CHAPTER 18—EARTHQUAKE-RESISTANT STRUCTURES

CODE

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

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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.

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

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

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

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18.2.3 *Anchoring to concrete*

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

18.2.4 *Strength reduction factors*

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

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

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

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

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

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R18.2.4 *Strength reduction factors*

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

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

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

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

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

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

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

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

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

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

R18.2.7 *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.

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.

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

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

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

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

R18.2.7.2 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.

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

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

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

18.3—Ordinary moment frames

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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.

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

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

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

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

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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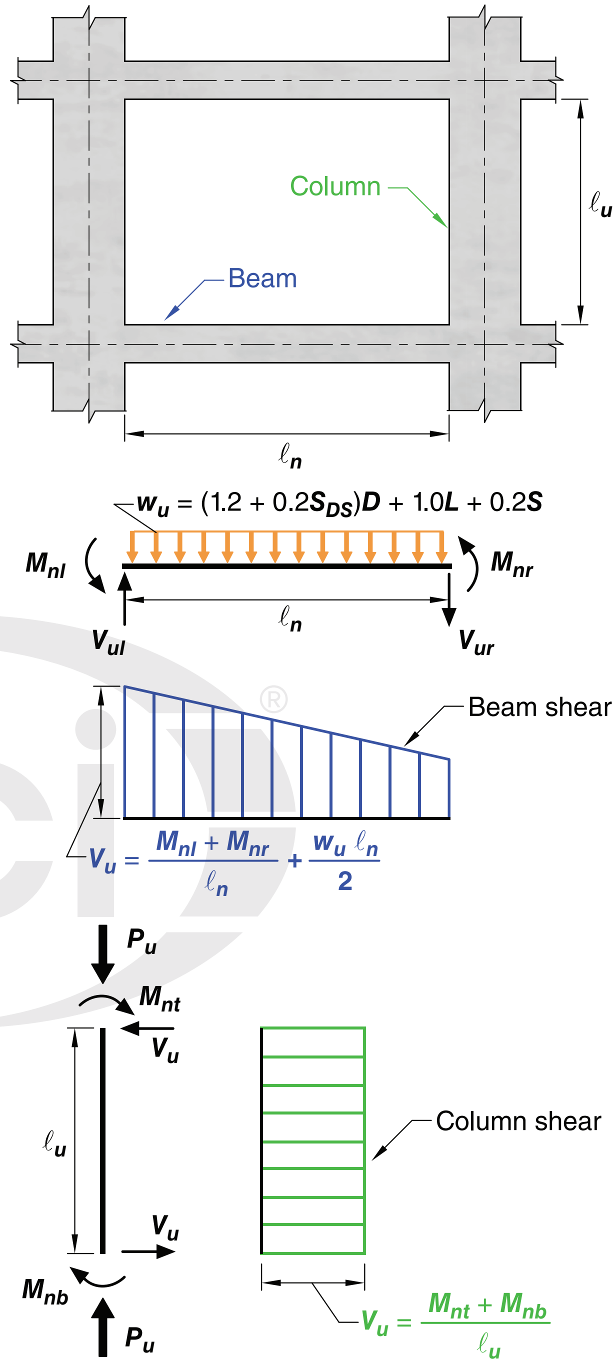

Fig. R18.4.2—Design shears for intermediate moment frames.

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.

18.4.4.7.3 ϕ shall be in accordance with 21.2.1 for shear.

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

18.4.5 *Two-way slabs without beams*

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

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.

R18.4.5 *Two-way slabs without beams*

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

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

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

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

R18.4.5.1

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COMMENTARY

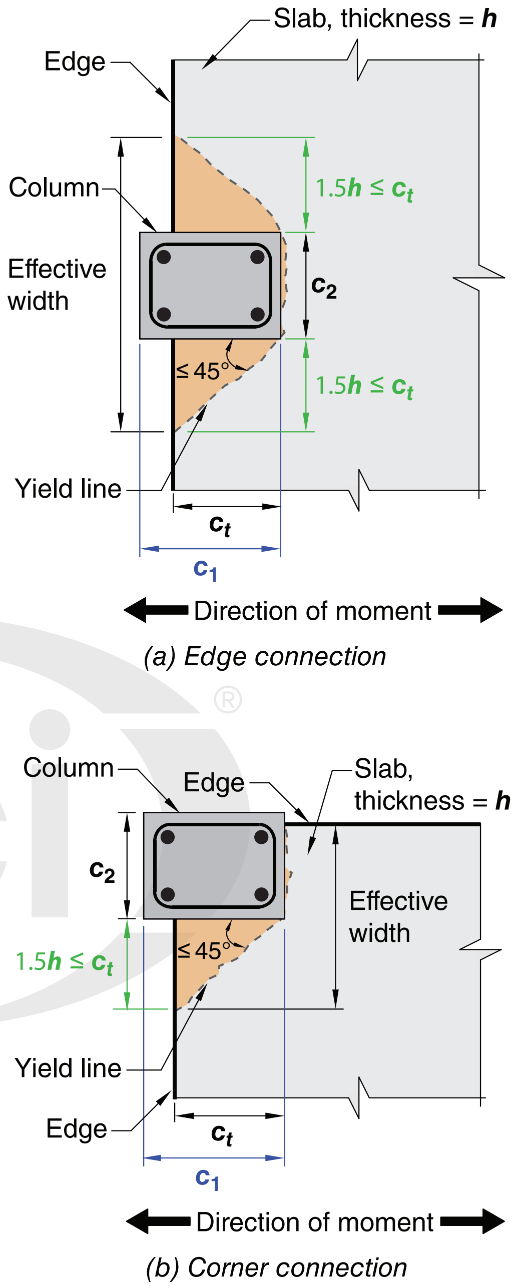

Fig. R18.4.5.1—Effective width for reinforcement placement in edge and corner connections.

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.

COMMENTARY

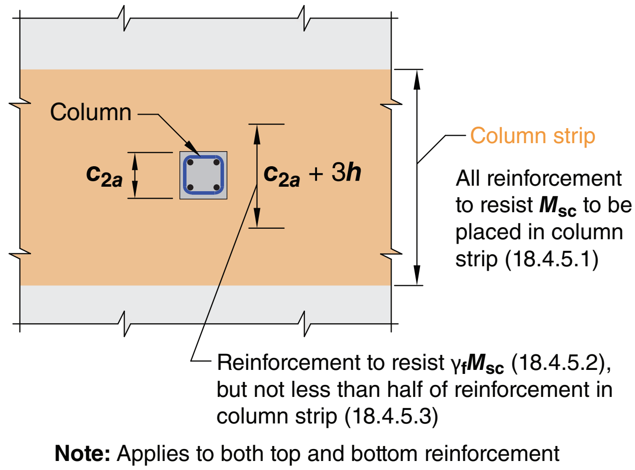

Fig. R18.4.5.2—Location of reinforcement in slabs.

R18.4.5.3

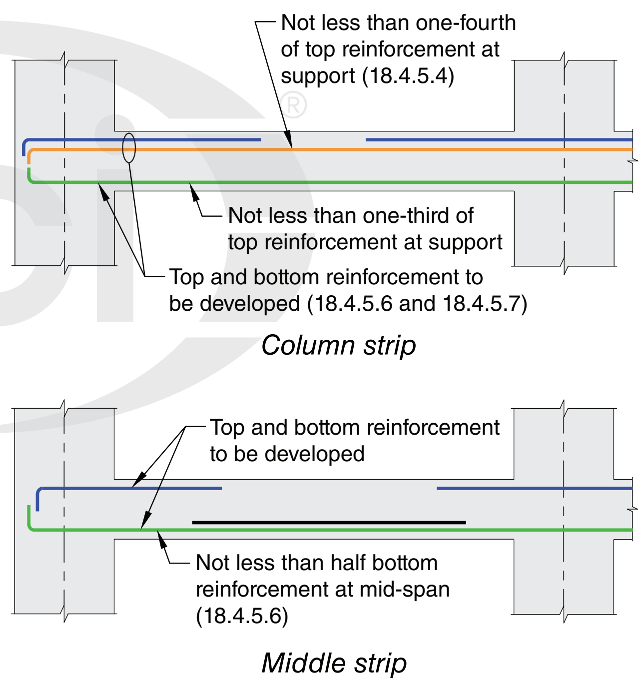

Fig. R18.4.5.3—Arrangement of reinforcement in slabs.

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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.

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.

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.

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.

18.6—Beams of special moment frames

18.6.1 *Scope*

R18.6—Beams of special moment frames

R18.6.1 *Scope*

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

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

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

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

18.6.2 *Dimensional limits*

R18.6.2 *Dimensional limits*

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

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

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COMMENTARY

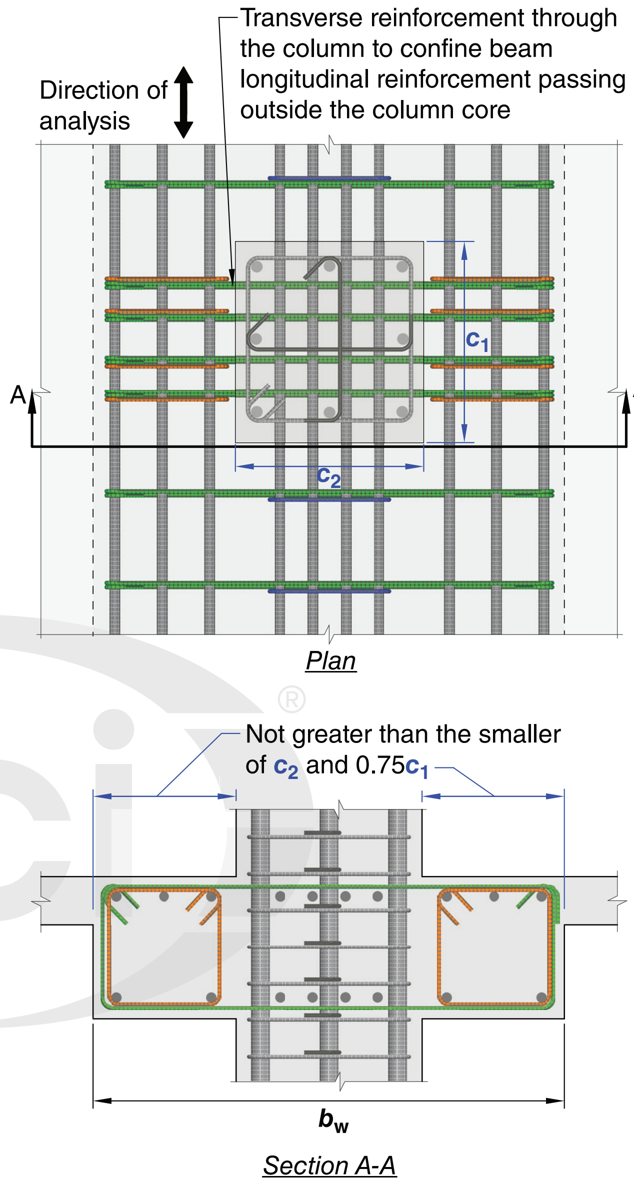

Fig. R18.6.2—Maximum effective width of wide beam and required transverse reinforcement.

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.

COMMENTARY

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-