wall sections, including box shapes, L-shapes, and C-shapes, the calculation to determine the need for special boundary elements should include a direction of lateral load consistent with the orthogonal combinations defined in ASCE/SEI 7. The value of $c/2$ in 18.10.6.4(a) is to provide a minimum length of the special boundary element. Good detailing practice is to arrange the longitudinal reinforcement and the confinement reinforcement such that all primary longitudinal reinforcement at the wall boundary is supported by transverse reinforcement.

A slenderness limit is introduced into the 2014 edition of the Code based on lateral instability failures of slender wall boundaries observed in recent earthquakes and tests (Wallace 2012; Wallace et al. 2012). For walls with large cover, where spalling of cover concrete would lead to a significantly reduced section, increased boundary element thickness should be considered.

A value of $c/\ell_w \geq 3/8$ is used to define a wall critical section that is not tension-controlled according to 21.2.2. A minimum wall thickness of 12 in. is imposed to reduce the likelihood of lateral instability of the compression zone after spalling of cover concrete.

Where flanges are highly stressed in compression, the web-to-flange interface is likely to be highly stressed and may sustain local crushing failure unless special boundary element transverse reinforcement extends into the web (Fig. R18.10.6.4.4b).

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hoop legs shall not exceed $2b_c$, and adjacent hoops shall overlap at least the lesser of 6 in. and $(2/3)b$:

(i) $b \geq \sqrt{\ell_w c/40}$ and $\delta_u/h_{wcs} < 0.012$

(ii) A flange is provided within depth c with a total width at least $2b_w$ and a thickness t_f at least $b_w/2$

(g) The amount of transverse reinforcement shall be in accordance with Table 18.10.6.4(g).

Table 18.10.6.4(g)—Transverse reinforcement for special boundary elements

Transverse reinforcement	Applicable expressions		
A_{sh}/sb_c for rectilinear hoop	Greater of	$0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$	(a)
		$0.09\frac{f_c'}{f_{yt}}$	(b)
ρ_s for spiral or circular hoop	Greater of	$0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$	(c)
		$0.12\frac{f_c'}{f_{yt}}$	(d)

(h) Concrete within the thickness of the floor system at the special boundary element location shall have specified compressive strength at least 0.7 times f_c' of the wall.

(i) For a distance above and below the critical section specified in 18.10.6.2(b), web vertical reinforcement shall have lateral support provided by the corner of a hoop or by a crosstie with seismic hooks at each end. Hoops and crossties shall have a vertical spacing not to exceed 12 in. and diameter satisfying 25.7.2.2. Alternatively, it shall be permitted to use crossties with a 90-degree hook at one end and a seismic hook at the other end, with the crossties alternated end for end along the length and the height of the web if vertical spacing of crossties does not exceed 9 in.

(j) Where the critical section occurs at the wall base, the boundary element transverse reinforcement at the wall base shall extend into the support at least ℓ_d, in accordance with 18.10.2.3, of the largest longitudinal reinforcement in the special boundary element. Where the special boundary element terminates on a footing, mat, or pile cap, special boundary element transverse reinforcement shall extend at least 12 in. into the footing, mat, or pile cap, unless a greater extension is required by 18.13.2.4.

(k) Horizontal reinforcement in the wall web shall extend to within 6 in. of the end of the wall. Reinforcement shall develop f_y in tension at the face of the confined core of the boundary element using standard hooks or heads. Where the confined boundary element has sufficient length to develop the horizontal web reinforcement, and $A_s f_y/s$ of the horizontal web reinforcement does not exceed $A_s f_{yt}/s$ of the boundary element transverse reinforcement parallel to the horizontal web reinforcement, it shall be permitted to terminate the horizontal web reinforcement without a standard hook or head.

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Required transverse reinforcement at wall boundaries is based on column provisions. Expression (a) of Table 18.10.6.4(g) was applied to wall special boundary elements prior to the 1999 edition of the Code. It is reinstated in the 2014 edition of the Code due to concerns that expression (b) of Table 18.10.6.4(g) by itself does not provide adequate transverse reinforcement for thin walls where concrete cover accounts for a significant portion of the wall thickness. For wall special boundary elements having rectangular cross section, A_g and A_{ch} in expressions (a) and (c) in Table 18.10.6.4(g) are defined as $A_g = \ell_{be}b$ and $A_{ch} = b_{c1}b_{c2}$, where dimensions are shown in Fig. R18.10.6.4a. This considers that concrete spalling is likely to occur only on the exposed faces of the confined boundary element. Tests (Thomsen and Wallace 2004) show that adequate performance can be achieved using vertical spacing greater than that permitted by 18.7.5.3(a). The limits on spacing between laterally supported longitudinal bars are intended to provide more uniform spacing of hoops and crossties for thin walls.

Configuration requirements for boundary element transverse reinforcement and crossties for web longitudinal reinforcement are summarized in Fig. R18.10.6.4a. Multiple overlapping hoops at elongated boundary elements are more effective in restraining bar buckling and confining the concrete than a single, elongated hoop with multiple crossties that have alternating 90- and 135-degree hooks (Segura and Wallace 2018; Welt et al. 2017; Arteta 2015).Overlapping hoops are not required if the drift demand for anticipated design-level earthquake shaking is low relative to the drift capacity, δ_c. Out-of-plane, lateral instability failure is more likely to occur at the boundaries of planar walls with relatively slender, deep compression zones—for example, where $(c/b)(\ell_w/b)$ is greater than approximately 40, or at the web boundary opposite a flanged boundary for a wall with a T-,C-, or L-shaped cross section (Abdullah and Wallace 2020). At a wall boundary where a web and flange intersect, use of overlapping hoops is not required in either the flange or web (Fig. R18.10.6.4b) because the flange provides lateral support to the wall web. A web hoop, however, may overlap with a flange hoop at the web-flange intersection. The geometric limits in 18.10.6.4f(ii) are based on judgment from observed damage of wall boundaries.

These tests also show that loss of axial load-carrying capacity of a wall can occur immediately following damage to the wall boundary elements if web vertical reinforcement within the plastic hinge region is not restrained. Use of web crossties outside of boundary elements also results in a less abrupt transition in transverse reinforcement used to provide concrete confinement and restrain buckling of longitudinal reinforcement, which addresses potential increases in the neutral axis depth due to shear (diagonal compression) and uncertainties in axial load.

Requirements for vertical extensions of boundary elements are summarized in Fig. R18.10.6.4d (Moehle et al. 2011).

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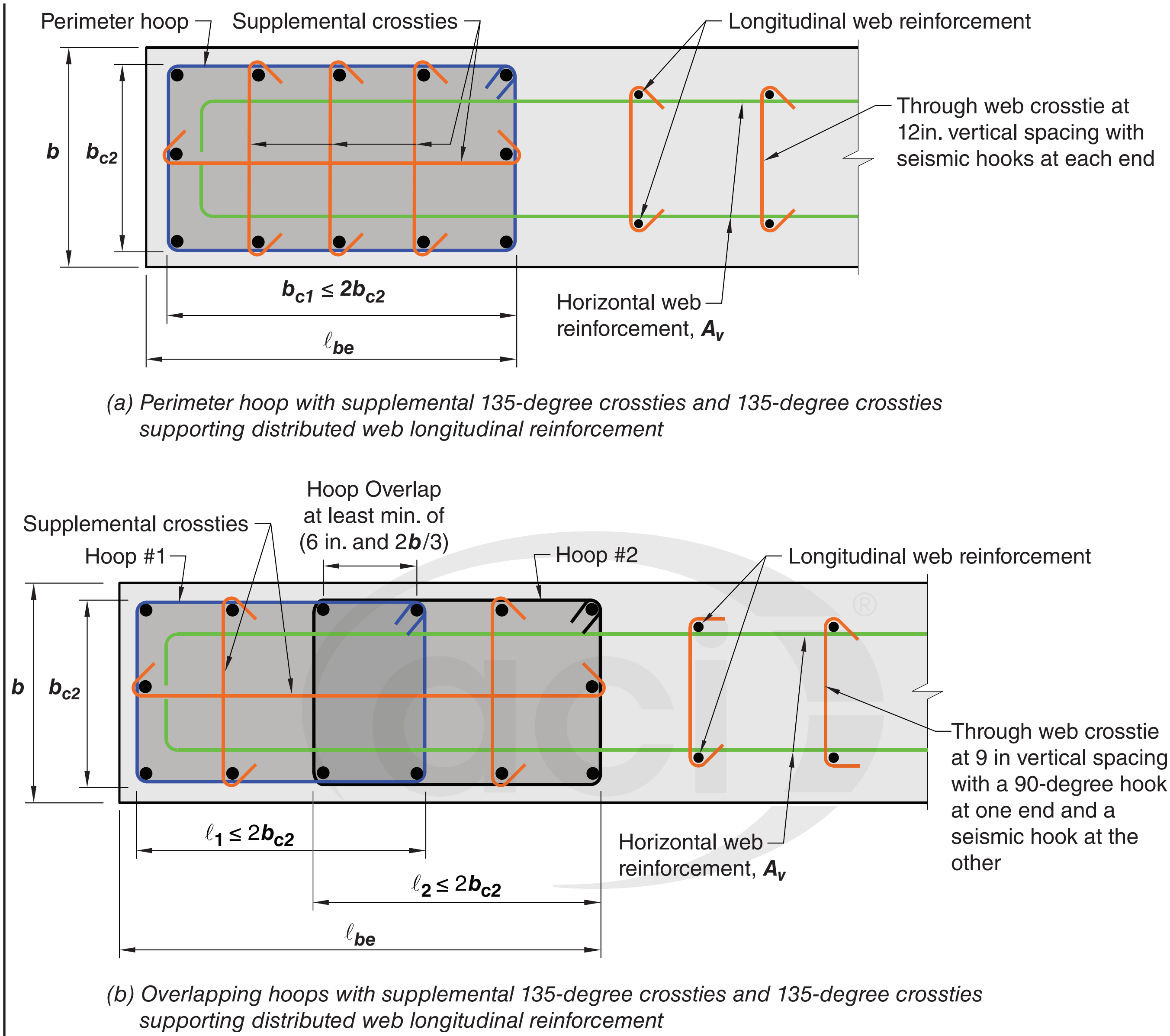

Fig. R18.10.6.4a—Configurations of boundary transverse reinforcement and web crossties.

The horizontal reinforcement in a structural wall with low shear-to-moment ratio resists shear through truss action, with the horizontal bars acting like the stirrups in a beam. Thus, the horizontal bars provided for shear reinforcement must be developed within the confined core of the boundary element and extended as close to the end of the wall as cover requirements and proximity of other reinforcement permit. The requirement that the horizontal web reinforcement be anchored within the confined core of the boundary element and extended to within 6 in. from the end of the wall applies to all horizontal bars whether straight, hooked, or headed, as illustrated in Fig. R18.10.6.4c.

The requirements in 18.10.2.4 apply to the minimum longitudinal reinforcement in the ends of walls, including those with special boundary elements.

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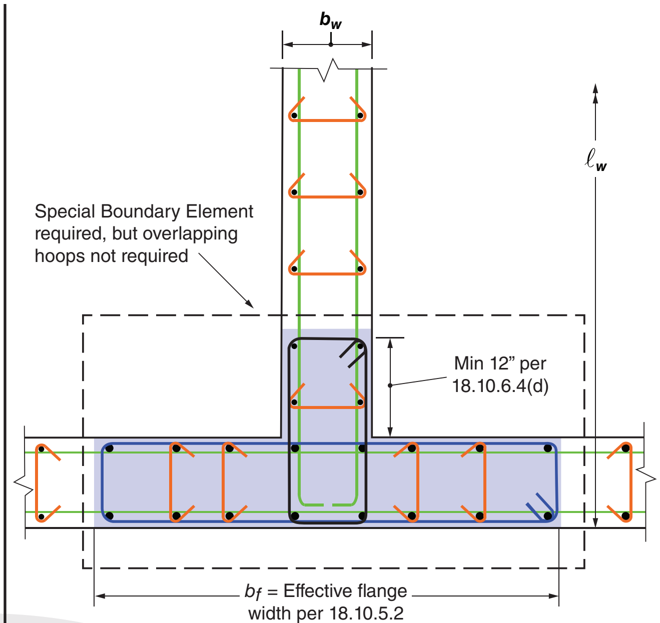

Fig. R18.10.6.4b—Example configuration for a case if a special boundary element is required for a flanged wall

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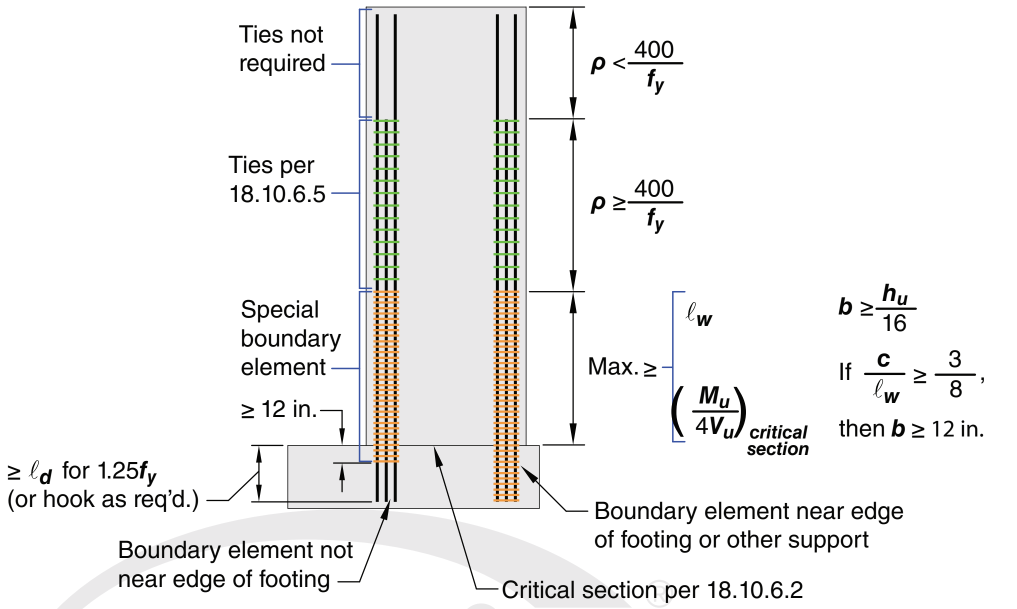
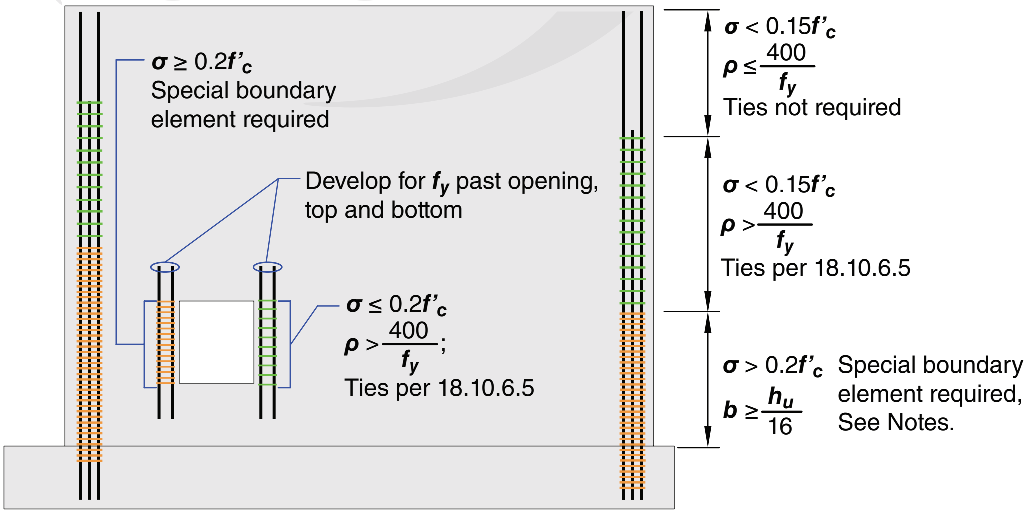


(a) Wall with $h_w/\ell_w \geq 2.0$ and a single critical section controlled by flexure and axial load designed using 18.10.6.2, 18.10.6.4, and 18.10.6.5



Notes: Requirement for special boundary element is triggered if maximum extreme fiber compressive stress $\sigma \geq 0.2f'_c$. Once triggered, the special boundary element extends until $\sigma < 0.15f'_c$. Since $h_w/\ell_w \leq 2.0$, 18.10.6.4(c) does not apply.

(b) Wall and wall pier designed using 18.10.6.3, 18.10.6.4, and 18.10.6.5.

Fig. R18.10.6.4c—Summary of boundary element requirements for special walls.

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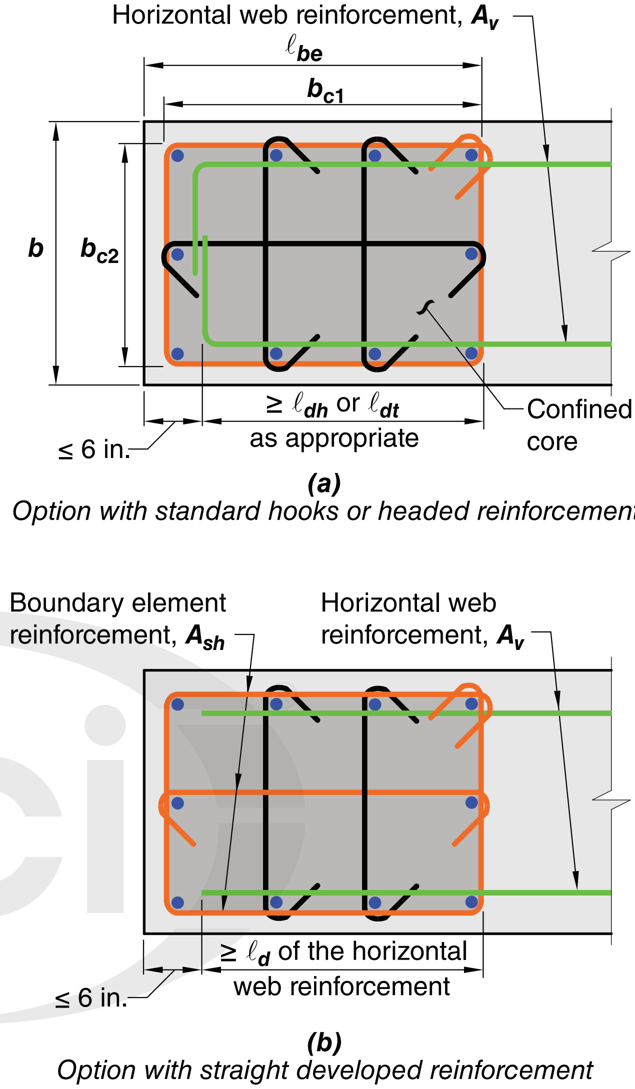

Fig. R18.10.6.4d—Development of wall horizontal reinforcement in confined boundary element.

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18.10.6.5 Where special boundary elements are not required by 18.10.6.2 or 18.10.6.3, (a) and (b) shall be satisfied:

(a) Except where $\omega_v \Omega_v V_u$ in the plane of the wall is less than $\lambda\sqrt{f_c'}A_{cv}$, horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

(b) If the maximum longitudinal reinforcement ratio at the wall boundary exceeds $400/f_y$, boundary transverse reinforcement shall satisfy 18.7.5.2(a) through (e) over the length calculated in accordance with 18.10.6.4(a). At corners where a wall web and flange intersect, boundary transverse reinforcement shall extend into the web and the flange at least 12 in. The vertical spacing of transverse reinforcement at the wall boundary shall be in accordance with Table 18.10.6.5(b).

Table 18.10.6.5(b)—Maximum vertical spacing of transverse reinforcement at wall boundary

Grade of primary flexural reinforcement	Transverse reinforcement required	Maximum vertical spacing of transverse reinforcement[1]	
60	Within the greater of ℓ_w and $M_u/4V_u$ above and below critical sections[2]	Lesser of:	$6d_b$
			6 in.
	Other locations	Lesser of:	$8d_b$
			8 in.
80	Within the greater of ℓ_w and $M_u/4V_u$ above and below critical sections[2]	Lesser of:	$5d_b$
			6 in.
	Other locations	Lesser of:	$6d_b$
			6 in.
100	Within the greater of ℓ_w and $M_u/4V_u$ above and below critical sections[2]	Lesser of:	$4d_b$
			6 in.
	Other locations	Lesser of:	$6d_b$
			6 in.

[1]In this table, d_b is the diameter of the smallest primary flexural reinforcing bar.
[2]Critical sections are defined as locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements.

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R18.10.6.5 Cyclic load reversals may lead to buckling of boundary longitudinal reinforcement even in cases where the demands on the boundary of the wall do not require special boundary elements. For walls with moderate amounts of boundary longitudinal reinforcement, ties are required to inhibit buckling. The longitudinal reinforcement ratio is intended to include only the reinforcement at the wall boundary, as indicated in Fig. R18.10.6.5a. A greater spacing of ties relative to 18.10.6.4(e) is allowed due to the lower deformation demands on the walls. Requirements of 18.10.6.5 apply over the entire wall height and are summarized in Fig. R18.10.6.4c for cases where special boundary elements are required (Moehle et al. 2011).

The addition of hooks or U-stirrups at the ends of horizontal wall reinforcement provides anchorage so that the reinforcement will be effective in resisting shear forces. It will also tend to inhibit the buckling of the vertical edge reinforcement. In walls with low in-plane shear, the development of horizontal reinforcement is not necessary.

Limits on spacing of transverse reinforcement are intended to prevent bar buckling until reversed cyclic strains extend well into the inelastic range. To achieve similar performance capability, smaller spacing is required for higher-strength longitudinal reinforcement.

To address potential significant tensile or compressive strain demands under biaxial loading, transverse reinforcement is required at corners where a wall web and flange intersect, as shown in Fig. R18.10.6.5b.

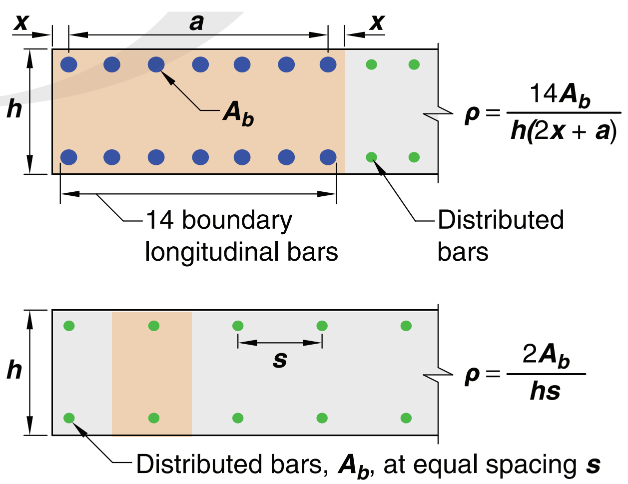

Fig. R18.10.6.5a—Longitudinal reinforcement ratios for typical wall boundary conditions.

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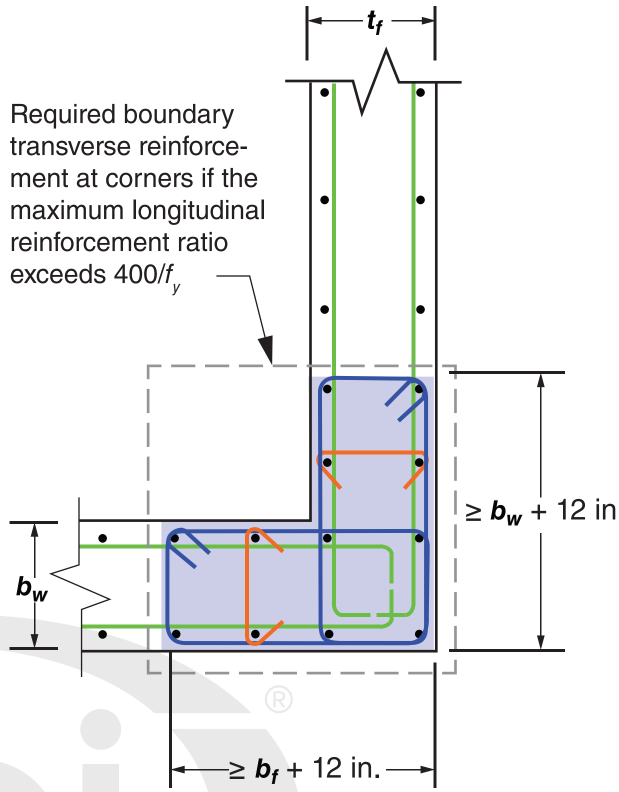

Fig. R18.10.6.5b—Transverse reinforcement at the intersection of a wall web and flange.

18.10.7 *Coupling beams*

R18.10.7 *Coupling beams*

Coupling beams connecting structural walls can provide stiffness and energy dissipation. In many cases, geometric limits result in coupling beams that are deep in relation to their clear span. Deep coupling beams may be controlled by shear and may be susceptible to strength and stiffness deterioration under earthquake loading. Test results (Paulay and Binney 1974; Barney et al. 1980) have shown that confined diagonal reinforcement provides adequate resistance in deep coupling beams.

Experiments show that diagonally oriented reinforcement is effective only if the bars are placed with a large inclination. Therefore, diagonally reinforced coupling beams are restricted to beams having aspect ratio $\ell_n/h < 4$. The 2008 edition of the Code was changed to clarify that coupling beams of intermediate aspect ratio can be reinforced according to 18.6.3 through 18.6.5.

Diagonal bars should be placed approximately symmetrically in the beam cross section, in two or more layers. The diagonally placed bars are intended to provide the entire shear and corresponding moment strength of the beam. Designs deriving their moment strength from combinations of diagonal and longitudinal bars are not covered by these provisions.

Two confinement options are described. According to 18.10.7.4(c), each diagonal element consists of a cage of longitudinal and transverse reinforcement, as shown in Fig. R18.10.7a. Each cage contains at least four diagonal

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bars and confines a concrete core. The requirement on side dimensions of the cage and its core is to provide adequate stability to the cross section when the bars are loaded beyond yielding. The minimum dimensions and required reinforcement clearances may control the wall width. Revisions were made in the 2008 Code to relax spacing of transverse reinforcement confining the diagonal bars, to clarify that confinement is required at the intersection of the diagonals, and to simplify design of the longitudinal and transverse reinforcement around the beam perimeter; beams with these new details are expected to perform acceptably. The expressions for transverse reinforcement A_{sh} are based on ensuring compression capacity of an equivalent column section is maintained after spalling of cover concrete.

Limits on transverse reinforcement spacing in Table 18.10.7.4 are intended to provide adequate support of diagonal and primary flexural reinforcement to control bar buckling.

Section 18.10.7.4(d) describes a second option for confinement of the diagonals introduced in the 2008 Code (refer to Fig. R18.10.7b). This second option is to confine the entire beam cross section instead of confining the individual diagonals. This option can considerably simplify field placement of hoops, which can otherwise be especially challenging where diagonal bars intersect each other or enter the wall boundary.

For coupling beams not used as part of the lateral-force-resisting system, the requirements for diagonal reinforcement may be waived.

Test results (Barney et al. 1980) demonstrate that beams reinforced as described in 18.10.7 have adequate ductility at shear forces exceeding $10\sqrt{f_c'}b_w d$. Consequently, the use of a limit of $10\sqrt{f_c'}A_{cw}$ provides an acceptable upper limit.

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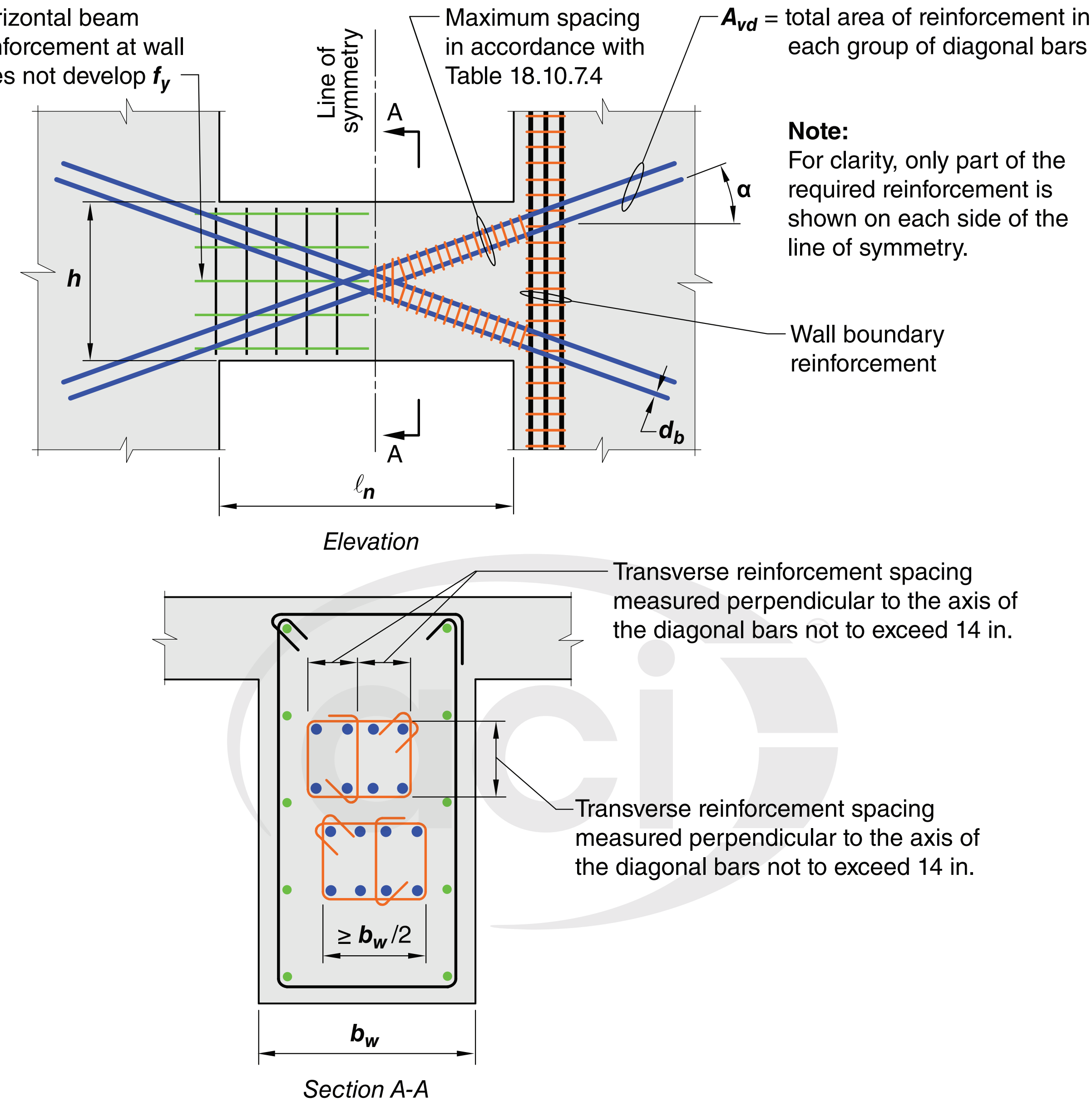

Fig. R18.10.7a—Confinement of individual diagonals in coupling beams with diagonally oriented reinforcement. Wall boundary reinforcement shown on one side only for clarity.

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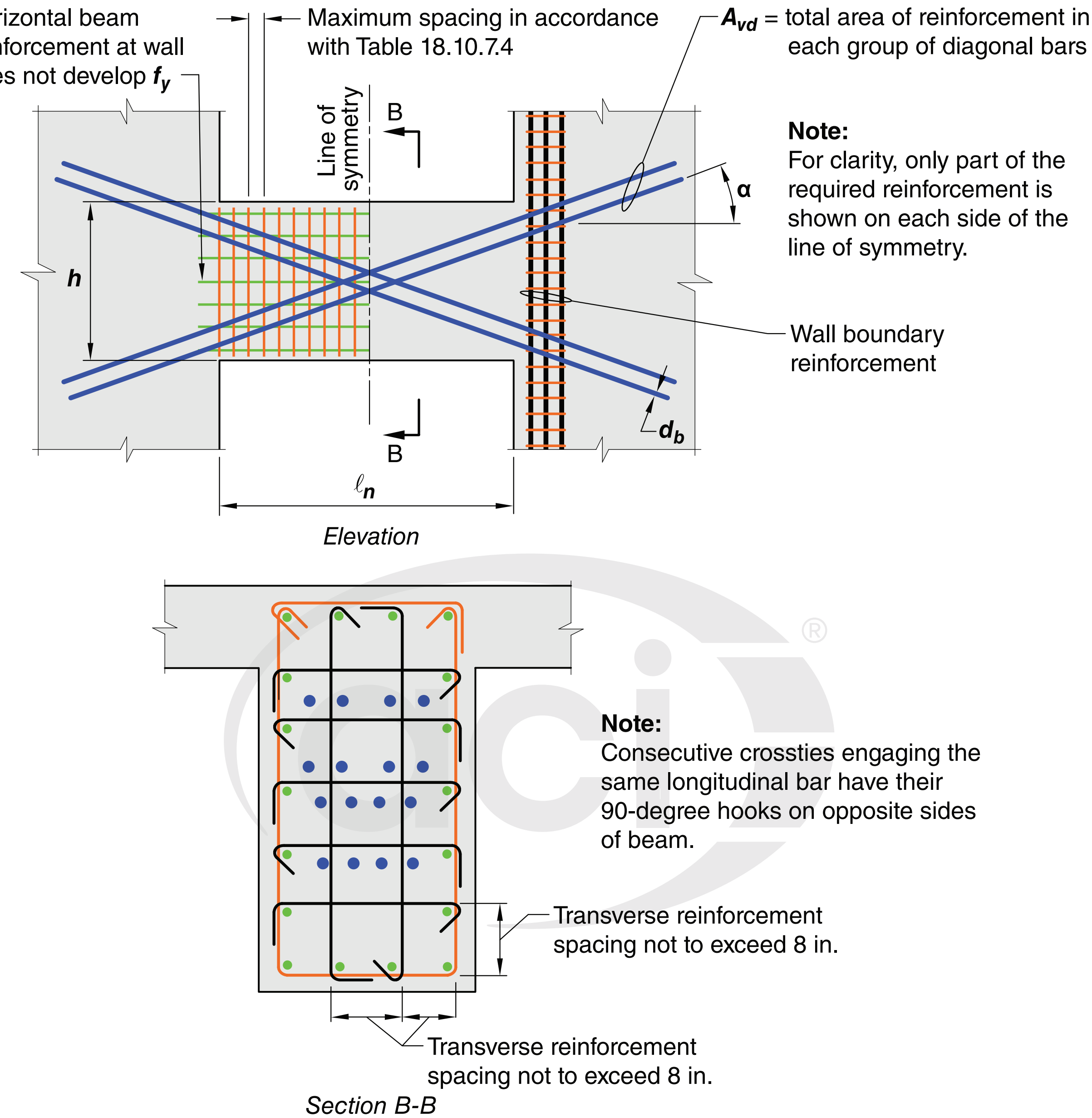

Fig. R18.10.7b—Full confinement of diagonally reinforced concrete beam section in coupling beams with diagonally oriented reinforcement. Wall boundary reinforcement shown on one side only for clarity.

18.10.7.1 Coupling beams with $(\ell_n/h) \geq 4$ shall satisfy the requirements of 18.6, with the wall boundary interpreted as being a column. The provisions of 18.6.2.1(b) and (c) need not be satisfied if it can be shown by analysis that the beam has adequate lateral stability.

18.10.7.2 Coupling beams with $(\ell_n/h) < 2$ and with $V_u \geq 4\lambda\sqrt{f_c'}A_{cw}$ shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load-carrying ability of the structure, the egress from the structure, or the

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integrity of nonstructural components and their connections to the structure.

18.10.7.3 Coupling beams not governed by 18.10.7.1 or 18.10.7.2 shall be reinforced in accordance with (a) or (b):

(a) Two intersecting groups of diagonally placed bars symmetrical about the midspan

(b) Longitudinal and transverse reinforcement satisfying (i) through (iii):

(i) 18.6.3 and 18.6.4, with the wall boundary interpreted as being a column.

(ii) Transverse reinforcement proportioned to satisfy the shear strength requirements of 18.6.5.

(iii) Spacing of transverse reinforcement not exceeding the limits in Table 18.10.7.4.

18.10.7.4 Coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan shall satisfy (a), (b), and either (c) or (d), and the requirements of 9.9 need not be satisfied:

(a) V_n shall be calculated by

$$V_n = 2A_{vd}f_y\sin\alpha \leq 10\sqrt{f_c'}A_{cw} \qquad (18.10.7.4)$$

where α is the angle between the diagonal bars and the longitudinal axis of the coupling beam.

(b) Each group of diagonal bars shall consist of a minimum of four bars provided in two or more layers.

(c) Each group of diagonal bars shall be enclosed by rectilinear transverse reinforcement having out-to-out dimensions of at least $b_w/2$ in the direction parallel to b_w and $b_w/5$ along the other sides, where b_w is the web width of the coupling beam. The transverse reinforcement shall be in accordance with 18.7.5.2(a) through (c) and shall provide lateral support to the diagonal reinforcement in accordance with 25.7.2.2 and 25.7.2.3. Reinforcement shall be arranged such that spacing of diagonal bars laterally supported by the corner of a crosstie or a hoop leg shall not exceed 14 in. around the perimeter of each group of diagonal bars, with A_{sh} not less than the greater of (i) and (ii):

(i) $0.09sb_c\dfrac{f_c'}{f_{yt}}$

(ii) $0.3sb_c\left(\dfrac{A_g}{A_{ch}} - 1\right)\dfrac{f_c'}{f_{yt}}$

In calculating A_g for each group of diagonal bars, the concrete cover in 20.5.1 shall be assumed on all four sides of each group of diagonal bars. The transverse reinforcement shall have spacing measured parallel to the diagonal bars satisfying 18.7.5.3(d) and not exceeding the limits in Table 18.10.7.4, and shall have spacing of crossties or legs of hoops measured perpendicular to the diagonal bars not exceeding 14 in. The transverse reinforcement shall continue through the intersection of the diagonal bars. At the intersec-

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tion, it is permitted to modify the arrangement of the transverse reinforcement provided the spacing and volume ratio requirements are satisfied. Additional longitudinal and transverse reinforcement shall be distributed around the beam perimeter with total area in each direction of at least $0.002b_ws$ and spacing not exceeding 12 in.

(d) Transverse reinforcement shall be provided for the entire beam cross section in accordance with 18.7.5.2(a) through (e) with A_{sh} not less than the greater of (i) and (ii):

(i) $0.09sb_c\dfrac{f_c'}{f_{yt}}$

(ii) $0.3sb_c\left(\dfrac{A_g}{A_{ch}}-1\right)\dfrac{f_c'}{f_{yt}}$

Longitudinal spacing of transverse reinforcement shall not exceed the limits in Table 18.10.7.4. Spacing of crossties or legs of hoops both vertically and horizontally in the plane of the beam cross section shall not exceed 8 in. Each crosstie and each hoop leg shall engage a longitudinal bar of equal or greater diameter. It shall be permitted to configure hoops as specified in 18.6.4.3.

Table 18.10.7.4—Maximum spacing of transverse reinforcement in coupling beams

Grade of diagonal or primary flexural reinforcement	Maximum spacing of transverse reinforcement[1]	
60	Lesser of:	$6d_b$
		6 in.
80	Lesser of:	$5d_b$
		6 in.
100	Lesser of:	$4d_b$
		6 in.

[1]d_b is the diameter of the smallest diagonal bar or primary flexural reinforcing bar.

18.10.7.5 Design shear force V_e of coupling beams shall be permitted to be redistributed to coupling beams at adjacent floor levels provided (a) through (d) are satisfied:

(a) Coupling beams sharing redistributed forces shall be vertically aligned within a special structural wall.

(b) Coupling beams sharing redistributed forces shall have $\ell_n/h \geq 2$.

(c) The maximum redistribution of V_e from any beam shall not exceed 20% of the value determined from analysis.

(d) The sum of ϕV_n of coupling beams sharing redistributed demands shall be equal to or greater than the sum of V_e in those beams.

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R18.10.7.5 Redistribution of coupling beam shear demands determined from linear analysis is permitted because coupling beams designed in accordance with 18.10.7 have significant plastic rotational capacity and are expected to be a primary yielding mechanism. For coupling beams designed in accordance with 18.6 as allowed by 18.10.7.1 and 18.10.7.3, the redistribution of earthquake beam moments in proportion to the redistributed shears is necessary to maintain internal equilibrium.

Redistributing demands in vertically aligned coupling beams generally creates more economical and constructible design details. Although precise vertical alignment of coupling beams sharing coupling demands is not necessary, coupling beams with similar stiffnesses more predictably and evenly share redistributed demands. The presence of one or more coupling beams or horizontal wall segments significantly deeper than coupling beams aligned above or below inhibits redistribution and should be avoided.

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18.10.7.6 Penetrations in a coupling beam designed according to 18.10.7.4 shall satisfy (a) through (d):

(a) The number of penetrations shall not exceed two.

(b) Each penetration shall comprise a cast-in horizontal cylindrical void oriented transverse to the plane of the coupling beam, with diameter not exceeding the larger of $h/6$ and 6 in.

(c) Penetrations shall be located at least 2 in. clear from diagonally placed bars, at least $h/4$ clear from the ends of the coupling beam, at least 4 in. clear from the top and bottom of the coupling beam, and at least a dimension equal to the larger penetration diameter from an adjacent penetration.

(d) Penetrations shall not cause transverse reinforcement to violate requirements of 18.10.7.4(d).

COMMENTARY

Redistribution of coupling beam demands relies upon beam end rotations beyond the elastic range. Vertically aligned coupling beams with close proximity are more likely to experience similar end rotations, and thus more reliably share redistributed forces. Consideration should be given to redistribute demands to coupling beams within reasonable proximity.

R18.10.7.6 Penetrations through diagonally reinforced coupling beams are at times unavoidable due to the routing of plumbing, electrical, and other building services. Penetrations should be avoided where possible or otherwise minimized. Penetrations with excessive size or located in critical regions of a beam have the potential to compromise the ductility capacity of the beam.

Test results (Abdullah et al. 2023) have shown that the shear resistance and rotational capacity of diagonally reinforced coupling beams are not appreciably affected by penetrations meeting the limitations of 18.10.7.6. Tests cited were conducted with circular openings; rectangular openings are to be avoided because corners can cause stress concentrations under dynamic loads that have not been evaluated experimentally. Figure R18.10.7.6 illustrates these limitations for a coupling beam with a typical ℓ_n/h aspect ratio.

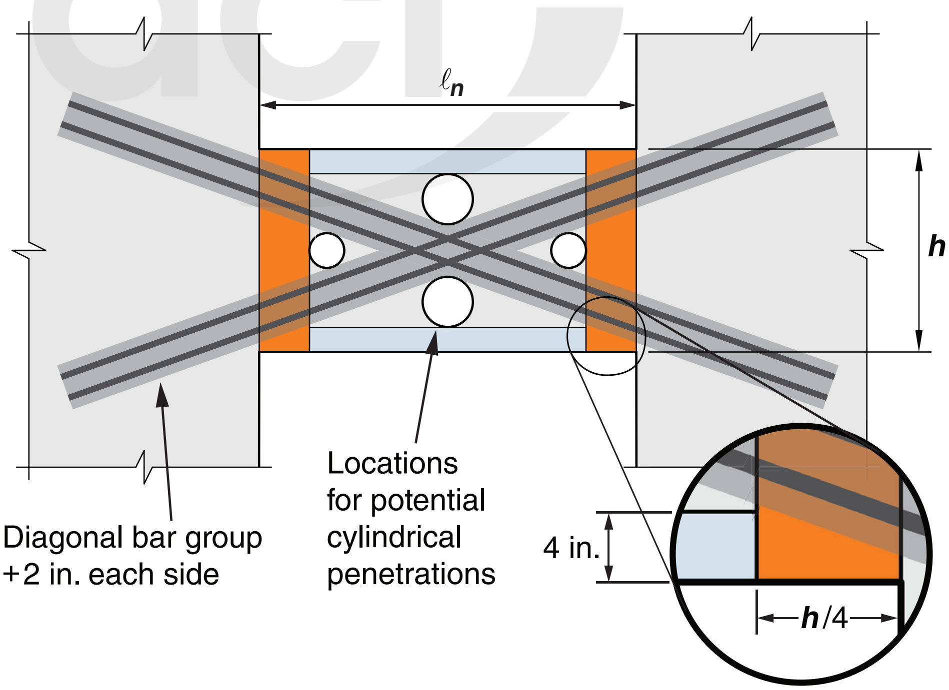

Fig. R18.10.7.6—Coupling beam elevation showing prohibited penetration regions shaded. Note that four potential locations for penetrations are shown although only two are permitted in any one beam.

CODE

18.10.8 *Wall piers*

COMMENTARY

R18.10.8 *Wall piers*

Door and window placements in structural walls sometimes lead to narrow vertical wall segments that are considered to be wall piers. The dimensions defining wall piers are given in Chapter 2. Shear failures of wall piers have been observed in previous earthquakes. The intent of this section is to provide sufficient shear strength to wall piers such that inelastic response, if it occurs, will be primarily in flexure. The provisions apply to wall piers designated as part of the seismic-force-resisting system. Provisions for wall piers not designated as part of the seismic-force-resisting system are given in 18.14. The effect of all vertical wall segments on the response of the structural system, whether designated as part of the seismic-force-resisting system or not, should be considered as required by 18.2.2. Wall piers having $(\ell_w/b_w) \leq 2.5$ behave essentially as columns. Provision 18.10.8.1 requires that such members satisfy reinforcement and shear strength requirements of 18.7.4 through 18.7.6. Alternative provisions are provided for wall piers having $(\ell_w/b_w) > 2.5$.

The design shear force determined according to 18.7.6.1 may be unrealistically large in some cases. As an alternative, 18.10.8.1(a) permits the design shear force to be determined using factored load combinations in which the earthquake effect has been amplified to account for system overstrength. Documents such as the NEHRP provisions (FEMA P-749), ASCE/SEI 7, and the 2021 IBC represent the amplified earthquake effect using the factor Ω_o.

Section 18.10.8.2 addresses wall piers at the edge of a wall. Under in-plane shear, inclined cracks can propagate into segments of the wall directly above and below the wall pier. Unless there is sufficient reinforcement in the adjacent wall segments, shear failure within the adjacent wall segments can occur. The length of embedment of the provided reinforcement into the adjacent wall segments should be determined considering both development length requirements and shear strength of the wall segments (refer to Fig. R18.10.8).

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COMMENTARY

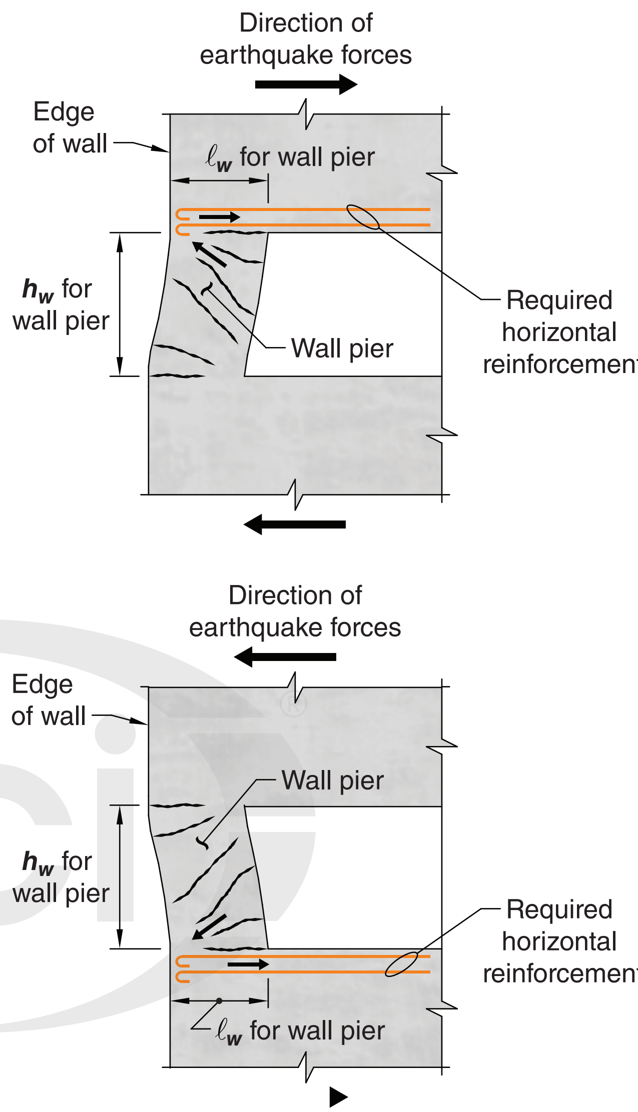

Fig. R18.10.8—Required horizontal reinforcement in wall segments above and below wall piers at the edge of a wall.

CODE

18.10.8.1 Wall piers shall satisfy the special moment frame requirements for columns of 18.7.4, 18.7.5, and 18.7.6, with joint faces taken as the top and bottom of the clear height of the wall pier. Alternatively, wall piers with $(\ell_w/b_w) > 2.5$ shall satisfy (a) through (f):

(a) Design shear force shall be calculated in accordance with 18.7.6.1 with joint faces taken as the top and bottom of the clear height of the wall pier. If the general building code includes provisions to account for overstrength of the seismic-force-resisting system, the design shear force need not exceed Ω_o times the factored shear calculated by analysis of the structure for earthquake load effects.

(b) V_n and distributed shear reinforcement shall satisfy 18.10.4.

(c) Transverse reinforcement shall be hoops except it shall be permitted to use single-leg horizontal reinforcement parallel to ℓ_w where only one curtain of distributed shear reinforcement is provided. Single-leg horizontal reinforcement shall have 180-degree bends at each end that engage wall pier boundary longitudinal reinforcement.

(d) Vertical spacing of transverse reinforcement shall not exceed 6 in.

(e) Transverse reinforcement shall extend at least 12 in. above and below the clear height of the wall pier.

(f) Special boundary elements shall be provided if required by 18.10.6.3.

18.10.8.2 For wall piers at the edge of a wall, horizontal reinforcement shall be provided in adjacent wall segments above and below the wall pier and be designed to transfer the design shear force from the wall pier into the adjacent wall segments.

18.10.9 *Ductile coupled walls*

18.10.9.1 Ductile coupled walls shall satisfy the requirements of this section.

18.10.9.2 Individual walls shall satisfy $h_{wcs}/\ell_w \geq 2$ and the applicable provisions of 18.10 for special structural walls.

18.10.9.3 Coupling beams shall satisfy 18.10.7 and (a) through (c) in the direction considered.

(a) Coupling beams shall have $\ell_n/h \geq 2$ at all levels of the building.

(b) All coupling beams at a floor level shall have $\ell_n/h \leq 5$ in at least 90% of the levels of the building.

COMMENTARY

R18.10.9 *Ductile coupled walls*

The aspect ratio limits and development length requirements for ductile coupled walls are intended to induce an energy dissipation mechanism associated with inelastic deformation reversal of coupling beams. Wall stiffness and strength at each end of coupling beams should be sufficient to develop this intended behavior.

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(c) The requirements of 18.10.2.5 shall be satisfied at both ends of all coupling beams.

18.10.10 *Construction joints*

18.10.10.1 Construction joints in structural walls shall be specified according to 26.5.6, and contact surfaces shall be roughened consistent with condition (b) of Table 22.9.4.2.

18.10.11 *Discontinuous walls*

18.10.11.1 Columns supporting discontinuous structural walls shall be reinforced in accordance with 18.7.5.6.

18.11—Special structural walls constructed using precast concrete

18.11.1 *Scope*

18.11.1.1 This section shall apply to special structural walls constructed using precast concrete forming part of the seismic-force-resisting system.

18.11.2 *General*

18.11.2.1 Special structural walls constructed using precast concrete shall satisfy 18.10 and 18.5.2, except 18.10.2.4 shall not apply for precast walls where deformation demands are concentrated at the panel joints.

18.11.2.2 Special structural walls constructed using precast concrete and unbonded post-tensioning tendons and not satisfying the requirements of 18.11.2.1 are permitted provided they satisfy the requirements of ACI CODE-550.6.

18.12—Diaphragms and trusses

18.12.1 *Scope*

COMMENTARY

R18.11—Special structural walls constructed using precast concrete

R18.11.2 *General*

R18.11.2.2 Experimental and analytical studies (Priestley et al. 1999; Perez et al. 2003; Restrepo 2002) have demonstrated that some types of precast structural walls post-tensioned with unbonded tendons, and not satisfying the prescriptive requirements of Chapter 18, provide satisfactory seismic performance characteristics. ACI CODE-550.6 defines a protocol for establishing a design procedure, validated by analysis and laboratory tests, for such walls, with or without coupling beams.

ACI CODE-550.7 defines design requirements for one type of special structural wall constructed using precast concrete and unbonded post-tensioning tendons, and validated for use in accordance with 18.11.2.2.

R18.12—Diaphragms and trusses

R18.12.1 *Scope*

Diaphragms as used in building construction are structural elements (such as a floor or roof) that provide some or all of the following functions:

(a) Support for building elements (such as walls, partitions, and cladding) resisting horizontal forces but not acting as part of the seismic-force-resisting system

(b) Transfer of lateral forces from the point of application to the vertical elements of the seismic-force-resisting system

CODE

COMMENTARY

(c) Connection of various components of the vertical seismic-force-resisting system with appropriate strength, stiffness, and ductility so the building responds as intended in the design (Wyllie 1987).

18.12.1.1 This section shall apply to diaphragms and collectors forming part of the seismic-force-resisting system in structures assigned to SDC D, E, or F and to SDC C if 18.12.1.2 applies.

18.12.1.2 Section 18.12.11 shall apply to diaphragms constructed using precast concrete members and forming part of the seismic-force-resisting system for structures assigned to SDC C, D, E, or F.

18.12.1.3 Section 18.12.12 shall apply to structural trusses forming part of the seismic-force-resisting system in structures assigned to SDC D, E, or F.

18.12.2 *Design forces*

18.12.2.1 The earthquake design forces for diaphragms shall be obtained from the general building code using the applicable provisions and load combinations.

R18.12.2 *Design forces*

R18.12.2.1 In the general building code, earthquake design forces for floor and roof diaphragms typically are not calculated directly during the lateral-force analysis that provides story forces and story shears. Instead, diaphragm design forces at each level are calculated by a formula that amplifies the story forces recognizing dynamic effects and includes minimum and maximum limits. These forces are used with the governing load combinations to design diaphragms for shear and moment.

For collector elements, the general building code in the United States specifies load combinations that amplify earthquake forces by a factor Ω_o. The forces amplified by Ω_o are also used for the local diaphragm shear forces resulting from the transfer of collector forces, and for local diaphragm flexural moments resulting from any eccentricity of collector forces. The specific requirements for earthquake design forces for diaphragms and collectors depend on which edition of the general building code is used. The requirements may also vary according to the SDC.

For most concrete buildings subjected to inelastic earthquake demands, it is desirable to limit inelastic behavior of floor and roof diaphragms under the imposed earthquake forces and deformations. It is preferable for inelastic behavior to occur only in the intended locations of the vertical seismic-force-resisting system that are detailed for ductile response, such as in beam plastic hinges of special moment frames, or in flexural plastic hinges at the base of structural walls or in coupling beams. For buildings without long diaphragm spans between lateral-force-resisting elements, elastic diaphragm behavior is typically not difficult to achieve. For buildings where diaphragms could reach their flexural or shear strength before yielding occurs in the vertical seismic-force-resisting system, the licensed

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design professional should consider providing increased diaphragm strength.

For reinforced concrete diaphragms, ASCE/SEI 7 Sections 12.10.1 and 12.10.2 provide requirements to determine design forces for reinforced concrete diaphragms. For precast concrete diaphragms in buildings assigned to SDC C, D, E, or F, the provisions of ASCE/SEI 7 Section 12.10.3 apply.

18.12.3 *Seismic load path*

18.12.3.1 All diaphragms and their connections shall be designed and detailed to provide for transfer of forces to collector elements and to the vertical elements of the seismic-force-resisting system.

18.12.3.2 Elements of a structural diaphragm system that are subjected primarily to axial forces and used to transfer diaphragm shear or flexural forces around openings or other discontinuities shall satisfy the requirements for collectors in 18.12.7.6 and 18.12.7.7.

R18.12.3 *Seismic load path*

R18.12.3.2 This provision applies to strut-like elements that occur around openings, diaphragm edges, or other discontinuities in diaphragms. Figure R18.12.3.2 shows an example. Such elements can be subjected to earthquake axial forces in combination with bending and shear from earthquake or gravity loads.

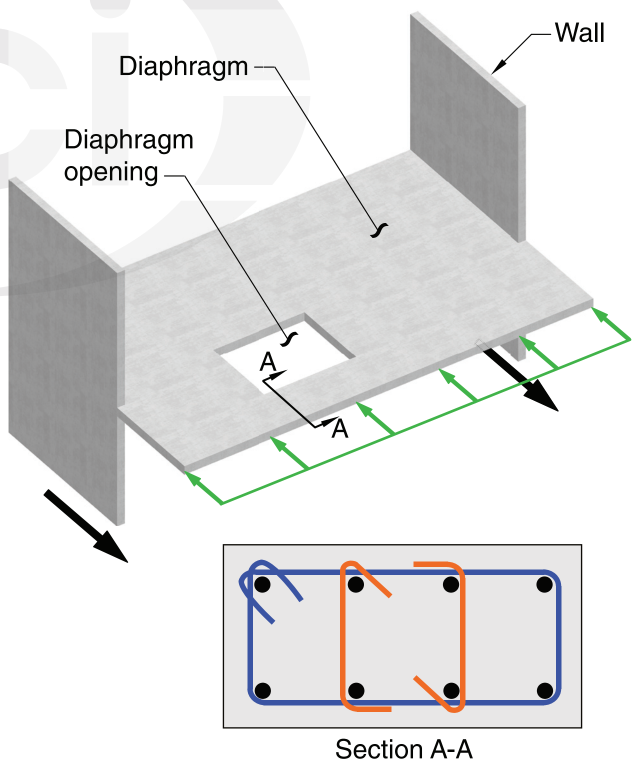

Fig. R18.12.3.2—Example of diaphragm subject to the requirements of 18.12.3.2 and showing an element having confinement as required by 18.12.7.6.

CODE

18.12.4 *Cast-in-place composite topping slab diaphragms*

18.12.4.1 A cast-in-place composite topping slab on a precast floor or roof shall be permitted as a structural diaphragm, provided the cast-in-place topping slab is reinforced and the surface of the previously hardened concrete on which the topping slab is placed is clean, free of laitance, and intentionally roughened.

18.12.5 *Cast-in-place noncomposite topping slab diaphragms*

18.12.5.1 A cast-in-place noncomposite topping on a precast floor or roof shall be permitted as a structural diaphragm, provided the cast-in-place topping slab acting alone is designed and detailed to resist the design earthquake forces.

18.12.6 *Minimum thickness of diaphragms*

18.12.6.1 Concrete slabs and composite topping slabs serving as diaphragms used to transmit earthquake forces shall be at least 2 in. thick. Topping slabs placed over precast floor or roof elements, acting as diaphragms and not relying on composite action with the precast elements to resist the design earthquake forces, shall be at least 2-1/2 in. thick.

18.12.7 *Reinforcement*

18.12.7.1 Minimum reinforcement for diaphragms shall be in conformance with 12.6. Except for post-tensioned slabs, reinforcement spacing each way in floor or roof systems shall not exceed 18 in. Where welded wire reinforcement is used as the distributed reinforcement to resist shear in topping slabs placed over precast floor and roof elements, the wires parallel to the joints between the precast elements shall be spaced not less than 10 in. on center. Reinforcement provided for shear strength shall be continuous and shall be distributed uniformly across the shear plane.

18.12.7.2 Bonded tendons used as reinforcement to resist collector forces, diaphragm shear, or flexural tension shall be designed such that the stress due to design earthquake forces does not exceed 60,000 psi. Precompression from unbonded tendons shall be permitted to resist diaphragm design forces if a seismic load path is provided.

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R18.12.4 *Cast-in-place composite topping slab diaphragms*

R18.12.4.1 A bonded topping slab is required so that the floor or roof system can provide restraint against slab buckling. Reinforcement is required to ensure the continuity of the shear transfer across precast joints. The connection requirements are introduced to promote a complete system with necessary shear transfers.

R18.12.5 *Cast-in-place noncomposite topping slab diaphragms*

R18.12.5.1 Composite action between the topping slab and the precast floor elements is not required, provided that the topping slab is designed to resist the design earthquake forces.

R18.12.6 *Minimum thickness of diaphragms*

R18.12.6.1 The minimum thickness of concrete diaphragms reflects current practice in joist and waffle systems and composite topping slabs on precast floor and roof systems. Thicker slabs are required if the topping slab is not designed to act compositely with the precast system to resist the design earthquake forces.

R18.12.7 *Reinforcement*

R18.12.7.1 The maximum spacing for reinforcement is intended to control the width of inclined cracks. Minimum average prestress requirements (refer to 24.4.4.1) are considered to be adequate to limit the crack widths in post-tensioned floor systems; therefore, the maximum spacing requirements do not apply to these systems.

The minimum spacing requirement for welded wire reinforcement in topping slabs on precast floor systems is to avoid fracture of the distributed reinforcement during an earthquake. Cracks in the topping slab open immediately above the boundary between the flanges of adjacent precast members, and the wires crossing those cracks are restrained by the transverse wires (Wood et al. 2000). Therefore, all the deformation associated with cracking should be accommodated in a distance not greater than the spacing of the transverse wires. A minimum spacing of 10 in. for the transverse wires is required to reduce the likelihood of fracture of the wires crossing the critical cracks during a design earthquake. The minimum spacing requirements do not apply to diaphragms reinforced with individual bars, because strains are distributed over a longer length.

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18.12.7.3 All reinforcement used to resist collector forces, diaphragm shear, or flexural tension shall be developed or spliced for f_y in tension.

18.12.7.4 Class G or Class S mechanical splices are required where mechanical splices are used in the region of the connection between the diaphragm and the vertical elements of the seismic-force-resisting system.

18.12.7.5 Longitudinal reinforcement for collectors shall be proportioned such that the average tensile stress over length (a) or (b) does not exceed ϕf_y where the value of f_y is limited to 60,000 psi.

(a) Length between the end of a collector and location at which transfer of load to a vertical element begins

(b) Length between two vertical elements

18.12.7.6 Collector elements with compressive stresses exceeding $0.2f_c'$ at any section shall have transverse reinforcement satisfying 18.7.5.2(a) through (e) and 18.7.5.3, except the spacing limit of 18.7.5.3(a) shall be one-third of the least dimension of the collector. The amount of transverse reinforcement shall be in accordance with Table 18.12.7.6. The specified transverse reinforcement is permitted to be discontinued at a section where the calculated compressive stress is less than $0.15f_c'$.

If design forces have been amplified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $0.2f_c'$ shall be increased to $0.5f_c'$, and the limit of $0.15f_c'$ shall be increased to $0.4f_c'$.

Table 18.12.7.6—Transverse reinforcement for collector elements

Transverse reinforcement	Applicable expressions		
A_{sh}/sb_c for rectilinear hoop	$0.9\dfrac{f_c'}{f_{yt}}$		(a)
ρ_s for spiral or circular hoop	Greater of:	$0.45\left(\dfrac{A_g}{A_{ch}}-1\right)\dfrac{f_c'}{f_{yt}}$	(b)
		$0.12\dfrac{f_c'}{f_{yt}}$	(c)

18.12.7.7 Longitudinal reinforcement detailing for collector elements at splices and anchorage zones shall satisfy (a) or (b):

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R18.12.7.3 Bar development and lap splices are designed according to requirements of Chapter 25 for reinforcement in tension. Reductions in development or splice length for calculated stresses less than f_y are not permitted, as indicated in 25.4.10.2.

R18.12.7.4 Although the Ω_o factor is to be applied to the design forces transferred by the connection between the diaphragm and the vertical elements of the seismic-force-resisting system, significant yielding of reinforcement at the connection may occur due to inelastic displacements, including but not limited to displacements caused by rocking structural walls and displacements arising from flexural elongation of wall boundaries occurring over several stories. The application of the Ω_o factor does not limit this kind of yielding, and a Class S splice should be considered.

R18.12.7.5 Table 20.2.2.4(a) permits the maximum design yield strength to be 80,000 psi for portions of a collector, for example, at and near critical sections. The average stress in the collector is limited to control diaphragm cracking over the length of the collector. The calculation of average stress along the length is not necessary if the collector is designed for f_y of 60,000 psi even if Grade 80 reinforcement is specified.

R18.12.7.6 In documents such as the NEHRP Provisions (FEMA P-750), ASCE/SEI 7, the 2024 IBC, and the Uniform Building Code (ICBO 1997), collector elements of diaphragms are designed for forces amplified by a factor Ω_o to account for the overstrength in the vertical elements of the seismic-force-resisting systems. The amplification factor Ω_o ranges between 2 and 3 for most concrete structures, depending on the document selected and on the type of seismic-force-resisting system. In some documents, the factor can be calculated based on the maximum forces that can be developed by the elements of the vertical seismic-force-resisting system.

Compressive stress calculated for the factored forces on a linearly elastic model based on gross section of the structural diaphragm is used as an index value to determine whether confining reinforcement is required. A calculated compressive stress of $0.2f_c'$, or $0.5f_c'$ for forces amplified by Ω_o, is assumed to indicate that integrity of the entire structure depends on the ability of that member to resist substantial compressive force under severe cyclic loading. Transverse reinforcement is required at such locations to provide confinement for the concrete and the reinforcement.

R18.12.7.7 This section is intended to reduce the possibility of bar buckling and provide adequate bar development conditions in the vicinity of splices and anchorage zones.

CODE

(a) Center-to-center spacing of at least three longitudinal bar diameters, but not less than 1-1/2 in., and concrete clear cover of at least two and one-half longitudinal bar diameters, but not less than 2 in.

(b) Area of transverse reinforcement, providing A_v at least the greater of $0.75\sqrt{f_c'}(b_w s/f_{yt})$ and $50b_w s/f_{yt}$, except as required in 18.12.7.6

18.12.8 *Combined moment and axial strength*

18.12.8.1 Diaphragms and portions of diaphragms shall be designed for flexure in accordance with Chapter 12. The effects of openings shall be considered.

18.12.9 *Shear strength*

COMMENTARY

R18.12.8 *Combined moment and axial strength*

R18.12.8.1 Flexural strength of diaphragms is calculated using the same assumptions as for walls, columns, or beams. The design of diaphragms for moment and other actions uses the applicable load combinations of 5.3.1 to consider earthquake forces acting concurrently with gravity or other loads.

The influence of slab openings on moment and shear strength is to be considered, including evaluating the potential critical sections created by the openings. The strut-and-tie method is potentially useful for designing diaphragms with openings.

In the 1999 through 2005 editions of the Code, it was assumed that design moments for diaphragms were resisted entirely by chord forces acting at opposite edges of the diaphragm consistent with a beam idealization of the diaphragm. While this approach is still used, it is also acceptable to resist design moments using distributed reinforcement or precompression with proper consideration of applicable load combinations and analysis including compatibility of stresses and strains.

R18.12.9 *Shear strength*

The shear strength requirements for diaphragms are similar to those for slender structural walls and are based on the shear provisions for beams. The term A_{cv} refers to the gross area of the diaphragm, but may not exceed the thickness times the width of the diaphragm. This corresponds to the gross area of the effective deep beam that forms the diaphragm. Distributed slab reinforcement ρ_t used to calculate shear strength of a diaphragm in Eq. (18.12.9.1) is positioned perpendicular to the diaphragm flexural reinforcement. Provision 18.12.9.2 limits the maximum shear strength of the diaphragm.

In addition to satisfying 18.12.9.1 and 18.12.9.2, cast-in-place topping slab diaphragms must also satisfy 18.12.9.3 and 18.12.9.4. Cast-in-place topping slabs on a precast floor or roof system tend to have shrinkage cracks that are aligned with the joints between adjacent precast members. Therefore, the additional shear strength requirements for topping slab diaphragms in 18.12.9.3 are based on a shear friction model (Wood et al. 2000), and the assumed crack plane corresponds to joints in the precast system along the direction of the applied shear, as shown in Fig. R22.9.4.3a. The coefficient of friction, μ, in the shear friction model is taken

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equal to 1.0 for normalweight concrete due to the presence of these shrinkage cracks.

Both distributed and boundary reinforcement in the topping slab may be considered as shear friction reinforcement A_{vf}. Boundary reinforcement within the diaphragm was called chord reinforcement in ACI 318 before 2008. Although the boundary reinforcement also resists forces due to moment and axial force in the diaphragm, the reduction in the shear friction resistance in the tension zone is offset by the increase in shear friction resistance in the compression zone. Therefore, the area of boundary reinforcement used to resist shear friction need not be added to the area of boundary reinforcement used to resist moment and axial force. The distributed topping slab reinforcement must contribute at least one-half of the nominal shear strength. It is assumed that connections between the precast elements do not contribute to the shear strength of the topping slab diaphragm.

Provision 18.12.9.4 limits the maximum shear that may be transmitted by shear friction within a topping slab diaphragm.

18.12.9.1 V_n of diaphragms shall not exceed:

$$V_n = A_{cv}(2\lambda\sqrt{f_c'} + \rho_t f_y) \qquad (18.12.9.1)$$

For cast-in-place topping slab diaphragms on precast floor or roof members, A_{cv} shall be calculated using only the thickness of topping slab for noncomposite topping slab diaphragms and the combined thickness of cast-in-place and precast elements for composite topping slab diaphragms. For composite topping slab diaphragms, the value of f_c' used to calculate V_n shall not exceed the lesser of f_c' for the precast members and f_c' for the topping slab.

18.12.9.2 V_n of diaphragms shall not exceed $8\sqrt{f_c'}A_{cv}$.

18.12.9.3 Above joints between precast elements in noncomposite and composite cast-in-place topping slab diaphragms, V_n shall not exceed:

$$V_n = A_{vf} f_y \mu \qquad (18.12.9.3)$$

where A_{vf} is the total area of shear friction reinforcement within the topping slab, including both distributed and boundary reinforcement, that is oriented perpendicular to joints in the precast system and coefficient of friction, μ, is 1.0λ, where λ is given in 19.2.4. At least one-half of A_{vf} shall be uniformly distributed along the length of the potential shear plane. The area of distributed reinforcement in the topping slab shall satisfy 24.4.3.2 in each direction.

18.12.9.4 Above joints between precast elements in noncomposite and composite cast-in-place topping slab diaphragms, V_n shall not exceed the limits in 22.9.4.4, where A_c is calculated using only the thickness of the topping slab.

CODE

18.12.10 *Construction joints*

18.12.10.1 Construction joints in diaphragms shall be specified according to 26.5.6, and contact surfaces shall be roughened consistent with condition (b) of Table 22.9.4.2.

18.12.11 *Precast concrete diaphragms*

18.12.11.1 Diaphragms and collectors constructed using precast concrete members with composite topping slab and not satisfying 18.12.4, and untopped precast concrete diaphragms, are permitted provided they satisfy the requirements of ACI CODE-550.5. Cast-in-place noncomposite topping slab diaphragms shall satisfy 18.12.5 and 18.12.6.

18.12.11.2 Connections and reinforcement at joints used in the construction of precast concrete diaphragms satisfying 18.12.11.1 shall have been tested in accordance with ACI CODE-550.4.

18.12.11.3 Extrapolation of data on connections and reinforcement at joints to project details that result in larger construction tolerances than those used to qualify connections in accordance with ACI CODE-550.4 shall not be permitted.

18.12.12 *Structural trusses*

18.12.12.1 Structural truss elements with compressive stresses exceeding $0.2f_c'$ at any section shall have transverse reinforcement, in accordance with 18.7.5.2, 18.7.5.3, 18.7.5.7, and Table 18.12.12.1, over the length of the element.

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R18.12.11 *Precast concrete diaphragms*

R18.12.11.1 ACI CODE-550.5 provides requirements for the design of precast concrete diaphragms with connections whose performance has been validated by ACI CODE-550.4 testing. ACI CODE-550.5 permits a maximum tolerance for positioning and completion of connections of 1/2 in., which can be difficult to achieve with normal construction practices. Section 26.13.1.3 requires continuous inspection of precast concrete diaphragm connections to verify that construction is performed properly and tolerances not greater than 1/2 in. for all connections are achieved. Results from ACI CODE-550.4 testing are not to be extrapolated to allow greater tolerances.

Topped precast concrete floors designed in accordance with Chapter 18 need careful consideration of support conditions to verify precast concrete members have sufficient seating for anticipated displacements and ability to accommodate relative rotations between beam supports and the member (Henry et al. 2017).

R18.12.12 *Structural trusses*

R18.12.12.1 The expressions for transverse reinforcement A_{sh} are based on ensuring compression capacity of an equivalent column section is maintained after spalling of cover concrete.

CODE

Table 18.12.12.1—Transverse reinforcement for structural trusses

Transverse reinforcement	Applicable expressions		
A_{sh}/sb_c for rectilinear hoop	Greater of:	$0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$	(a)
		$0.09\frac{f_c'}{f_{yt}}$	(b)
ρ_s for spiral or circular hoop	Greater of:	$0.45\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$	(c)
		$0.12\frac{f_c'}{f_{yt}}$	(d)

18.12.12.2 All continuous reinforcement in structural truss elements shall be developed or spliced for f_y in tension.

18.13—Foundations

18.13.1 *Scope*

18.13.1.1 This section shall apply to foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground.

18.13.1.2 The provisions in this section for piles, drilled piers, caissons, and slabs-on-ground shall supplement other applicable Code design and construction criteria, including 1.4.7 and 1.4.8.

18.13.2 *Footings, foundation mats, and pile caps*

18.13.2.1 The provisions of this section shall apply to structures assigned to SDC D, E, or F.

18.13.2.2 Longitudinal reinforcement of columns and structural walls resisting forces induced by earthquake effects shall extend into the footing, mat, or pile cap, and shall develop f_y in tension at the interface.

18.13.2.3 Columns designed assuming fixed-end conditions at the foundation shall comply with 18.13.2.2 and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90-degree hooks near the bottom of the

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R18.13—Foundations

R18.13.1 *Scope*

Requirements for foundations supporting buildings assigned to SDC C, D, E, or F represent a consensus of a minimum level of good practice in designing and detailing concrete foundations. However, because repairs to foundations can be extremely difficult and expensive, it may be desirable that the elements of the foundation remain essentially elastic during strong ground motions. Methods to achieve this goal include designing the foundation to include an overstrength factor or an increased seismic demand level when compared to the superstructure, or comparing strengths to demands predicted by nonlinear response history analyses with appropriate consideration of uncertainty in demands (Klemencic et al. 2012).

R18.13.2 *Footings, foundation mats, and pile caps*

R18.13.2.3 Tests (Nilsson and Losberg 1976) have demonstrated that flexural members terminating in a footing, slab, or beam (a T-joint or L-joint) should have their hooks turned inward toward the axis of the member for the joint to

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foundation with the free end of the bars oriented toward the center of the column.

18.13.2.4 Columns or boundary elements of special structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with 18.7.5.2 through 18.7.5.4 provided below the top of the footing. This reinforcement shall extend into the footing, mat, or pile cap a length equal to the development length, calculated for f_y in tension, of the column or boundary element longitudinal reinforcement.

18.13.2.5 Where earthquake effects create uplift forces in boundary elements of special structural walls or columns, flexural reinforcement shall be provided in the top of the footing, mat, or pile cap to resist actions resulting from the factored load combinations, and shall be at least that required by 7.6.1 or 9.6.1.

18.13.2.6 Structural plain concrete in footings and basement walls shall be in accordance with 14.1.3.

18.13.2.7 Pile caps incorporating batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns. The slenderness effects of batter piles shall be considered for the portion of the piles in soil that is not capable of providing lateral support, or in air or water.

18.13.3 *Grade beams and slabs-on-ground*

18.13.3.1 For structures assigned to SDC D, E, or F, grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the seismic-force-resisting system shall be in accordance with 18.6.

18.13.3.2 For structures assigned to SDC C, D, E, or F, slabs-on-ground that resist in-plane earthquake forces from walls or columns that are part of the seismic-force-resisting system shall be designed as diaphragms in accordance with 18.12. The construction documents shall clearly indicate that the slab-on-ground is a structural diaphragm and part of the seismic-force-resisting system.

18.13.4 *Foundation seismic ties*

18.13.4.1 For structures assigned to SDC C, D, E, or F, individual pile caps, piers, or caissons shall be interconnected by foundation seismic ties in orthogonal directions,

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be able to resist the flexure in the member forming the stem of the T or L.

R18.13.2.4 Columns or boundary members supported close to the edge of the foundation, as often occurs near property lines, should be detailed to prevent an edge failure of the footing, pile cap, or mat.

R18.13.2.5 The purpose of this section is to emphasize that top reinforcement in footings, mats, and pile caps may be required, in addition to other required reinforcement.

R18.13.2.6 Foundation and basement walls should be reinforced in buildings assigned to SDC D, E, or F.

R18.13.2.7 Batter piles typically attract higher lateral forces during earthquakes than vertical piles. Extensive structural damage has been observed at the junction of batter piles and building foundations. The pile cap and surrounding structure should be designed for the potentially large forces that can be developed in batter piles.

R18.13.3 *Grade beams and slabs-on-ground*

For earthquake conditions, slabs-on-ground (soil-supported slabs) are often part of the lateral-force-resisting system and should be designed in accordance with this Code as well as other appropriate standards or guidelines (refer to 1.4.8).

R18.13.3.1 Grade beams resisting flexural stresses from column moments should have reinforcement details similar to the beams of the frame above the foundation.

R18.13.3.2 Slabs-on-ground often act as a diaphragm to tie the building together at the ground level and minimize the effects of out-of-phase ground motion that may occur over the footprint of the building. The construction documents should clearly state that these slabs-on-ground are structural members so as to prohibit saw cutting of the slab.

R18.13.4 *Foundation seismic ties*

R18.13.4.1 The foundation seismic ties should sufficiently interconnect foundations to act as a unit and be designed to minimize the relative movement of an individual column or tie relative to the foundation. This is essential where surface

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unless it can be demonstrated that equivalent restraint is provided by other means.

18.13.4.2 For structures assigned to SDC D, E, or F, individual spread footings founded on soil defined in ASCE/SEI 7 as Site Class E or F shall be interconnected by foundation seismic ties.

18.13.4.3 Where required, foundation seismic ties shall have a design strength in tension and compression at least equal to $0.1S_{DS}$ times the greater of the pile cap or column factored dead load plus factored live load unless it is demonstrated that equivalent restraint will be provided by (a), (b), (c), or (d):

(a) Reinforced concrete beams within the slab-on-ground
(b) Reinforced concrete slabs-on-ground
(c) Confinement by competent rock, hard cohesive soils, or very dense granular soils
(d) Other means approved by the building official

18.13.4.4 For structures assigned to SDC D, E, or F, grade beams designed to act as horizontal foundation seismic ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supported column or anchored within the pile cap or footing at all discontinuities and shall satisfy (a) and (b):

(a) The smallest cross-sectional dimension of the grade beam shall be at least equal to the clear spacing between connected columns divided by 20, but need not exceed 18 in.
(b) Closed tie transverse reinforcement shall be provided at a spacing not to exceed the lesser of 0.5 times the smallest orthogonal cross-sectional dimension and 12 in.

18.13.5 *Deep foundations*

18.13.5.1 This section shall apply to the following types of deep foundations

(a) Uncased cast-in-place concrete drilled or augered piles
(b) Metal cased concrete piles
(c) Concrete filled pipe piles
(d) Precast concrete piles

18.13.5.2 For structures assigned to SDC C, D, E, or F, piles, piers, or caissons resisting tension loads shall have continuous longitudinal reinforcement over their length to resist design tension forces.

18.13.5.3 For structures assigned to SDC C, D, E, or F, the minimum longitudinal and transverse reinforcement required by 18.13.5.7 through 18.13.5.10 shall be extended

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soils are soft enough to require deep foundations or where the site soils are susceptible to liquefaction.

R18.13.4.2 The ties between footings should have the same characteristics as the ties between pile caps in R18.13.4.1.

R18.13.4.3 The minimum foundation seismic tie design strength requirement based on a percentage of the factored dead plus live load provides a minimum connection between foundation elements. Other types of restraint can be used if substantiated as equivalent to the minimum tie design strength. The required design strength for the tie beam must be at least equal to $0.1S_{DS}$ times the larger force on either end of the tie beam, and that force is from the column or pile cap, whichever applies.

R18.13.5 *Deep foundations*

Adequate performance of piles and caissons for earthquake effects requires that these provisions be met in addition to other applicable standards or guidelines (refer to R1.4.7).

R18.13.5.3 Minimum reinforcement lengths for both longitudinal and transverse reinforcement are based on the assumption that soil is capable of providing lateral

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over the entire unsupported length for the portion of deep foundation member in air or water, or in soil that is not capable of providing adequate lateral restraint to prevent buckling throughout this length.

18.13.5.4 For structures assigned to SDC C, D, E, or F, hoops, spirals, and ties in deep foundation members shall be terminated with seismic hooks.

18.13.5.5 For structures assigned to SDC D, E, or F or located in Site Class E or F, concrete deep foundation members shall have transverse reinforcement in accordance with 18.7.5.2, 18.7.5.3, and Table 18.7.5.4 Item (e) within seven member diameters above and below the interfaces between strata that are hard or stiff and strata that are liquefiable or soft.

18.13.5.6 For structures assigned to SDC D, E, or F, in foundations supporting one- and two-story stud bearing wall construction, concrete piles, piers or caissons, and foundation ties are exempt from the transverse reinforcement requirements of 18.13.5.3 through 18.13.5.5.

18.13.5.7 *Uncased cast-in-place drilled or augered concrete piles or piers*

18.13.5.7.1 For structures assigned to SDC C, D, E, or F, reinforcement shall be provided in uncased cast-in-place drilled or augered concrete piles where required by analysis and in accordance with the requirements in Table 18.13.5.7.1.

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support. For portions of the pile above ground, typically in air or water, or where soil is not capable of providing this lateral restraint, the minimum reinforced lengths should be increased, and the member should be designed as a column.

R18.13.5.5 During earthquakes, piles can be subjected to high flexural and shear demands at points of discontinuity, such as at interfaces between stiff and soft soil strata. ASCE/SEI 7 defines limits for soil strata. Transverse reinforcement is required in these regions to provide ductile behavior. In determining the portions of a pile with increased transverse reinforcement, accommodations are often made to the length of the reinforced zone for transverse reinforcement to account for variations in the driven pile tip elevations and variations in the interface elevations between stiff and soft soil strata.

R18.13.5.7 *Uncased cast-in-place drilled or augered concrete piles or piers*

R18.13.5.7.1 Longitudinal and transverse reinforcement requirements prescribed by this section result in ductility consistent with the applicable Seismic Design Category (SDC) to withstand ground deformation that occurs during earthquakes.

Where piles are subjected to significant uplift forces, the longitudinal reinforcement length required by analysis may exceed the minimum reinforcement length requirements.

Transverse reinforcement is required at the top of the pile to provide ductile performance where flexural yielding can potentially occur. For SDC D, E, and F and Site Classes A, B, C, and D, one-half of the transverse reinforcement for special moment frame columns is acceptable because some level of confinement is attributed to competent soils. For Site Class E and F, full column confinement is required because the soils are either liquefiable or not considered competent enough to provide confinement.

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Table 18.13.5.7.1—Minimum reinforcement for uncased cast-in-place or augered concrete piles or piers

<table>
<tr><th colspan="2">Minimum reinforcement</th><th>SDC C –
All Site Classes</th><th>SDC D, E, and F –
Site Class A, B, C, and D</th><th>SDC D, E, and F –
Site Class E and F</th></tr>
<tr><td colspan="2">Minimum longitudinal reinforcement ratio (minimum number of bars)</td><td>0.0025
(minimum number of bars in accordance with 10.7.3.1)</td><td>0.005
(minimum number of bars in accordance with 10.7.3.1)</td><td>0.005
(minimum number of bars in accordance with 10.7.3.1)</td></tr>
<tr><td colspan="2">Minimum reinforced pile length</td><td>Longest of (a) through (d):
(a) 1/3 pile length
(b) 10 ft
(c) 3 times the pile diameter
(d) Flexural length of pile - distance from bottom of pile cap to where $0.4M_{cr}$ exceeds M_u</td><td>Longest of (a) through (d):
(a) 1/2 pile length
(b) 10 ft
(c) 3 times the pile diameter
(d) Flexural length of pile - distance from bottom of pile cap to where $0.4M_{cr}$ exceeds M_u</td><td>Full length of pile except in accordance with [1] or [2].</td></tr>
<tr><td rowspan="4">Transverse confinement reinforcement zone</td><td>Length of reinforcement zone</td><td>3 times the pile diameter from the bottom of the pile cap</td><td>3 times the pile diameter from the bottom of the pile cap</td><td>7 times the pile diameter from the bottom of the pile cap</td></tr>
<tr><td rowspan="2">Type of transverse reinforcement</td><td rowspan="2">Closed ties or spirals with a minimum 3/8 in. diameter</td><td colspan="2">Minimum of No. 3 closed tie or 3/8 in. diameter spiral for piles ≤ 20 in. diameter
Minimum No. 4 closed tie or 1/2 in. diameter spiral for piles > 20 in. diameter</td></tr>
<tr><td colspan="2">In accordance with 18.7.5.2</td></tr>
<tr><td>Spacing and amount of transverse reinforcement</td><td>Spacing shall not exceed lesser of 6 in. or 8 longitudinal bar diameters</td><td>In accordance with 18.7.5.3 and not less than one-half the requirement of Table 18.7.5.4 Item (e)</td><td>In accordance with 18.7.5.3 and not less than the requirement of Table 18.7.5.4 Item (e).</td></tr>
<tr><td rowspan="3">Transverse reinforcement in remainder of reinforced pile length</td><td rowspan="2">Type of transverse reinforcement</td><td rowspan="2">Closed ties or spirals with minimum 3/8 in. diameter</td><td colspan="2">Minimum of No. 3 closed tie or 3/8 in. diameter spiral for piles ≤ 20 in. diameter
Minimum of No. 4 closed tie or 1/2 in. diameter spiral for piles > 20 in. diameter</td></tr>
<tr><td colspan="2">In accordance with 18.7.5.2</td></tr>
<tr><td>Spacing and amount of transverse reinforcement</td><td>Maximum spacing of 16 longitudinal bar diameters</td><td colspan="2">Spacing shall not exceed the least of (a) through (c):
(a) 12 longitudinal bar diameters
(b) 1/2 the pile diameter
(c) 12 in.</td></tr>
</table>

[1] For piles sufficiently embedded in firm soil or rock, reinforcement shall be permitted to be terminated a length above the tip equal to the lesser of 5 percent of the pile length and 33 percent of the length of the pile within rock or firm soil.

[2] In lieu of providing full length minimum flexural reinforcement, the deep foundation element shall be designed to withstand maximum imposed curvatures from the earthquake ground motions and structural response. Curvatures shall include free-field soil strains modified for soil-foundation-structure interaction coupled with foundation element deformations associated with earthquake loads imparted to the foundation by the structure. Minimum reinforced length shall not be less than the requirement for SDC D, E, or F; Site Class D.

18.13.5.7.2 Minimum longitudinal and transverse reinforcement shall be provided along minimum reinforced lengths measured from the top of the pile in accordance with Table 18.13.5.7.1.

18.13.5.7.3 Longitudinal reinforcement shall extend at least the development length, calculated for f_y in tension, beyond the flexural length of the pile, which is defined in Table 18.13.5.7.1 as the distance from the bottom of the pile cap to where $0.4M_{cr} > M_u$.

R18.13.5.7.3 Reinforcement should extend ℓ_d beyond the point where plain concrete is no longer adequate to resist the factored moment.

18.13.5.8 *Metal-cased concrete piles*

R18.13.5.8 *Metal-cased concrete piles*

18.13.5.8.1 For structures assigned to SDC C, D, E, or F, longitudinal reinforcement requirements and minimum reinforced lengths for metal-cased concrete piles shall be the same as for uncased concrete piles in 18.13.5.7.

18.13.5.8.2 Metal-cased concrete piles shall have a spiral-welded metal casing of a thickness not less than 0.0747 in.

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(No. 14 gauge) that is adequately protected from possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

18.13.5.9 *Concrete-filled pipe piles*

18.13.5.9.1 For structures assigned to SDC C, D, E or F, concrete-filled pipe piles shall have longitudinal reinforcement in the top of the pile with a total area of at least $0.01A_g$ and with a minimum length within the pile equal to two times the required embedment length into the pile cap, but not less than the development length, calculated for f_y in tension, of the reinforcement.

18.13.5.10 *Precast concrete piles*

18.13.5.10.1 For precast concrete driven piles, the length of transverse reinforcement provided shall be sufficient to account for potential variations in the elevation of pile tips.

18.13.5.10.2 Precast nonprestressed concrete piles for structures assigned to SDC C shall satisfy (a) through (d):

(a) Minimum longitudinal steel reinforcement ratio shall be 0.01.

(b) Longitudinal reinforcement shall be enclosed within a minimum of No. 3 closed ties or 3/8 in. diameter spirals, for up to 20 in. diameter piles, and No. 4 closed ties or 1/2 in. diameter spirals, for larger diameter piles.

(c) Spacing of transverse reinforcement within a distance of 3 times the least cross-sectional dimension of the pile from the bottom of the pile cap shall not exceed the lesser of 8 times the diameter of the smallest longitudinal bar and 6 in.

(d) Transverse reinforcement shall be provided throughout the length of the pile at a spacing not exceeding 6 in.

18.13.5.10.3 For structures assigned to SDC D, E, or F, precast nonprestressed concrete piles shall satisfy the requirements of 18.13.5.10.2 and the requirements for uncased cast-in-place or augered concrete piles in SDC D, E, or F in Table 18.13.5.7.1.

18.13.5.10.4 For structures assigned to SDC C, precast-prestressed concrete piles shall satisfy (a) and (b):

(a) If the transverse reinforcement consists of spirals or circular hoops, the volumetric ratio of transverse reinforcement, ρ_s, in the upper 20 ft shall not be less than that

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R18.13.5.8.2 Spiral-welded metal casing with the specified wall thickness provides confinement equivalent to closed ties or spirals required in an uncased concrete pile and eliminates the need for confinement ties.

R18.13.5.9 *Concrete-filled pipe piles*

R18.13.5.9.1 For resistance to uplift forces, concrete bond to the steel pipe is to be ignored in determining anchorage of the pile. Concrete shrinkage can be detrimental to bond, therefore shrinkage should be controlled, or force transfer via other methods such as headed studs or surface irregularities on the pipe should be considered. Reinforcement at the top of the pile is extended into the pile cap to tie the elements together and assist transfer of force to the pile cap.

R18.13.5.10 *Precast concrete piles*

R18.13.5.10.1 The potential for driving precast piles to a tip elevation different than that specified in the construction documents should be considered when detailing the pile. If the pile reaches refusal at a shallower depth, a longer length of pile will need to be cut off. If this possibility is not foreseen, the length of transverse reinforcement required by these provisions may not be provided after the excess pile length is cut off.

R18.13.5.10.4

(a) In a study of minimum confinement reinforcement for prestressed concrete piles (Sritharan et al. 2016), the relationship between curvature ductility demand on prestressed piles and overall system ductility demand was considered in the context of all soil profiles identified in ASCE/SEI 7. It was concluded that Eq. (18.13.5.10.4b)

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calculated by Eq. (18.13.5.10.4a) or calculated from a more detailed analysis by Eq. (18.13.5.10.4b):

$$0.15\left(\frac{f_c'}{f_{yt}}\right) \qquad (18.13.5.10.4a)$$

$$0.04\left(\frac{f_c'}{f_{yt}}\right)\left(2.8+\frac{2.3P_u}{f_c'A_g}\right) \qquad (18.13.5.10.4b)$$

and f_{yt} shall not be taken greater than 100,000 psi.

(b) A minimum of one-half of the volumetric ratio of spiral reinforcement required by Eq. (18.13.5.10.4a) or Eq. (18.13.5.10.4b) shall be provided for the remaining length of the pile.

18.13.5.10.5 For structures assigned to SDC D, E, or F, precast-prestressed concrete piles shall satisfy (a) through (e) and the ductile pile region shall be defined as the length of pile measured from the bottom of the pile cap to the point of zero curvature plus 3 times the least pile dimension, but not less than 35 ft. If the total pile length in the soil is 35 ft or less, the ductile pile region shall be taken as the entire length of the pile:

(a) In the ductile pile region, the center-to-center spacing of spirals or hoop reinforcement shall not exceed the least of 0.2 times the least pile dimension, 6 times the diameter of the longitudinal strand, and 6 in.

(b) Spiral reinforcement shall be spliced by lapping one full turn, by welding, or by the use of a mechanical splice. If spiral reinforcement is lap spliced, the ends of the spiral shall terminate in a seismic hook. Mechanical and welded splices of deformed bars shall comply with 25.5.7.

(c) If the transverse reinforcement consists of spirals, or circular hoops, the volumetric ratio of transverse reinforcement, ρ_s, in the ductile pile region shall not be less than that calculated by Eq. (18.13.5.10.5a) or calculated from a more detailed analysis by Eq. (18.13.5.10.5b), and the required volumetric ratio shall be permitted to be obtained by providing an inner and outer spiral.

$$0.2\left(\frac{f_c'}{f_{yt}}\right) \qquad (18.13.5.10.5a)$$

$$0.06\left(\frac{f_c'}{f_{yt}}\right)\left(2.8+\frac{2.3P_u}{f_c'A_g}\right) \qquad (18.13.5.10.5b)$$

and f_{yt} shall not be taken as greater than 100,000 psi.

(d) Outside of the ductile pile region, spiral or hoop reinforcement shall be provided with a volumetric ratio not less than one-half of that required within the ductile pile region, and the maximum spacing shall be in accordance with Table 13.4.5.6(b).

(e) If transverse reinforcement consists of rectangular hoops and crossties, the total cross-sectional area of lateral transverse reinforcement in the ductile region shall be the greater of Eq. (18.13.5.10.5c) and Eq. (18.13.5.10.5d). The hoops and crossties shall be equivalent to deformed

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results in adequate deformation capacity for structures assigned to SDC C. The factored axial force on a pile should be determined from Eq. (5.3.1c) and Eq. (5.3.1g) with 5.3.7 and 5.3.8 as applicable.

R18.13.5.10.5 Observed damage from earthquakes and concerns about the accuracy of calculated pile demands have led to prescriptive requirements for confinement of potential yielding regions of piles. The required confinement is intended to provide adequate ductility capacity for structures assigned to SDC D, E, and F (Sritharan et al. 2016).

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bars not less than No. 3 in size, and rectangular hoop ends shall terminate at a corner with seismic hooks.

$$A_{sh} = 0.3 s b_c \left(\frac{f_c'}{f_{yt}}\right)\left(\frac{A_g}{A_{ch}} - 1.0\right)\left(0.5 + \frac{1.4P_u}{f_c' A_g}\right) \quad (18.13.5.10.5c)$$

$$A_{sh} = 0.12 s b_c \left(\frac{f_c'}{f_{yt}}\right)\left(0.5 + \frac{1.4P_u}{f_c' A_g}\right) \quad (18.13.5.10.5d)$$

and f_{yt} shall not be taken as greater than 100,000 psi.

18.13.5.10.6 For structures assigned to SDC C, D, E, or F, the maximum factored axial load that can be applied to precast, prestressed piles subjected to a combination of earthquake lateral force and axial load shall not exceed the following values:

(a) $0.2f_c'A_g$ for square piles with side dimension of 14 in. or less

(b) $0.25f_c'A_g$ for square piles with side dimension greater than 14 in.

(c) $0.4f_c'A_g$ for circular or octagonal piles less than or equal to 24 in. in diameter

(d) $0.45f_c'A_g$ for circular or octagonal piles greater than 24 in. in diameter

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R18.13.5.10.6 The axial load in precast prestressed piles is limited to preclude spalling of the concrete cover prior to the pile section experiencing flexural cracking, as this will result in a significant loss in pile resistance (Sritharan et al. 2016).

Research (Ryan and Mays 2021) indicates an increase in axial load may be permitted for precast prestressed concrete piles with larger cross-sections; hence, the distinction in the axial load limit as a function of pile size.

18.13.6 *Anchorage of piles, piers, and caissons*

R18.13.6 *Anchorage of piles, piers, and caissons*

18.13.6.1 For structures assigned to SDC C, D, E, or F, the longitudinal reinforcement in piles, piers, or caissons resisting tension loads shall be detailed to transfer tension forces within the pile cap to supported structural members.

R18.13.6.1 A load path is necessary at pile caps to transfer tension forces from the reinforcing bars in the column or boundary element through the pile cap to the reinforcement of the pile or caisson. Examples of different types of pile connections to pile caps are available in ASCE/COPRI Standard for the Seismic Design of Piers and Wharves (61-14).

18.13.6.2 For structures assigned to SDC C, D, E, or F, concrete piles and concrete filled pipe piles shall be connected to the pile cap by embedding the pile reinforcement in the pile cap a distance equal to the development length or by the use of field-placed dowels anchored in the concrete pile. For deformed bars, the compression development length is used if the pile is in compression. In the case of uplift, the tension development length is used without reduction in length for excess reinforcement.

R18.13.6.2 Development length is determined according to requirements of Chapter 25. Embedment lengths less than the development length for calculated stresses less than f_y are not permitted, as indicated in 25.4.10.2. Development of the pile longitudinal reinforcement into the pile cap is intended to enable the capacity of the pile to pile cap connection to meet or exceed the pile section strength.

18.13.6.3 For structures assigned to SDC D, E, or F, if tension forces induced by earthquake effects are transferred between pile cap or mat foundation and precast pile by reinforcing bars grouted or post-installed in the top of the pile, the grouting system shall have been demonstrated by testing to develop at least $1.25f_y$ of the bar.

R18.13.6.3 Grouted dowels in a blockout in the top of a precast concrete pile need to be developed, and testing is a practical means of demonstrating strength. Alternatively, reinforcing bars can be cast in the upper portion of the pile, exposed by chipping of concrete and mechanically spliced or welded to an extension.

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18.14—Members not designated as part of the seismic-force-resisting system

18.14.1 *Scope*

18.14.1.1 This section shall apply to members not designated as part of the seismic-force-resisting system in structures assigned to SDC D, E, and F.

18.14.2 *Design actions*

18.14.2.1 Members not designated as part of the seismic-force-resisting system shall be evaluated for gravity load combinations of 5.3 including the effect of vertical ground motion acting simultaneously with the design displacement δ_u.

18.14.3 *Cast-in-place beams, columns, and joints*

18.14.3.1 Cast-in-place beams, columns, and joints shall be detailed in accordance with 18.14.3.2 or 18.14.3.3 depending on the magnitude of moments and shears induced in those members when subjected to the design displacement δ_u. If effects of δ_u are not explicitly checked, the provisions of 18.14.3.3 shall be satisfied.

18.14.3.2 Where the induced moments and shears do not exceed the design moment and shear strength of the frame member, (a) through (d) shall be satisfied:

(a) Beams shall satisfy 18.6.3.1. Transverse reinforcement satisfying 25.7 shall be provided throughout the length of the beam at spacing not to exceed $d/2$. Where factored axial force exceeds $A_g f_c'/10$, transverse reinforcement shall be hoops satisfying 18.7.5.2 at a spacing not to exceed the lesser of $6d_b$ of the smallest enclosed longitudinal bar and 6 in.

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R18.14—Members not designated as part of the seismic-force-resisting system

This section applies only to structures assigned to SDC D, E, or F. For those SDCs, all structural members not designated as a part of the seismic-force-resisting system are required to be designed to support gravity loads and the load effects of vertical ground motion, while subjected to the design displacement. For concrete structures, the provisions of this section satisfy this requirement for columns, beams, slabs, and wall piers of the gravity system.

Design displacement is defined in Chapter 2. Models used to determine design displacement of buildings should be chosen to produce results that conservatively bound the values expected during the design earthquake and should include, as appropriate, effects of concrete cracking, foundation flexibility, and deformation of floor and roof diaphragms.

The provisions of 18.14 are intended to enable ductile flexural yielding of columns, beams, slabs, and wall piers under the design displacement, by providing sufficient confinement and shear strength in elements that yield.

R18.14.3 *Cast-in-place beams, columns, and joints*

R18.14.3.1 Cast-in-place columns and beams are assumed to yield if the combined effects of factored gravity loads and design displacements exceed the strengths specified, or if the effects of design displacements are not calculated. Requirements for transverse reinforcement and shear strength vary with member type and whether the member yields under the design displacement.

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(b) Columns shall satisfy 18.7.4.1 and 18.7.6. Spiral reinforcement satisfying 25.7.3 or hoop reinforcement satisfying 25.7.4 shall be provided over the full length of the column with spacing not to exceed the lesser of $6d_b$ of the smallest enclosed longitudinal bar and 6 in. Transverse reinforcement satisfying 18.7.5.2(a) through (e) shall be provided over a length ℓ_o, as defined in 18.7.5.1, from each joint face.

(c) Columns with factored gravity axial forces exceeding $0.35P_o$ shall satisfy 18.14.3.2(b) and 18.7.5.7. The minimum amount of transverse reinforcement provided shall be, for rectilinear hoops, one-half the greater of Table 18.7.5.4 parts (a) and (b) and, for spiral or circular hoops, one-half the greater of Table 18.7.5.4 parts (d) and (e). This transverse reinforcement shall be provided over a length ℓ_o, as defined in 18.7.5.1, from each joint face.

(d) Joints shall satisfy Chapter 15.

18.14.3.3 Where the induced moments or shears exceed ϕM_n or ϕV_n of the frame member, or if induced moments or shears are not calculated, (a) through (d) shall be satisfied:

(a) Materials, mechanical splices, and welded splices shall satisfy the requirements for special moment frames in 18.2.5 through 18.2.8.

(b) Beams shall satisfy 18.14.3.2(a) and 18.6.5.

(c) Columns shall satisfy 18.7.4, 18.7.5, and 18.7.6, except 18.7.4.3 need not be satisfied.

(d) Joints shall satisfy 18.4.4.1.

18.14.4 *Precast beams and columns*

18.14.4.1 Precast concrete frame members assumed not to contribute to lateral resistance, including their connections, shall satisfy (a) through (d):

(a) Requirements of 18.14.3

(b) Ties specified in 18.14.3.2(b) over the entire column height, including the depth of the beams

(c) Structural integrity reinforcement, in accordance with 4.10

(d) Bearing length at the support of a beam shall be at least 2 in. longer than determined from 16.2.6

18.14.5 *Slab-column connections*

18.14.5.1 For slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the requirements of 18.14.5.3 and either 8.7.6 or 8.7.7 shall be provided at any slab critical section defined in 22.6.4.1 for the following conditions:

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R18.14.4 *Precast beams and columns*

R18.14.4.1 Damage to some buildings with precast concrete gravity systems during the 1994 Northridge earthquake was attributed to several factors addressed in this section. Columns should contain ties over their entire height, frame members not proportioned to resist earthquake forces should be tied together, and longer bearing lengths should be used to maintain integrity of the gravity system during ground motion. The 2 in. increase in bearing length is based on an assumed 4 percent story drift ratio and 50 in. beam depth, and is considered to be conservative for the ground motions expected for structures assigned to SDC D, E, or F. In addition to this provision, precast frame members assumed not to contribute to lateral resistance should also satisfy the requirements for cast-in-place construction addressed in 18.14.3, as applicable.

R18.14.5 *Slab-column connections*

R18.14.5.1 Provisions for shear reinforcement at slab-column connections are intended to reduce the likelihood of slab punching shear failure if the design story drift ratio exceeds the value specified.

No calculation of induced moments is required, based on research (Megally and Ghali 2002; Moehle 1996; Kang and

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(a) Nonprestressed slabs where $\Delta_x/h_{sx} \geq 0.035 - (1/20)(v_{uv}/\phi v_c)$

(b) Unbonded post-tensioned slabs with f_{pc} in each direction meeting the requirements of 8.6.2.1, where $\Delta_x/h_{sx} \geq 0.040 - (1/20)(v_{uv}/\phi v_c)$

The load combinations to be evaluated for v_{uv} shall only include those with E. The value of (Δ_x/h_{sx}) shall be taken as the greater of the values of the adjacent stories above and below the slab-column connection, v_c shall be calculated in accordance with 22.6.5; and, for unbonded post-tensioned slabs, the value of V_p shall be taken as zero when calculating v_c.

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Wallace 2006; Kang et al. 2007) that identifies the likelihood of punching shear failure considering the story drift ratio and shear stress v_{uv} due to gravity loads and the vertical component of earthquake loads, without moment transfer, about the slab critical section. Figure R18.14.5.1 illustrates the requirement for nonprestressed and unbonded post-tensioned slab-column connections. The requirement can be satisfied by adding slab shear reinforcement, increasing slab thickness, changing the design to reduce the design story drift ratio, or a combination of these.

If column capitals, drop panels, shear caps, or other changes in slab thickness are used, the requirements of 18.14.5 are evaluated at all potential critical sections, as required by 22.6.5.1.

Post-tensioned slab-column connections with f_{pc} in each direction not meeting the requirements of 8.6.2.1 can be designed as nonprestressed slab-column connections in accordance with 8.2.3.

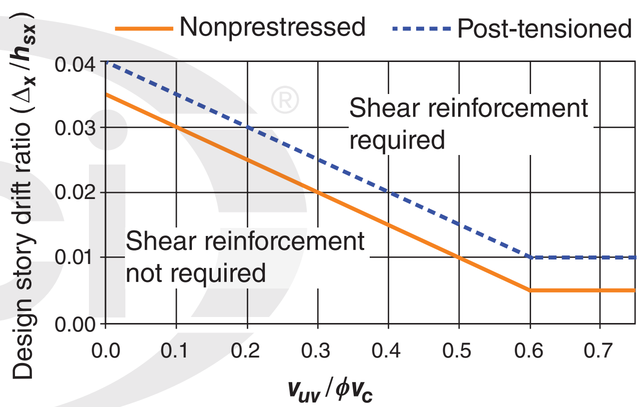

Fig. R18.14.5.1—Illustration of the criteria of 18.14.5.1.

18.14.5.2 The shear reinforcement requirements of 18.14.5.1 need not be satisfied if (a) or (b) is met:

(a) $\Delta_x/h_{sx} \leq 0.005$ for nonprestressed slabs

(b) $\Delta_x/h_{sx} \leq 0.01$ for unbonded post-tensioned slabs with f_{pc} in each direction meeting the requirements of 8.6.2.1

18.14.5.3 Required slab shear reinforcement shall provide $v_s \geq 3.5\sqrt{f_c'}$ at the slab critical section and shall extend at least four times the slab thickness from the face of the support adjacent to the slab critical section.

18.14.6 *Wall piers*

18.14.6.1 Wall piers not designated as part of the seismic-force-resisting system shall satisfy the requirements of 18.10.8. Where the general building code includes provisions to account for overstrength of the seismic-force-resisting system, it shall be permitted to calculate the design shear force as Ω_o times the shear induced under design displacements, δ_u.

R18.14.6 *Wall piers*

R18.14.6.1 Section 18.10.8 requires that the design shear force be determined according to 18.7.6.1, which in some cases may result in unrealistically large forces. As an alternative, the design shear force can be determined as the product of an overstrength factor and the shear induced when the wall pier is displaced by δ_u. The overstrength factor Ω_o included in FEMA P-749, ASCE/SEI 7, and the 2021 IBC can be used for this purpose.