CHAPTER 18—EARTHQUAKE-RESISTANT STRUCTURES

18.1—Scope

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

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).

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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 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.

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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.

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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*

R18.2.8 *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.

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).

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

R18.2.8.2 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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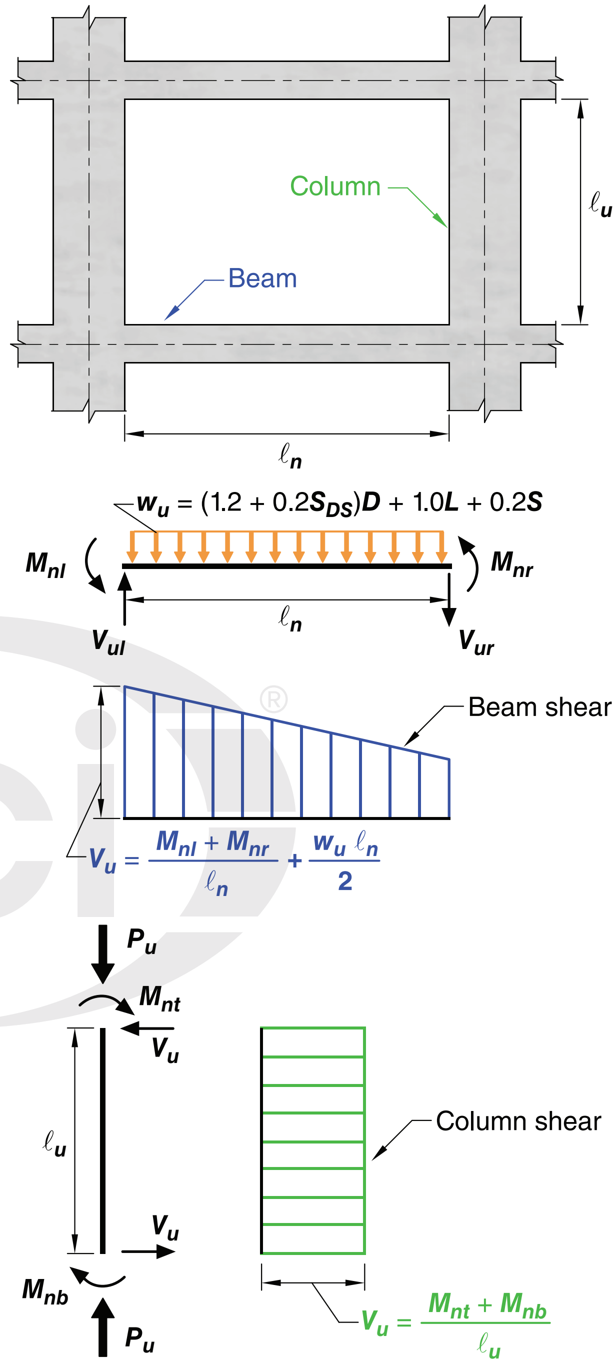

Fig. R18.4.2—Design shears for intermediate moment frames.

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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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, Ω_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:

$$\phi V_n \geq V_u$$

COMMENTARY

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.

COMMENTARY

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.

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*

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.

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.

R18.4.5.1

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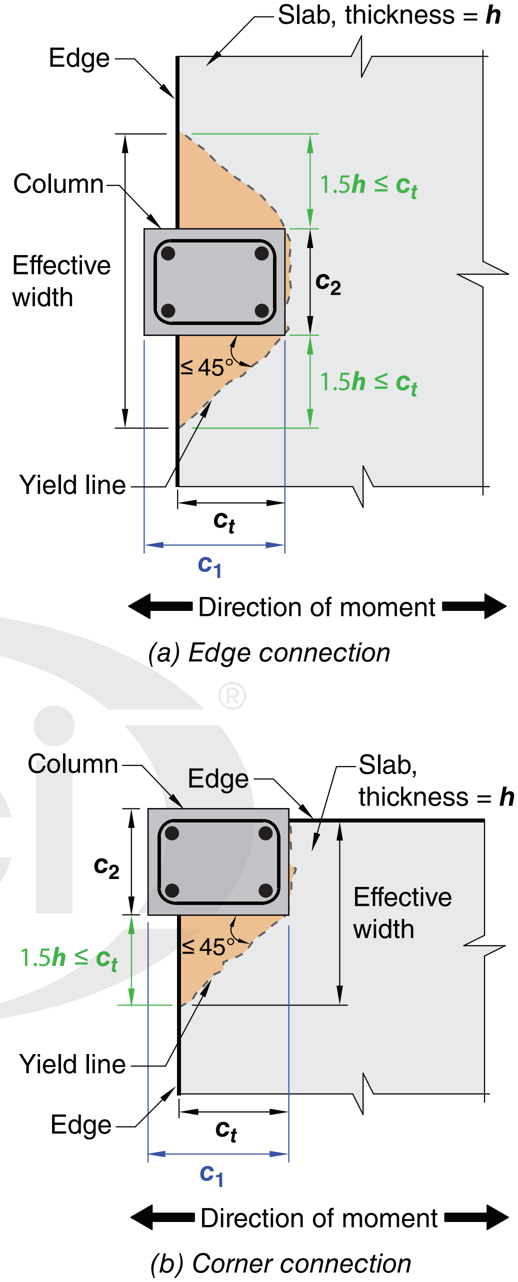

Fig. R18.4.5.1—Effective width for reinforcement placement in edge and corner connections.

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.

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.

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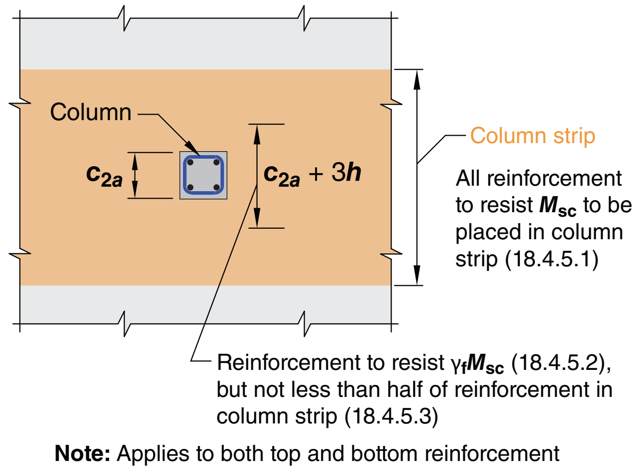

Fig. R18.4.5.2—Location of reinforcement in slabs.

R18.4.5.3

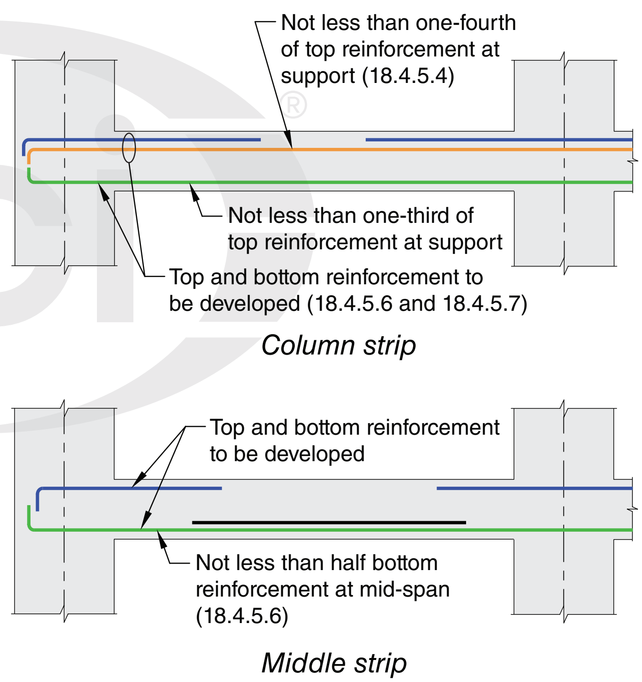

Fig. R18.4.5.3—Arrangement of reinforcement in slabs.

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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.

18.5—Intermediate precast structural walls

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}.

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*

COMMENTARY

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.

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.

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

R18.6—Beams of special moment frames

R18.6.1 *Scope*

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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*

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.

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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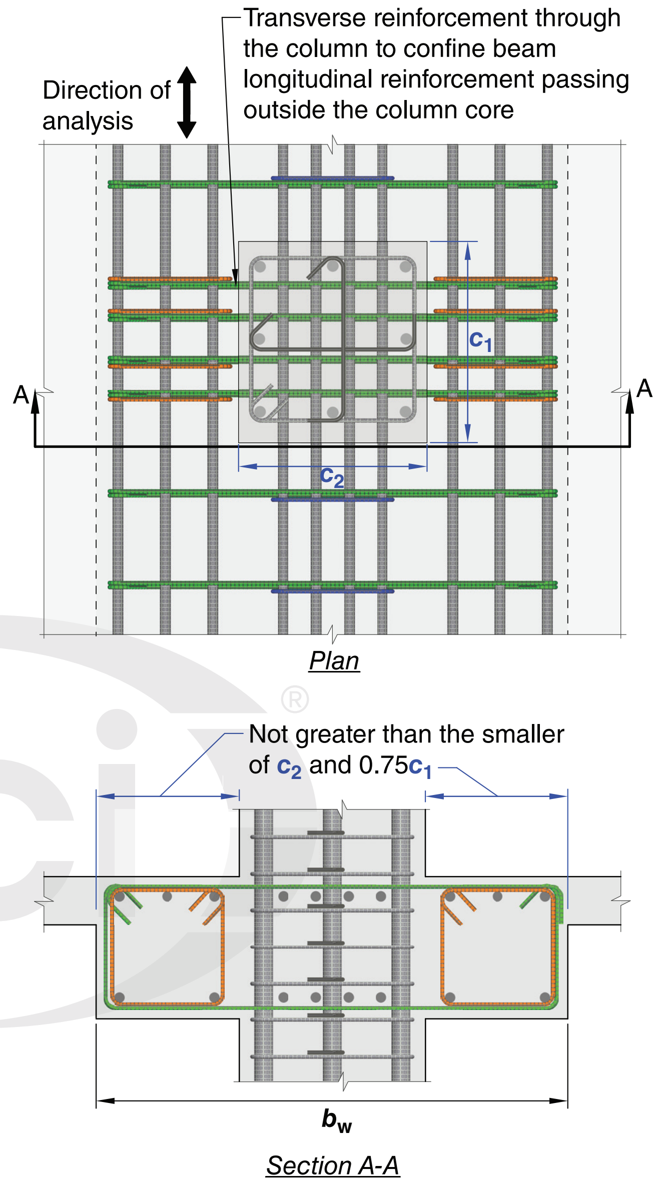

Fig. R18.6.2—Maximum effective width of wide beam and required transverse reinforcement.

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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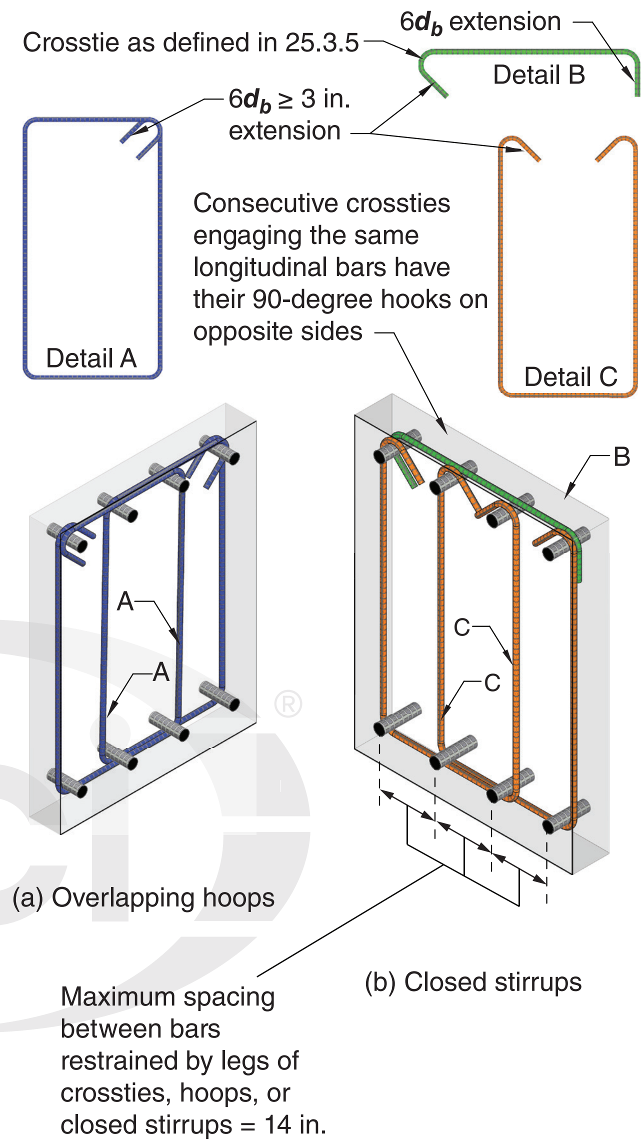

Fig. R18.6.4—Examples of beam transverse reinforcement and illustration of limit on maximum horizontal spacing of supported longitudinal bars.

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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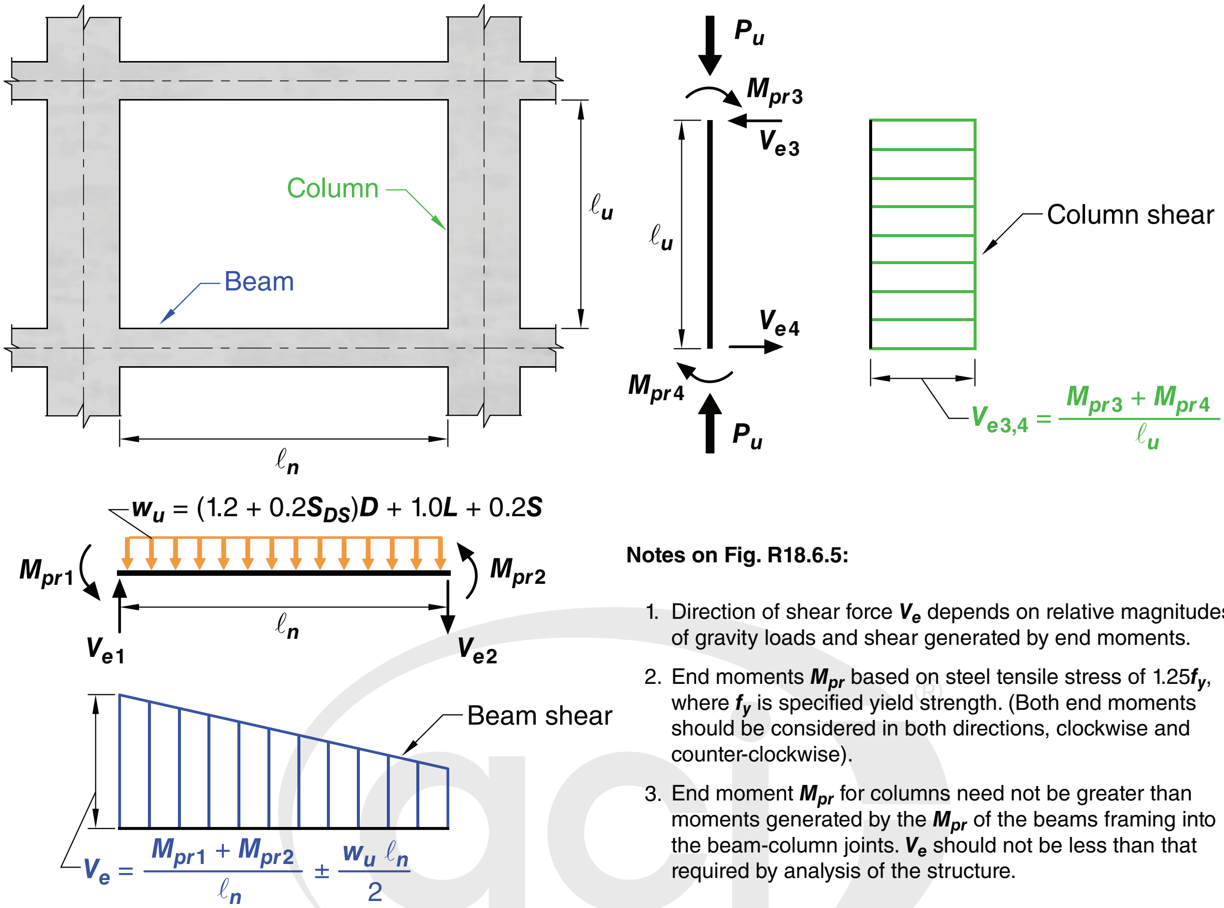

Notes on Fig. R18.6.5:

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*

COMMENTARY

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

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

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

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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.

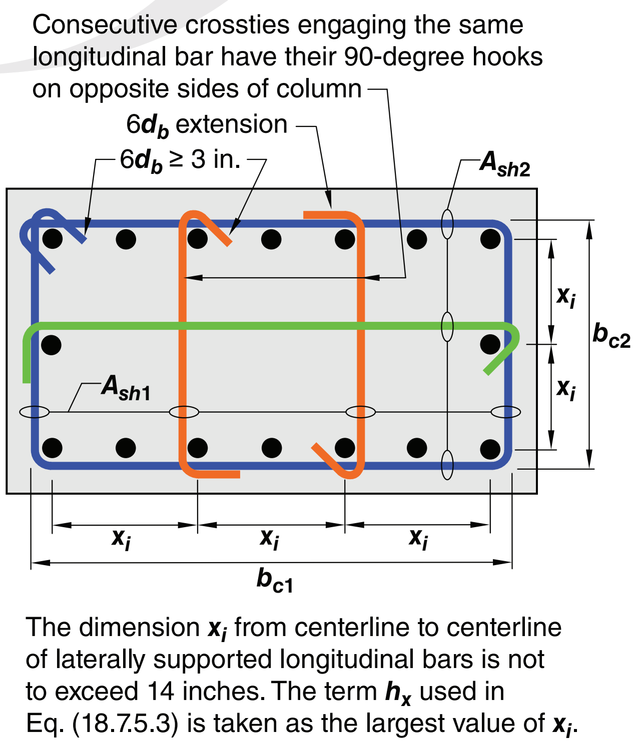

Fig. R18.7.5.2—Example of transverse reinforcement in columns.

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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:

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

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).

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

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

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.

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

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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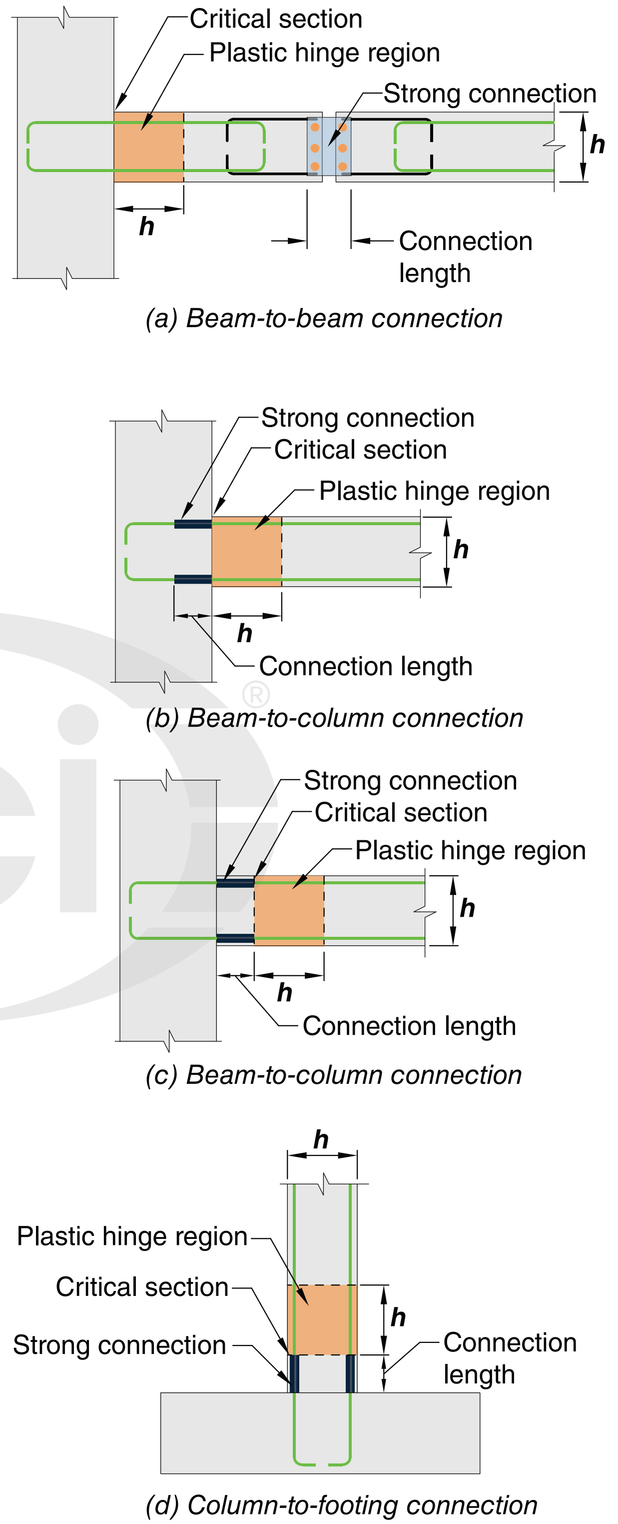

Fig. R18.9.2.2—Strong connection examples.

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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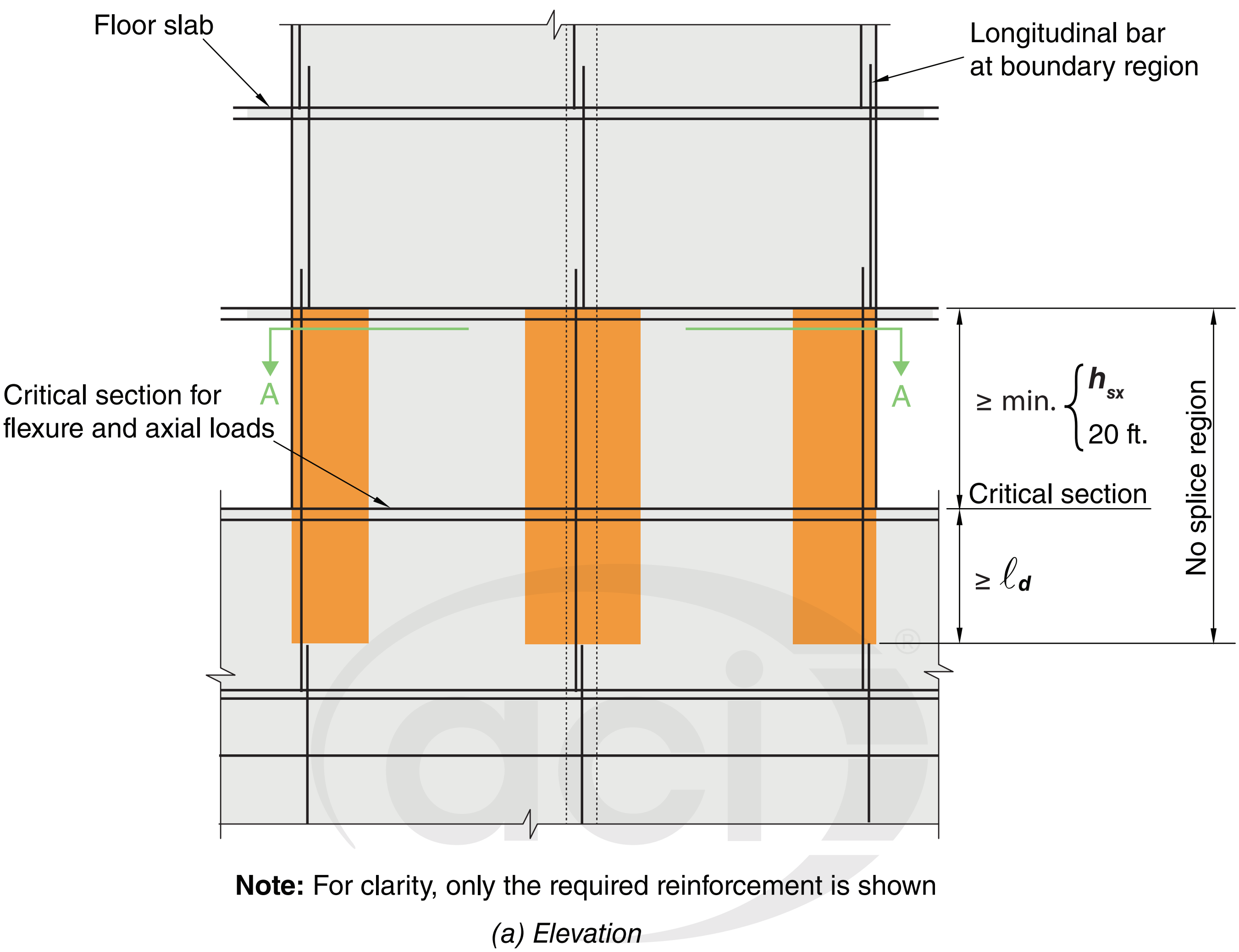

Fig. R18.10.2.3—Wall boundary regions within heights where lap splices are not permitted.

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-