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

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

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

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

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

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

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

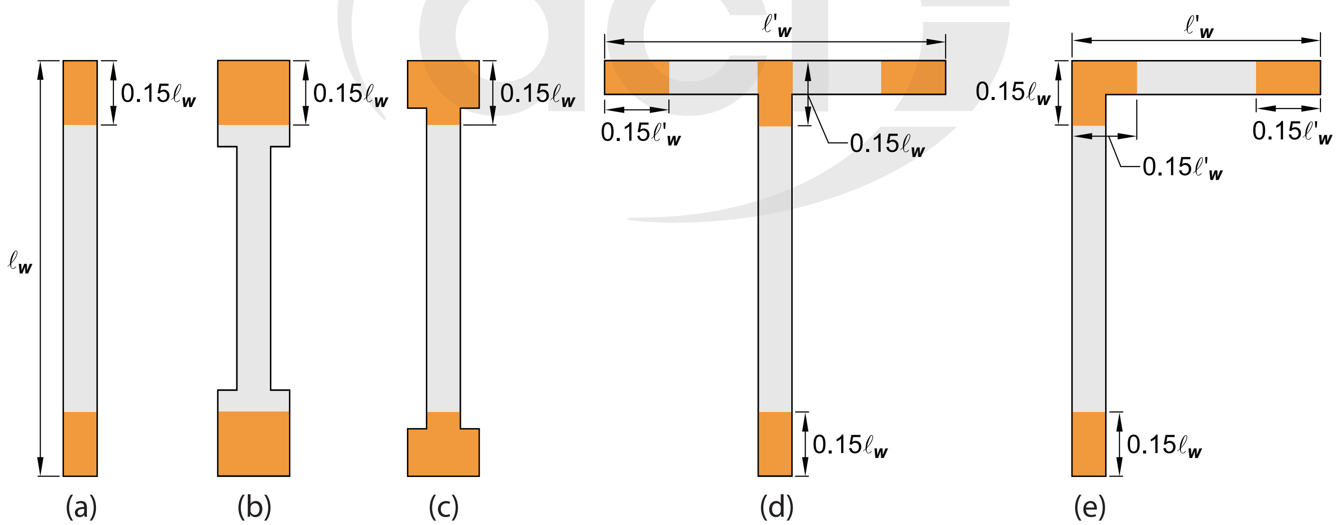

Fig. R18.10.2.4—Locations of longitudinal reinforcement required by 18.10.2.4(a) in different configurations of wall sections.

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

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

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

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18.10.3 *Design forces*

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

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

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

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R18.10.3 *Design forces*

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

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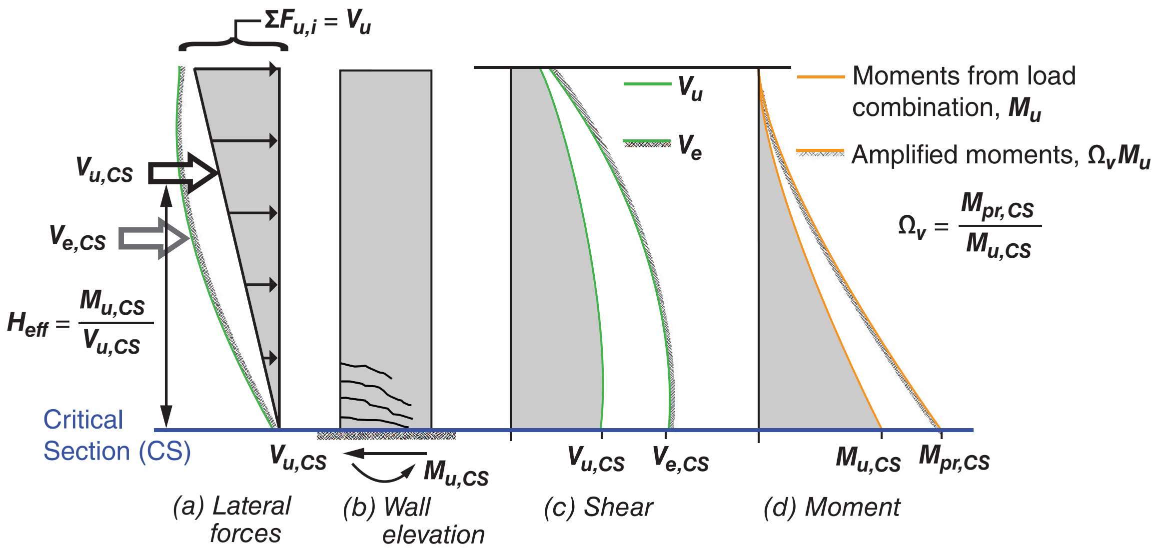

Fig. R18.10.3.3—Determination of shear demand for walls with $h_w/\ell_w \geq 2.0$ *(Moehle et al. 2011).*

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

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

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

Table 18.10.3.3.3—Factors Ω_v and ω_v

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

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

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

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

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

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

18.10.4 *Shear strength*

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

R18.10.4 *Shear strength*

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

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

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

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

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

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

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

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

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18.10.4.1 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 $h_w/\ell_w \le 1.5$

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

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

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

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

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

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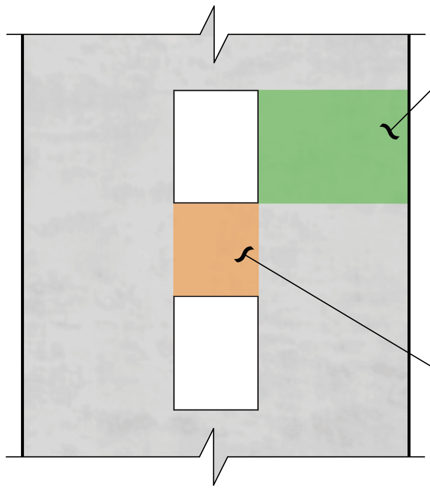

Fig. R18.10.4.5—Wall with openings.

18.10.5 *Design for flexure and axial force*

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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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

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

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).

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

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

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

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

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

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

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

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

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

(k) Horizontal reinforcement in the wall web shall extend to within 6 in. of the end of the wall. Reinforcement shall develop f_y in tension at the face of the confined core of the boundary element using standard hooks or heads. Where the confined boundary element has sufficient length to develop the horizontal web reinforcement, and A_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.

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

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

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

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

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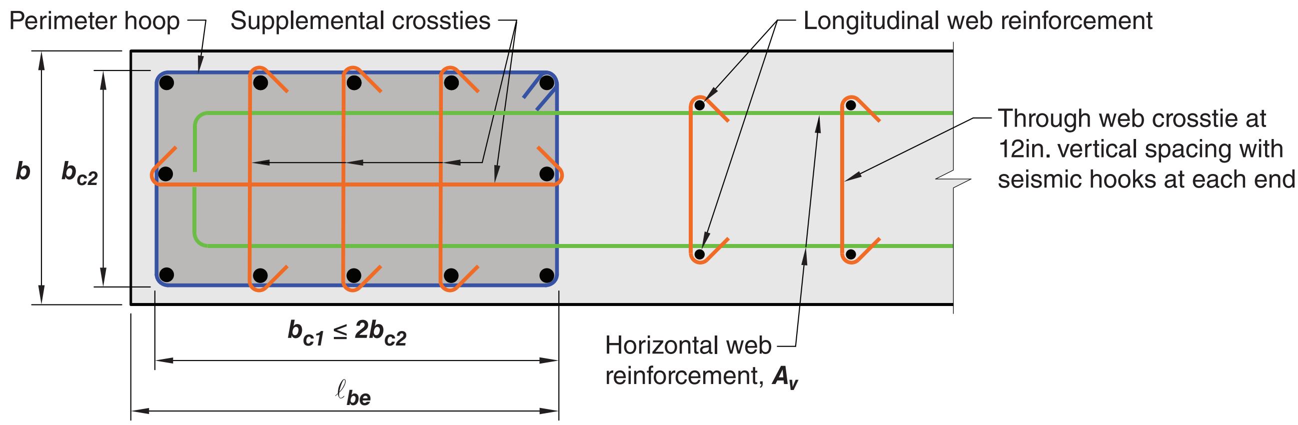
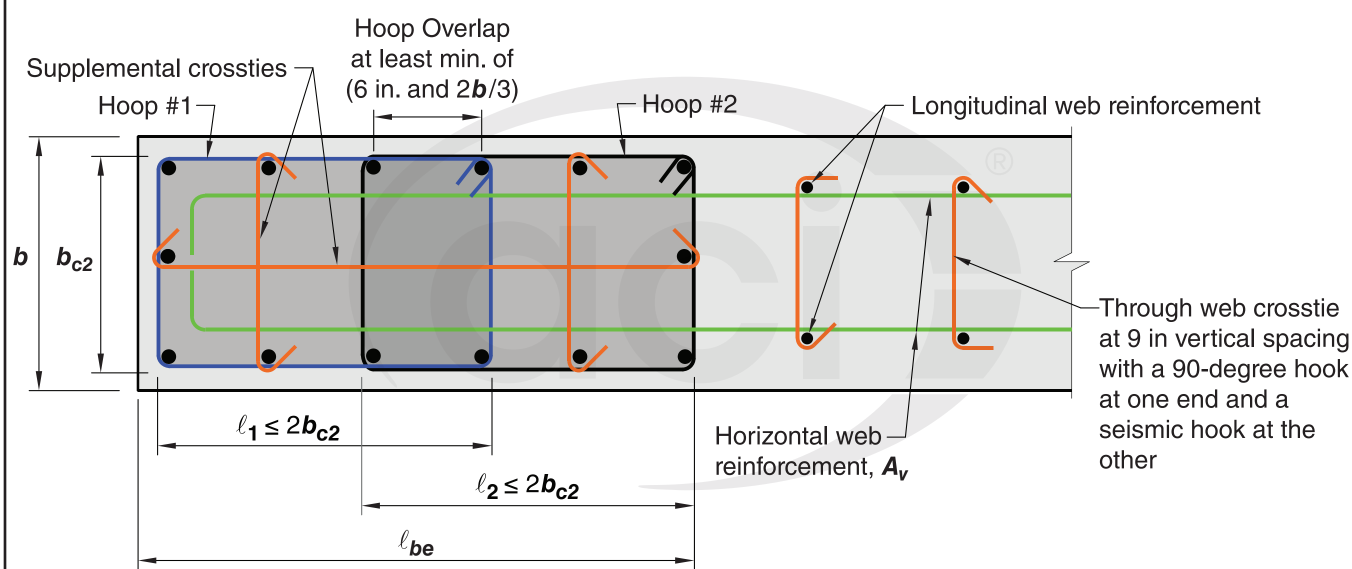


(a) Perimeter hoop with supplemental 135-degree crossties and 135-degree crossties supporting distributed web longitudinal reinforcement



(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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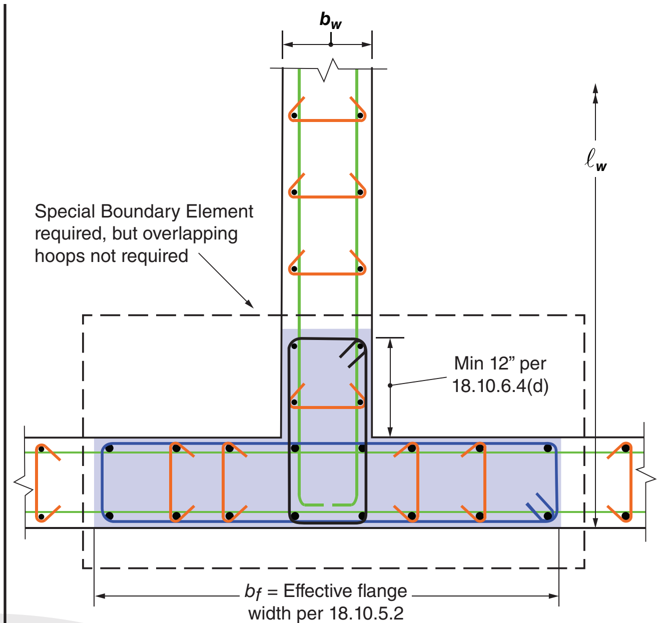

Fig. R18.10.6.4b—Example configuration for a case if a special boundary element is required for a flanged wall

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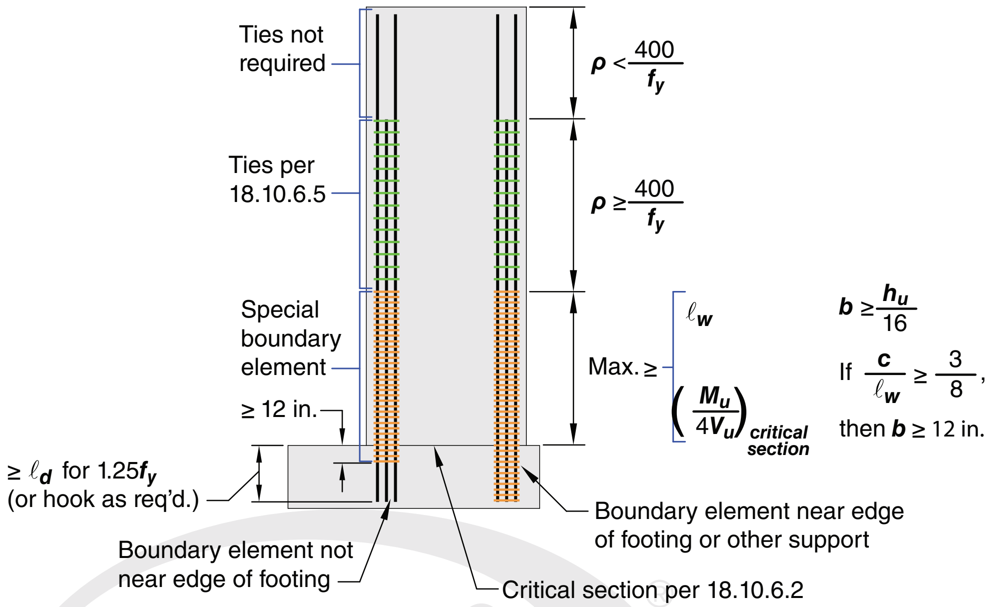
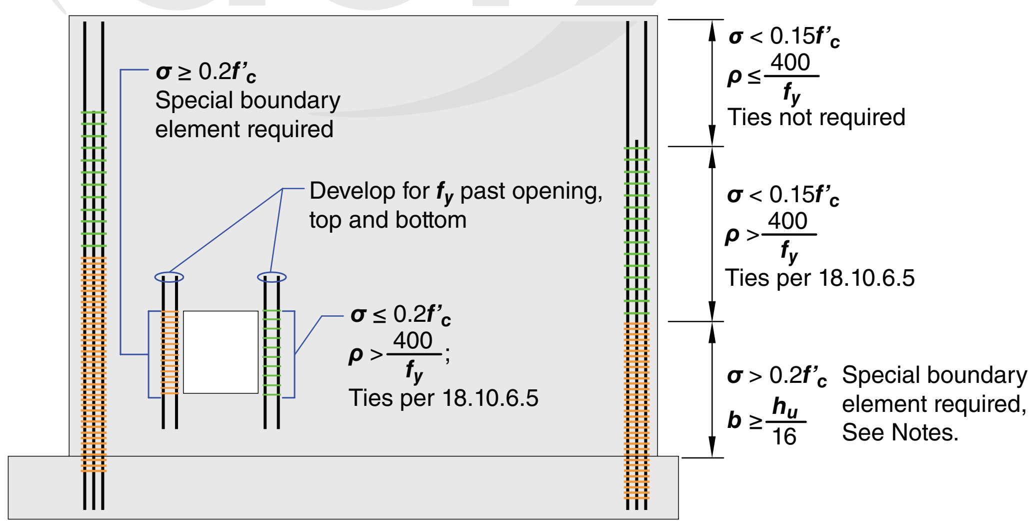


(a) Wall with $h_w/\ell_w \geq 2.0$ and a single critical section controlled by flexure and axial load designed using 18.10.6.2, 18.10.6.4, and 18.10.6.5



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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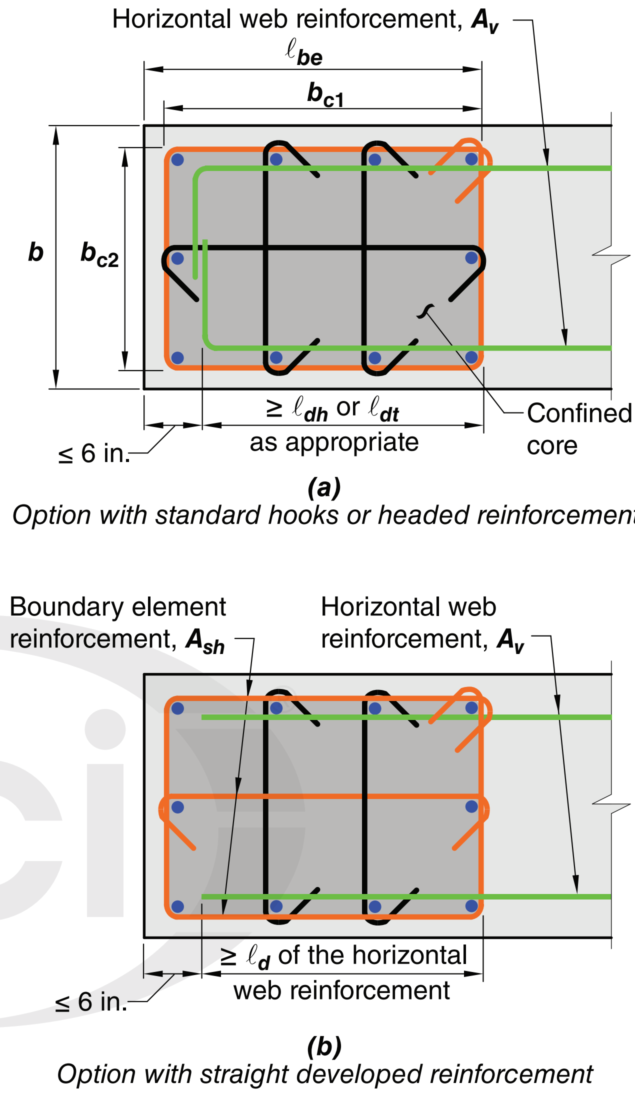

Fig. R18.10.6.4d—Development of wall horizontal reinforcement in confined boundary element.

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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.

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

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

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

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

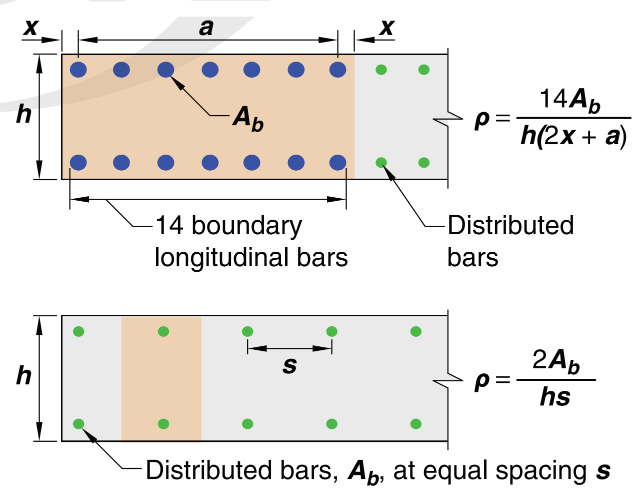

Fig. R18.10.6.5a—Longitudinal reinforcement ratios for typical wall boundary conditions.

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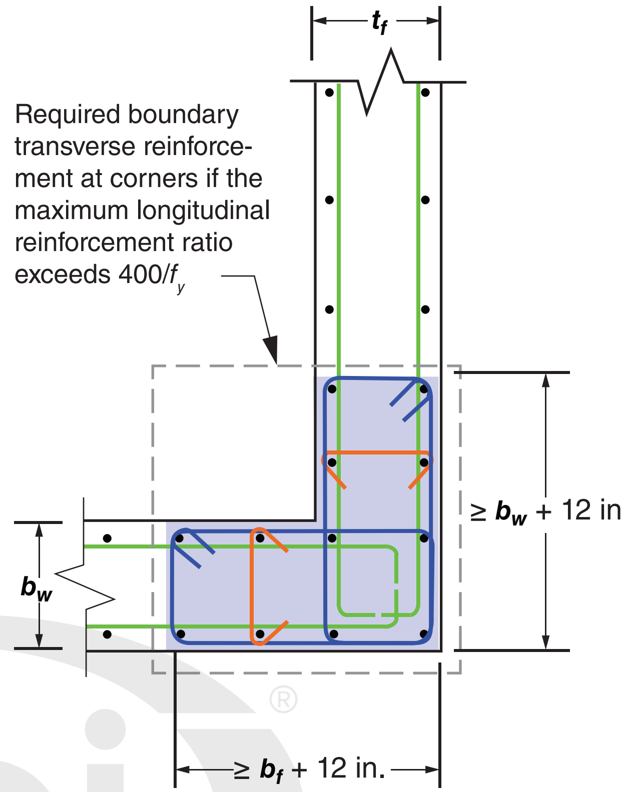

Fig. R18.10.6.5b—Transverse reinforcement at the intersection of a wall web and flange.

18.10.7 *Coupling beams*

R18.10.7 *Coupling beams*

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

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

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

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

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

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

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

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

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

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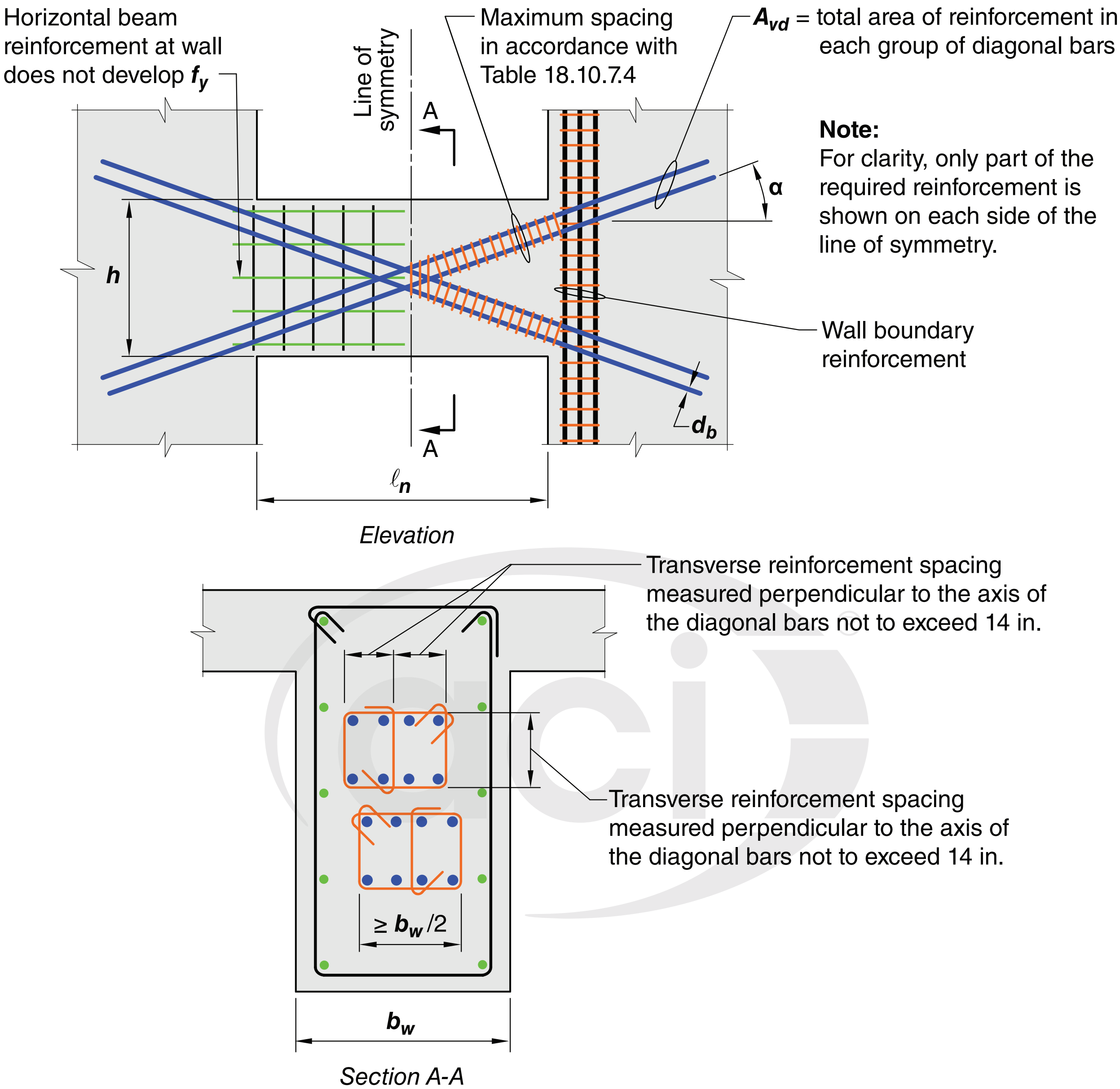

Fig. R18.10.7a—Confinement of individual diagonals in coupling beams with diagonally oriented reinforcement. Wall boundary reinforcement shown on one side only for clarity.

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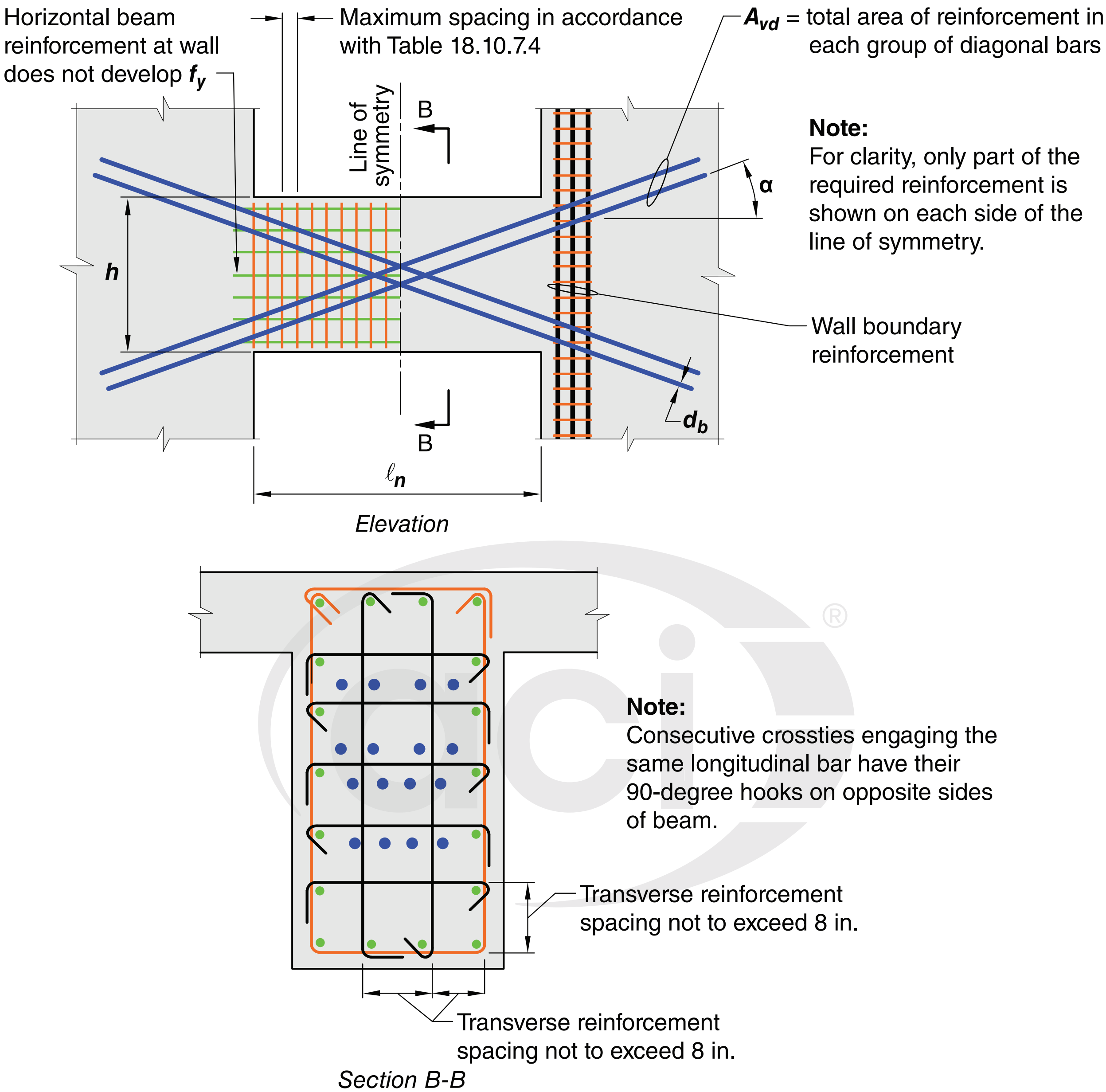

Fig. R18.10.7b—Full confinement of diagonally reinforced concrete beam section in coupling beams with diagonally oriented reinforcement. Wall boundary reinforcement shown on one side only for clarity.

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

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integrity of nonstructural components and their connections to the structure.

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

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

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

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

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

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

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

(a) V_n shall be calculated by

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

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

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

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

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

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

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

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tion, it is permitted to modify the arrangement of the transverse reinforcement provided the spacing and volume ratio requirements are satisfied. Additional longitudinal and transverse reinforcement shall be distributed around the beam perimeter with total area in each direction of at least $0.002b_w s$ 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.

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

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

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

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

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

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

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

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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.

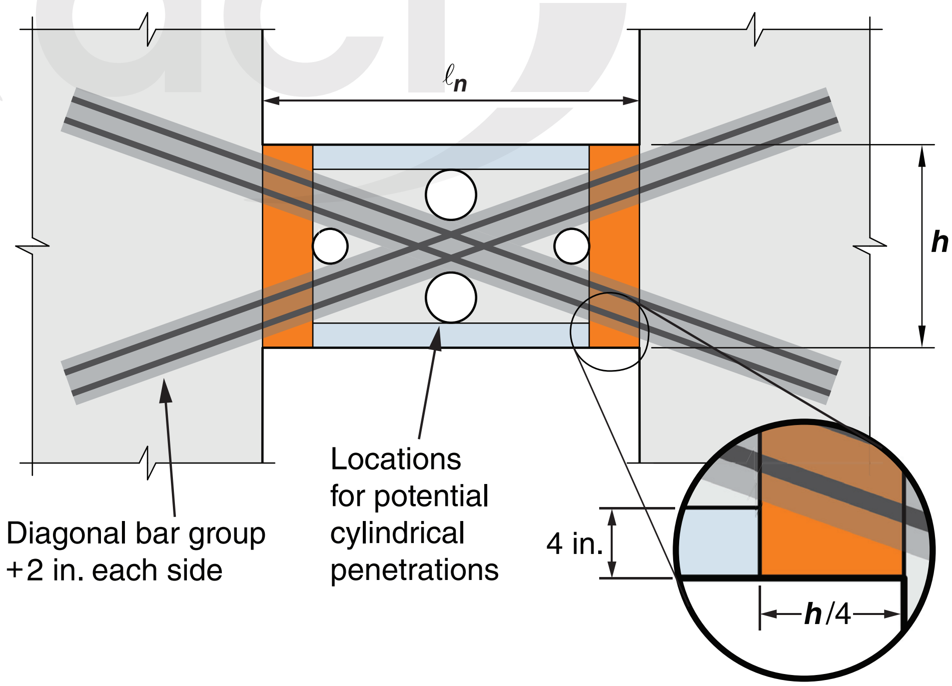

Fig. R18.10.7.6—Coupling beam elevation showing prohibited penetration regions shaded. Note that four potential locations for penetrations are shown although only two are permitted in any one beam.

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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).