CODE

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

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

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

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

COMMENTARY

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

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

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

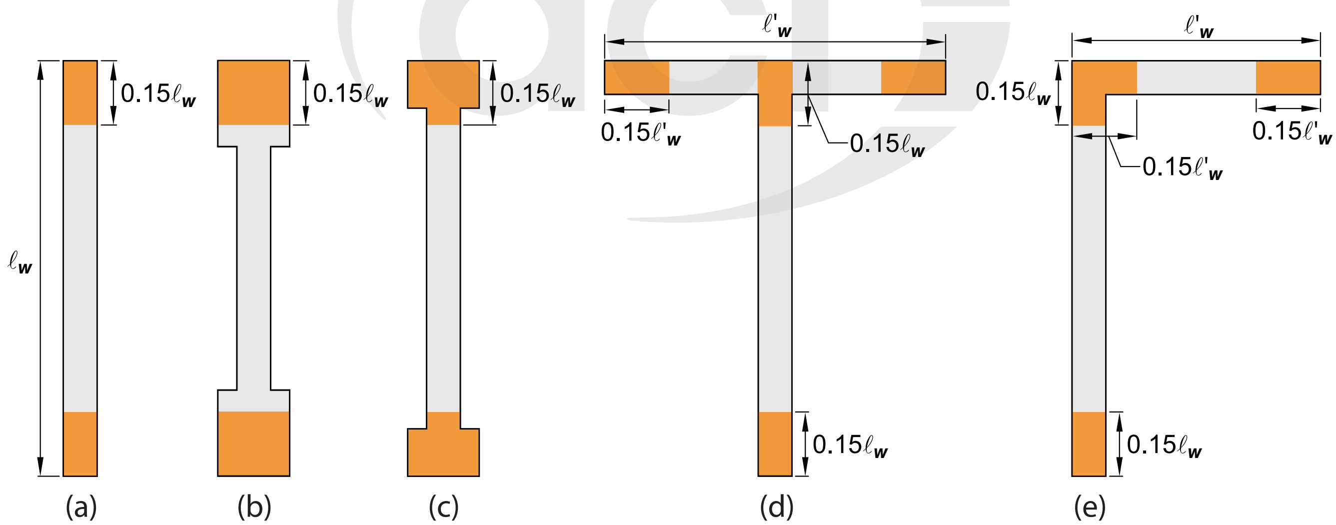

Fig. R18.10.2.4—Locations of longitudinal reinforcement required by 18.10.2.4(a) in different configurations of wall sections.

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

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

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

CODE

18.10.3 *Design forces*

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

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

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

COMMENTARY

R18.10.3 *Design forces*

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

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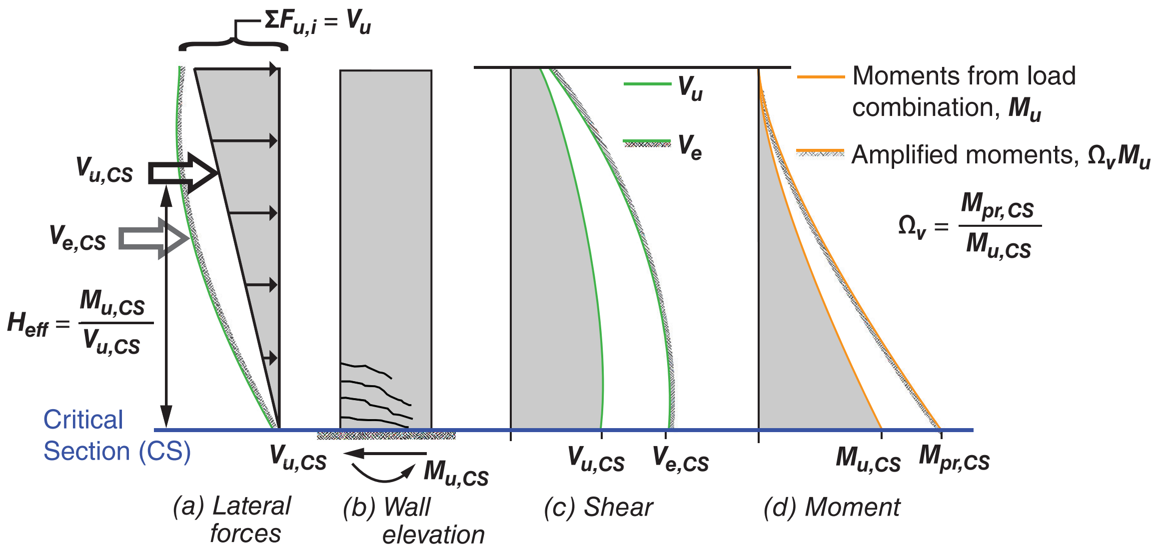

Fig. R18.10.3.3—Determination of shear demand for walls with $h_w/\ell_w \geq 2.0$ *(Moehle et al. 2011).*

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

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

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

Table 18.10.3.3.3—Factors Ω_v and ω_v

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

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

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

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

CODE

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

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

18.10.4 *Shear strength*

COMMENTARY

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

R18.10.4 *Shear strength*

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

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

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

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

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

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

CODE

18.10.4.1 V_n shall be calculated by:

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

where

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

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

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

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

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

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

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

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

COMMENTARY

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

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

CODE

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

COMMENTARY

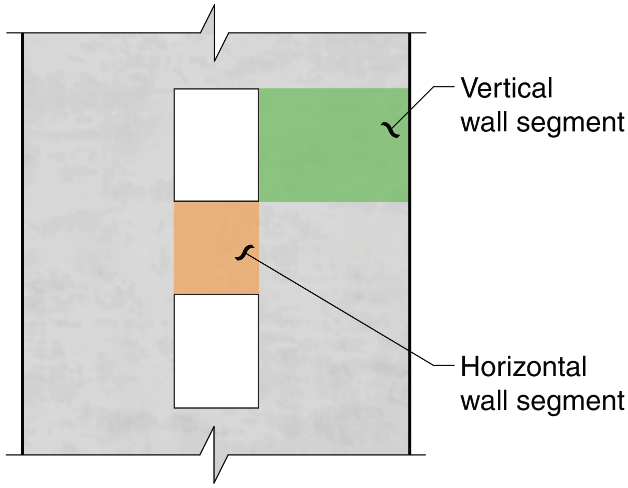

Fig. R18.10.4.5—Wall with openings.

18.10.5 *Design for flexure and axial force*

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

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

R18.10.5 *Design for flexure and axial force*

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

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

18.10.6 *Boundary elements of special structural walls*

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

R18.10.6 *Boundary elements of special structural walls*

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

CODE

COMMENTARY

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

CODE

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

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

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

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

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

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

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

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

COMMENTARY

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

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

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

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

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

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

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

CODE

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

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

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

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

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

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

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

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

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

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

COMMENTARY

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

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

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

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

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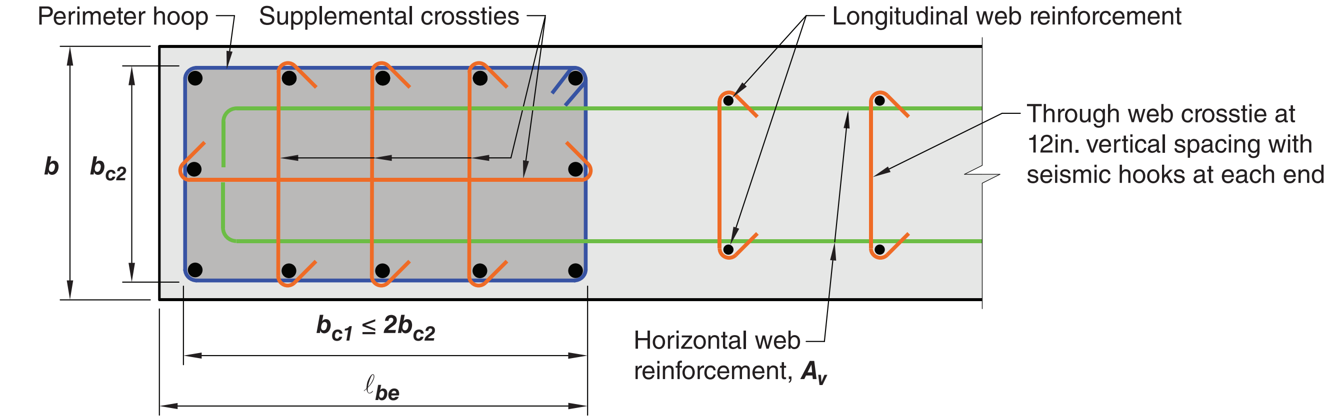
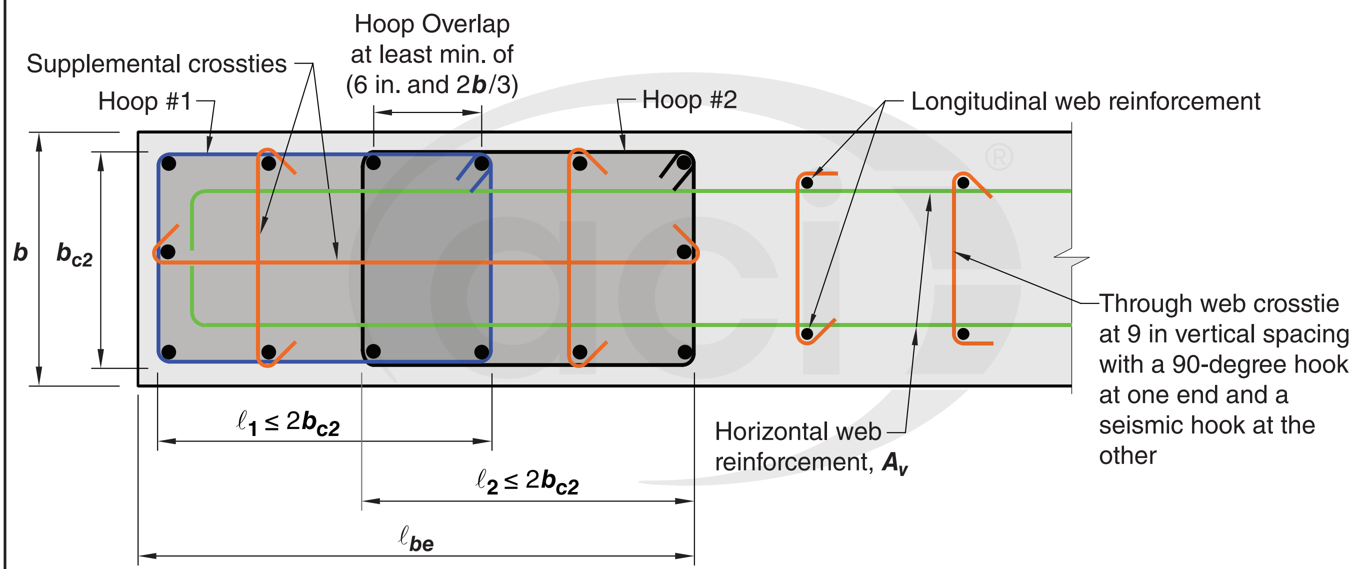


(a) Perimeter hoop with supplemental 135-degree crossties and 135-degree crossties supporting distributed web longitudinal reinforcement



(b) Overlapping hoops with supplemental 135-degree crossties and 135-degree crossties supporting distributed web longitudinal reinforcement

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

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

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

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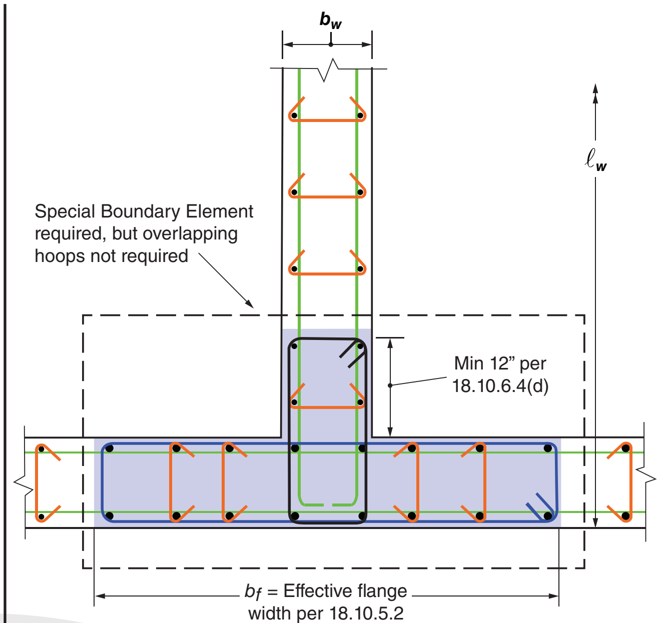

Fig. R18.10.6.4b—Example configuration for a case if a special boundary element is required for a flanged wall

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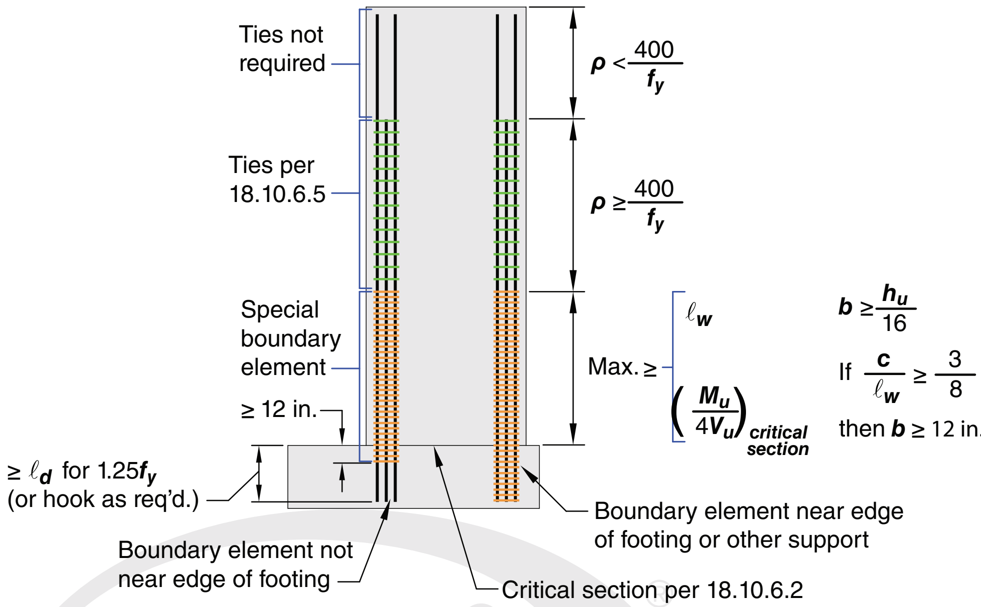
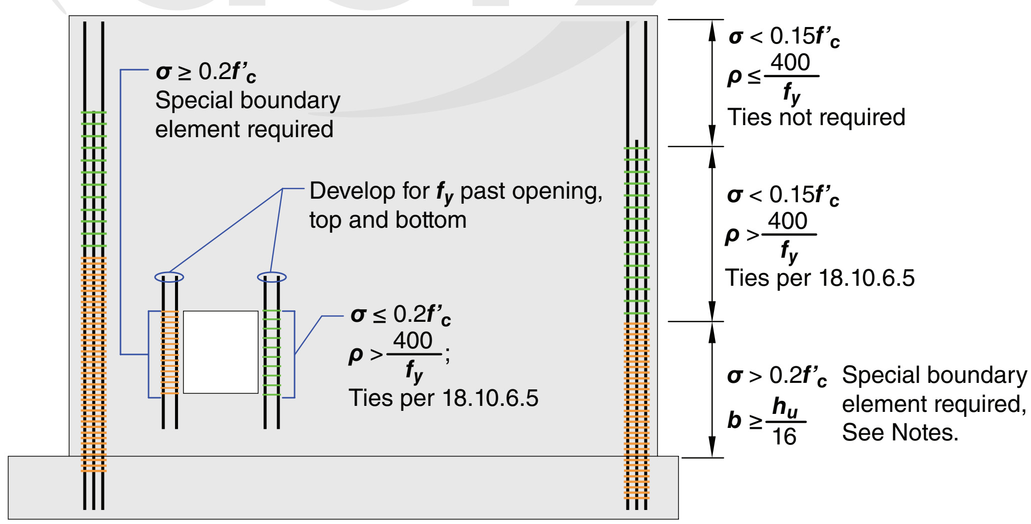


(a) Wall with $h_w/\ell_w \geq 2.0$ and a single critical section controlled by flexure and axial load designed using 18.10.6.2, 18.10.6.4, and 18.10.6.5



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

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

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

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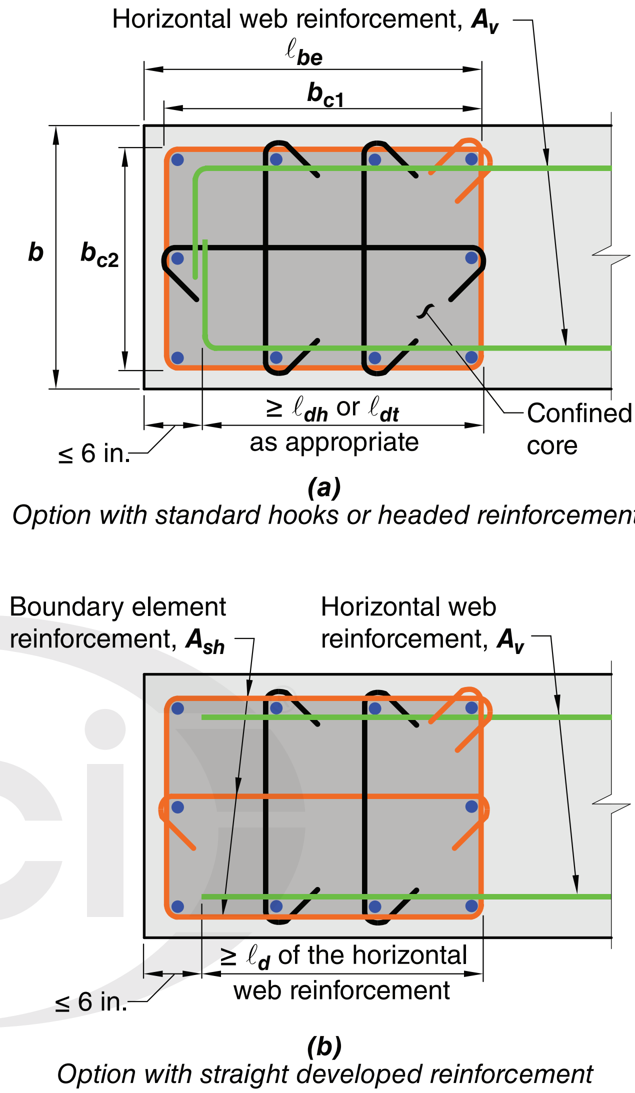

Fig. R18.10.6.4d—Development of wall horizontal reinforcement in confined boundary element.

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

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

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

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

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

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

COMMENTARY

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

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

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

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

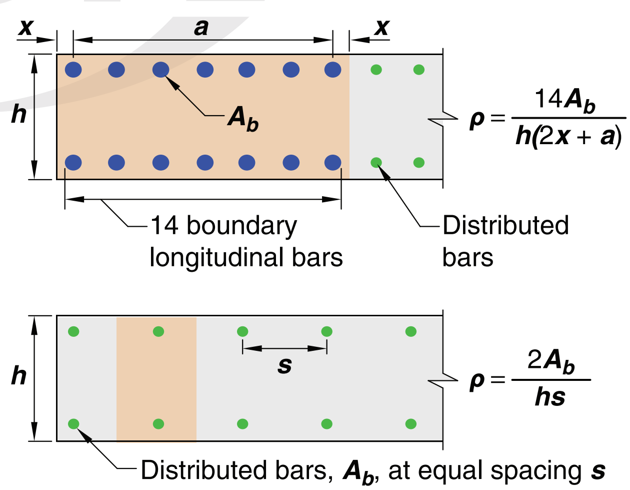

Fig. R18.10.6.5a—Longitudinal reinforcement ratios for typical wall boundary conditions.

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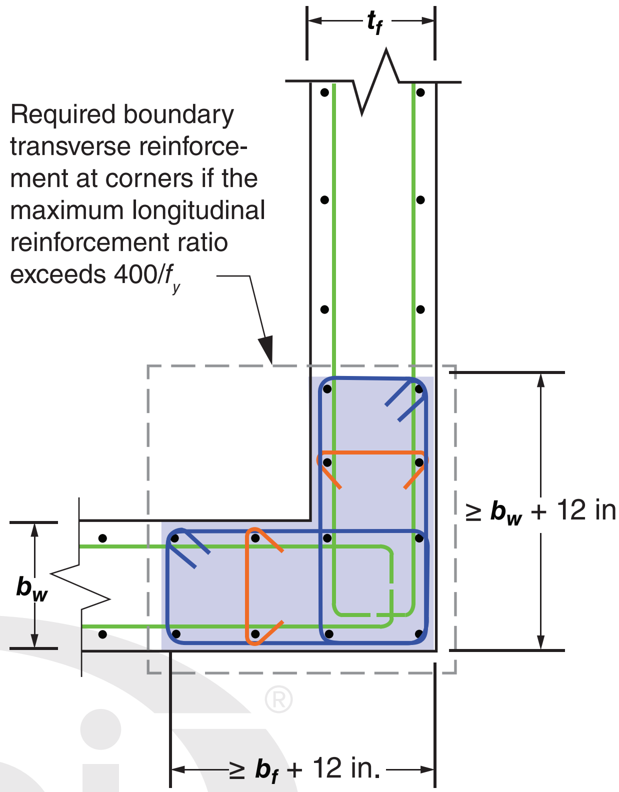

Fig. R18.10.6.5b—Transverse reinforcement at the intersection of a wall web and flange.

18.10.7 *Coupling beams*

R18.10.7 *Coupling beams*

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

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

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

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

CODE

COMMENTARY

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

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

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

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

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

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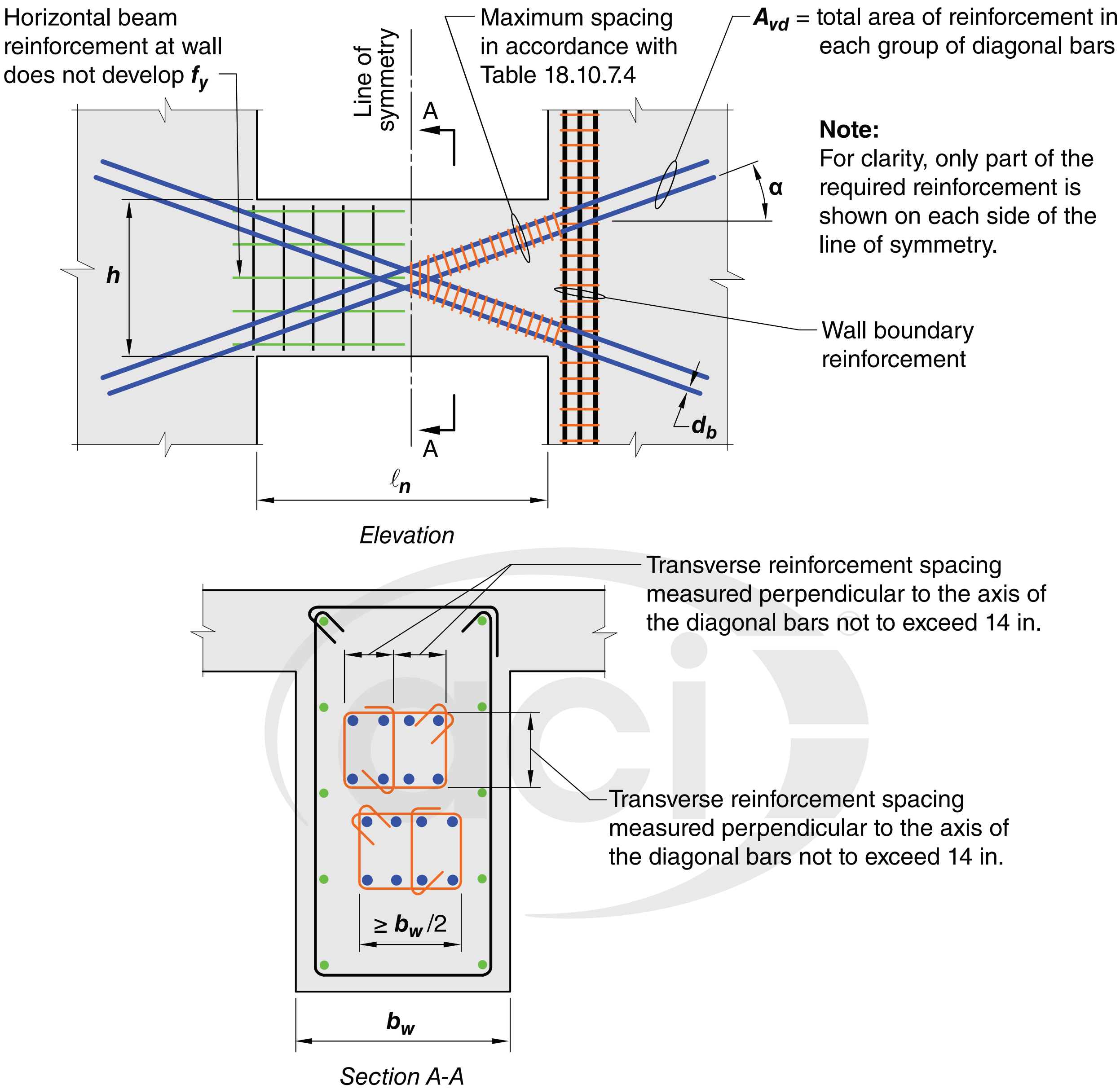

Fig. R18.10.7a—Confinement of individual diagonals in coupling beams with diagonally oriented reinforcement. Wall boundary reinforcement shown on one side only for clarity.

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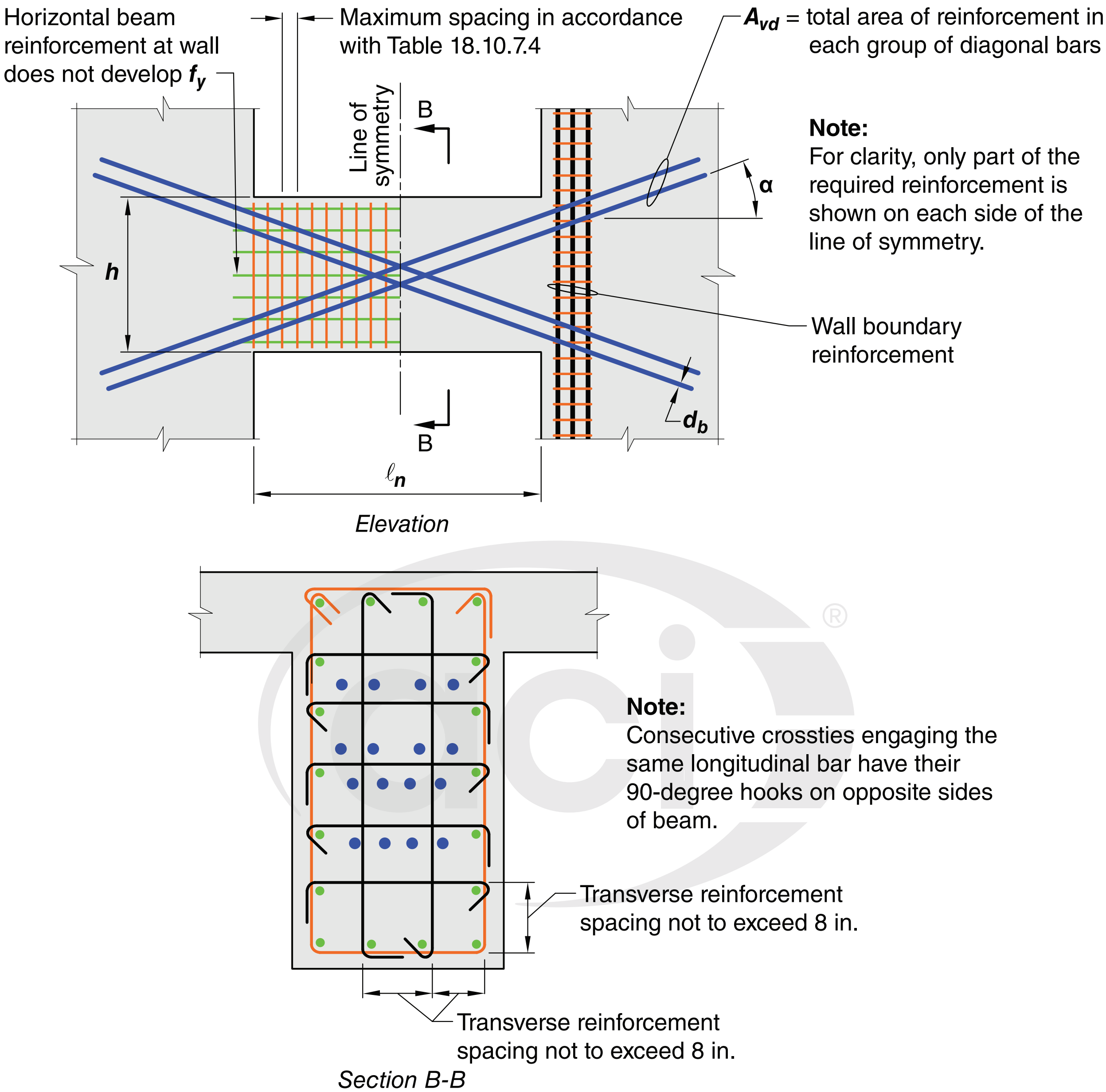

Fig. R18.10.7b—Full confinement of diagonally reinforced concrete beam section in coupling beams with diagonally oriented reinforcement. Wall boundary reinforcement shown on one side only for clarity.

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

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

CODE

integrity of nonstructural components and their connections to the structure.

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

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

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

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

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

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

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

(a) V_n shall be calculated by

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

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

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

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

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

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

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

COMMENTARY

CODE

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

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

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

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

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

Table 18.10.7.4—Maximum spacing of transverse reinforcement in coupling beams

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

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

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

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

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

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

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

COMMENTARY

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

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

CODE

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

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

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

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

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

COMMENTARY

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

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

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

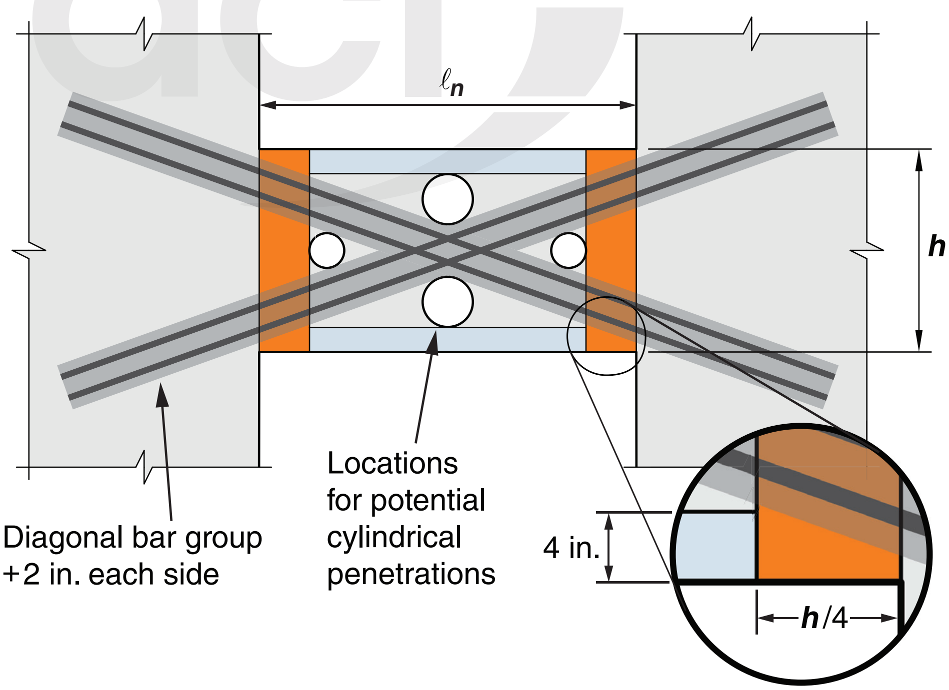

Fig. R18.10.7.6—Coupling beam elevation showing prohibited penetration regions shaded. Note that four potential locations for penetrations are shown although only two are permitted in any one beam.

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18.10.8 *Wall piers*

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R18.10.8 *Wall piers*

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

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

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

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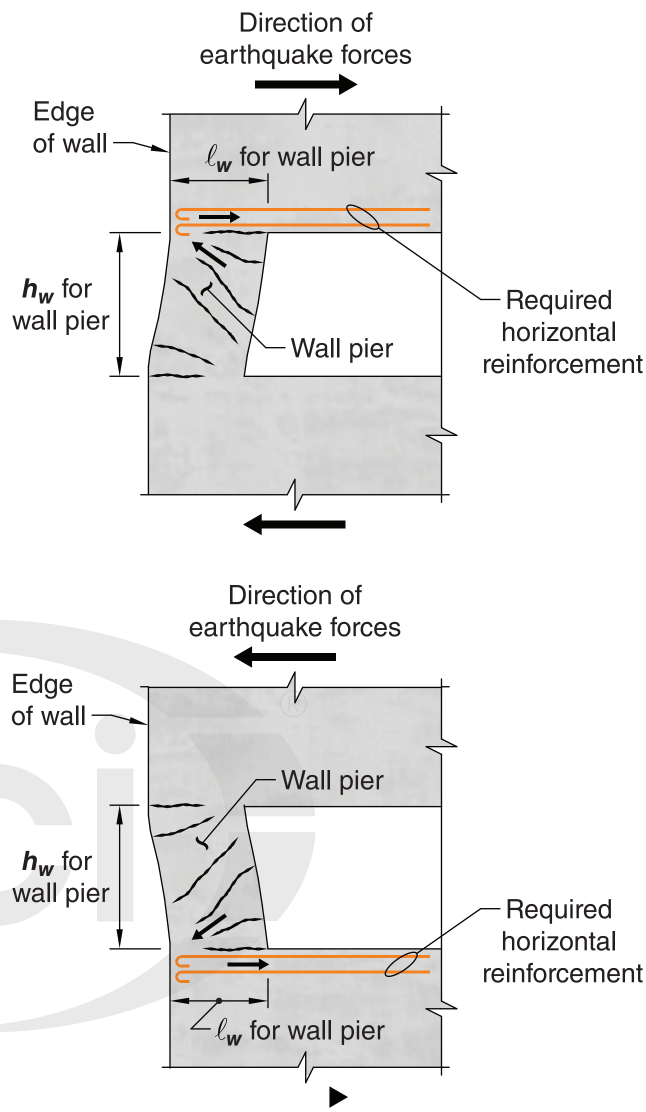

Fig. R18.10.8—Required horizontal reinforcement in wall segments above and below wall piers at the edge of a wall.

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

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

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

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

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

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

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

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

18.10.9 *Ductile coupled walls*

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

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

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

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

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

COMMENTARY

R18.10.9 *Ductile coupled walls*

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

CODE

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

18.10.10 *Construction joints*

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

18.10.11 *Discontinuous walls*

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

18.11—Special structural walls constructed using precast concrete

18.11.1 *Scope*

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

18.11.2 *General*

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

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

18.12—Diaphragms and trusses

18.12.1 *Scope*

COMMENTARY

R18.11—Special structural walls constructed using precast concrete

R18.11.2 *General*

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

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

R18.12—Diaphragms and trusses

R18.12.1 *Scope*

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

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

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

CODE

COMMENTARY

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

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

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

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

18.12.2 *Design forces*

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

R18.12.2 *Design forces*

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

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

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

CODE

COMMENTARY

design professional should consider providing increased diaphragm strength.

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

18.12.3 *Seismic load path*

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

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

R18.12.3 *Seismic load path*

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

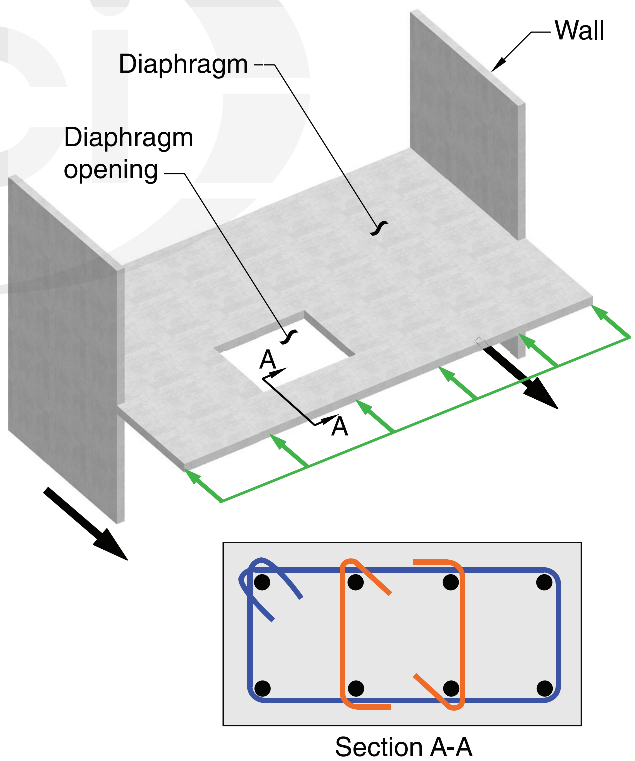

Fig. R18.12.3.2—Example of diaphragm subject to the requirements of 18.12.3.2 and showing an element having confinement as required by 18.12.7.6.

CODE

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

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

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

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

18.12.6 *Minimum thickness of diaphragms*

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

18.12.7 *Reinforcement*

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

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

COMMENTARY

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

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

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

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

R18.12.6 *Minimum thickness of diaphragms*

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

R18.12.7 *Reinforcement*

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

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

CODE

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

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

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

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

(b) Length between two vertical elements

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

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

Table 18.12.7.6—Transverse reinforcement for collector elements

<table>
<tr><th>Transverse reinforcement</th><th colspan="2">Applicable expressions</th><th></th></tr>
<tr><td>A_{sh}/sb_c for rectilinear hoop</td><td colspan="2">$0.9\dfrac{f_c'}{f_{yt}}$</td><td>(a)</td></tr>
<tr><td rowspan="2">ρ_s for spiral or circular hoop</td><td rowspan="2">Greater of:</td><td>$0.45\left(\dfrac{A_g}{A_{ch}}-1\right)\dfrac{f_c'}{f_{yt}}$</td><td>(b)</td></tr>
<tr><td>$0.12\dfrac{f_c'}{f_{yt}}$</td><td>(c)</td></tr>
</table>

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

COMMENTARY

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

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

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

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

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

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

CODE

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

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

18.12.8 *Combined moment and axial strength*

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

18.12.9 *Shear strength*

COMMENTARY

R18.12.8 *Combined moment and axial strength*

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

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

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

R18.12.9 *Shear strength*

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

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

CODE

COMMENTARY

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

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

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

18.12.9.1 V_n of diaphragms shall not exceed:

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

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

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

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

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

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

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

CODE

18.12.10 *Construction joints*

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

18.12.11 *Precast concrete diaphragms*

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

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

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

18.12.12 *Structural trusses*

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

COMMENTARY

R18.12.11 *Precast concrete diaphragms*

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

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

R18.12.12 *Structural trusses*

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

CODE

Table 18.12.12.1—Transverse reinforcement for structural trusses

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

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

18.13—Foundations

18.13.1 *Scope*

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

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

18.13.2 *Footings, foundation mats, and pile caps*

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

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

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

COMMENTARY

R18.13—Foundations

R18.13.1 *Scope*

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

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

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

CODE

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

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

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

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

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

18.13.3 *Grade beams and slabs-on-ground*

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

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

18.13.4 *Foundation seismic ties*

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

COMMENTARY

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

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

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

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

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

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

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

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

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

R18.13.4 *Foundation seismic ties*

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

CODE

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

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

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

(a) Reinforced concrete beams within the slab-on-ground

(b) Reinforced concrete slabs-on-ground

(c) Confinement by competent rock, hard cohesive soils, or very dense granular soils

(d) Other means approved by the building official

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

(a) The smallest cross-sectional dimension of the grade beam shall be at least equal to the clear spacing between connected columns divided by 20, but need not exceed 18 in.

(b) Closed tie transverse reinforcement shall be provided at a spacing not to exceed the lesser of 0.5 times the smallest orthogonal cross-sectional dimension and 12 in.

18.13.5 *Deep foundations*

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

(a) Uncased cast-in-place concrete drilled or augered piles

(b) Metal cased concrete piles

(c) Concrete filled pipe piles

(d) Precast concrete piles

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

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

COMMENTARY

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

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

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

R18.13.5 *Deep foundations*

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

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

CODE

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

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

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

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

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

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

COMMENTARY

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

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

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

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

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

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

CODE

COMMENTARY

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

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

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

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

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

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

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

18.13.5.8 *Metal-cased concrete piles*

R18.13.5.8 *Metal-cased concrete piles*

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

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

CODE

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

18.13.5.9 *Concrete-filled pipe piles*

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

18.13.5.10 *Precast concrete piles*

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

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

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

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

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

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

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

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

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

COMMENTARY

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

R18.13.5.9 *Concrete-filled pipe piles*

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

R18.13.5.10 *Precast concrete piles*

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

R18.13.5.10.4

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

CODE

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

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

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

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

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

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

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

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

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

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

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

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

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

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

COMMENTARY

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

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

CODE

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

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

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

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

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

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

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

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

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

18.13.6 *Anchorage of piles, piers, and caissons*

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

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

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

COMMENTARY

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

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

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

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

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

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

CODE

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

18.14.1 *Scope*

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

18.14.2 *Design actions*

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

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

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

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

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

COMMENTARY

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

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

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

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

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

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

CODE

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

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

(d) Joints shall satisfy Chapter 15.

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

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

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

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

(d) Joints shall satisfy 18.4.4.1.

18.14.4 *Precast beams and columns*

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

(a) Requirements of 18.14.3

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

(c) Structural integrity reinforcement, in accordance with 4.10

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

18.14.5 *Slab-column connections*

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

COMMENTARY

R18.14.4 *Precast beams and columns*

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

R18.14.5 *Slab-column connections*

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

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

CODE

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

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

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

COMMENTARY

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

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

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

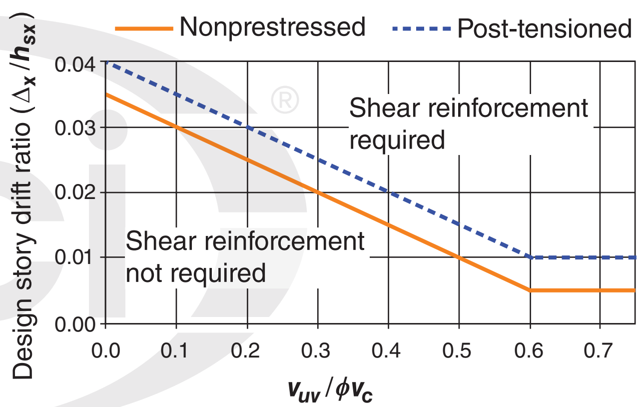

Fig. R18.14.5.1—Illustration of the criteria of 18.14.5.1.

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

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

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

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

18.14.6 *Wall piers*

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

R18.14.6 *Wall piers*

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