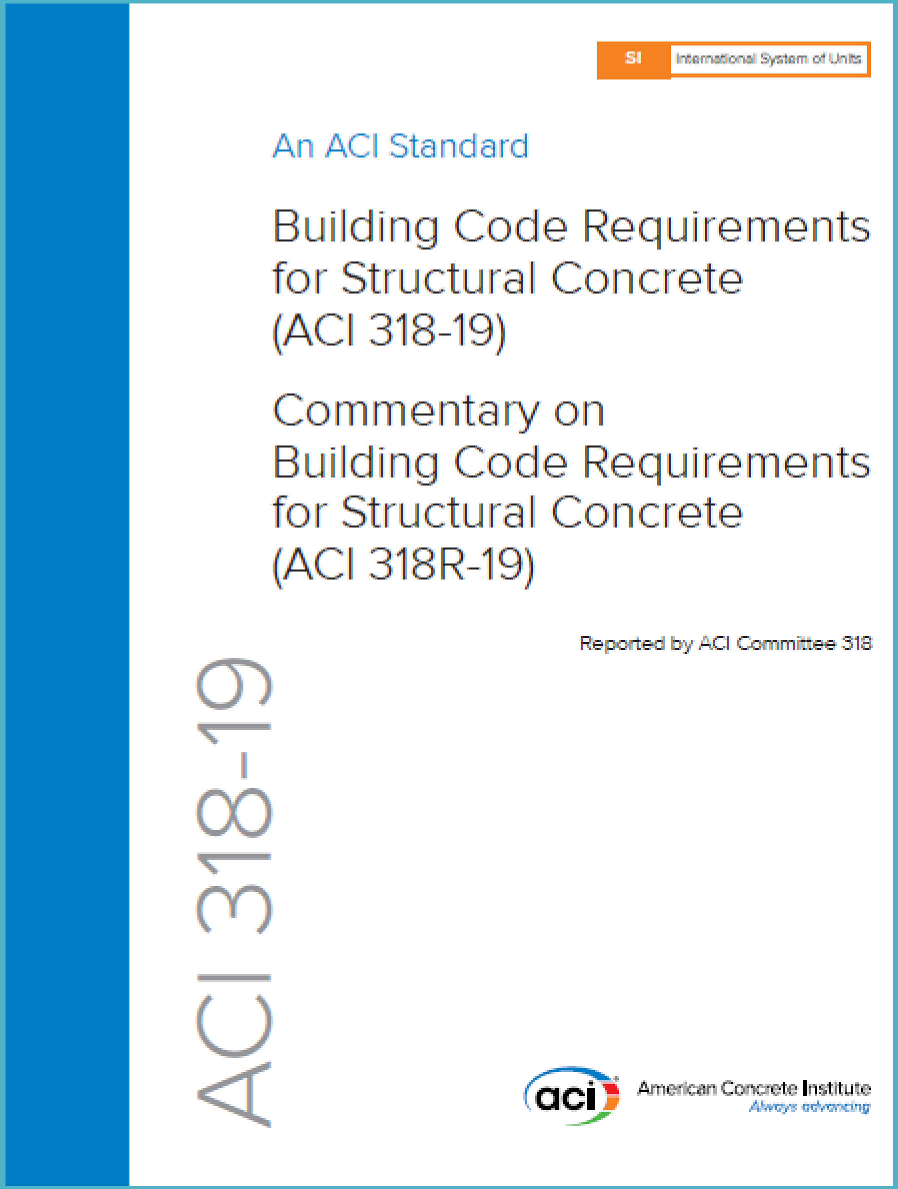

موسسه دیباگران نوید پارس

گروه آموزشی نوید

ACI318-19

مرور تغییرات و نکات مهم

محمد بنان

هیات علمی و مدیرگروه بخش مهندسی عمران دانشگاه آپادانا

دانشجوی دکتری سازه

Webinar

1399/01/26

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ACI 318-19: BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE 5

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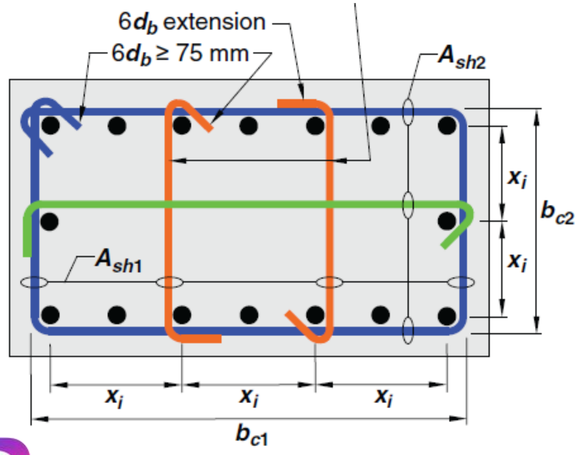

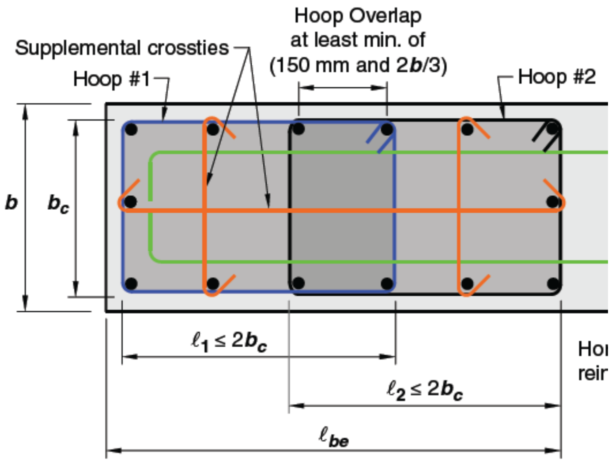

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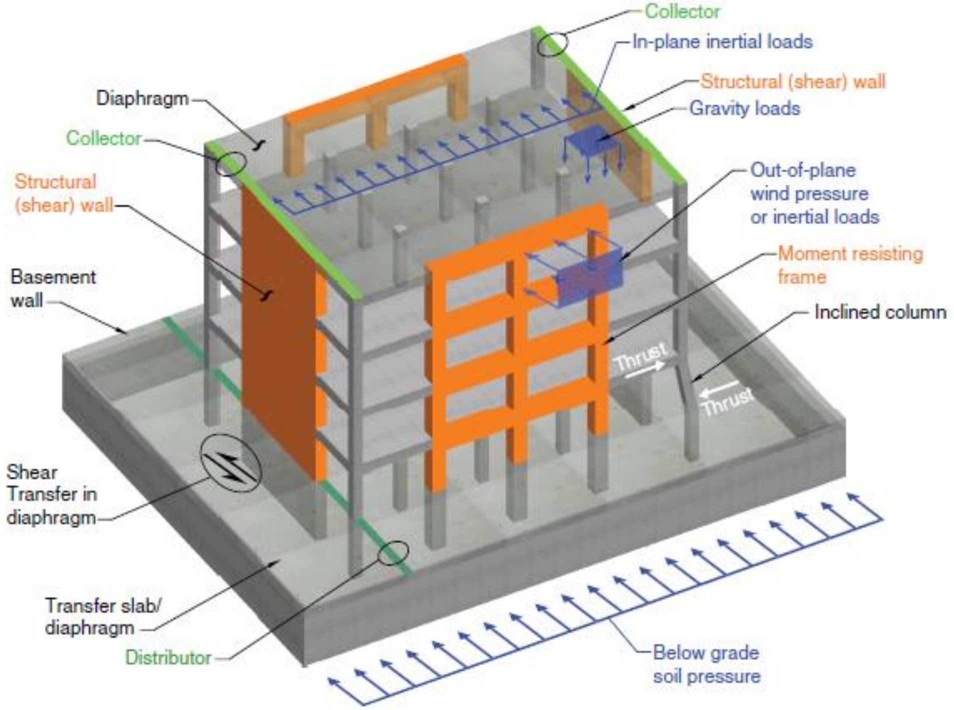
CODE COMMENTARY



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ضریب ترک خورگی اعضا

Table 6.6.3.1.1(a)—Moments of inertia and cross-sectional areas permitted for elastic analysis at factored load level

Member and condition		Moment of inertia	Cross-sectional area for axial deformations	Cross-sectional area for shear deformations
Columns		$0.70I_g$	$1.0A_g$	$b_w h$
Walls	Uncracked	$0.70I_g$		
	Cracked	$0.35I_g$		
Beams		$0.35I_g$		
Flat plates and flat slabs		$0.25I_g$		

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Table 6.6.3.1.1(a)—Moment of inertia and cross-sectional area permitted for elastic analysis at factored load level

Member and condition		Moment of Inertia	Cross-sectional area
Columns		$0.70I_g$	$1.0A_g$
Walls	Uncracked	$0.70I_g$	
	Cracked	$0.35I_g$	
Beams		$0.35I_g$	
Flat plates and flat slabs		$0.25I_g$	

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7.6—Reinforcement limits

7.6.1 *Minimum flexural reinforcement in nonprestressed slabs*

7.6.1.1 A minimum area of flexural reinforcement, $A_{s,min}$, of $\mathbf{0.0018}A_g$ shall be provided.

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7.6.1.1 A minimum area of flexural reinforcement, $A_{s,min}$, shall be provided in accordance with Table 7.6.1.1.

Table 7.6.1.1—$A_{s,min}$ for nonprestressed one-way slabs

Reinforcement type	f_y, MPa	$A_{s,min}$	
Deformed bars	< 420	$0.0020A_g$	
Deformed bars or welded wire reinforcement	≥ 420	Greater of:	$\frac{0.0018\times 420}{f_y}A_g$
			$0.0014A_g$

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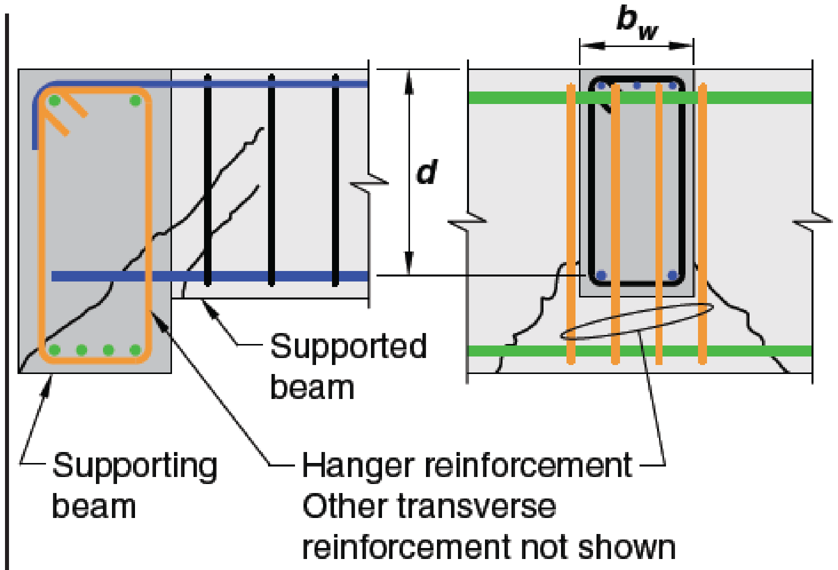

Fig. R9.7.6.2.1—Hanger reinforcement for shear transfer.

Table 9.7.6.2.2—Maximum spacing of legs of shear reinforcement

Required V_s		Maximum s, mm			
		Nonprestressed beam		Prestressed beam	
		Along length	Across width	Along length	Across width
$\leq 0.33\sqrt{f'_c}b_wd$	Lesser of:	$d/2$	d	$3h/4$	$3h/2$
		600			
$> 0.33\sqrt{f'_c}b_wd$	Lesser of:	$d/4$	$d/2$	$3h/8$	$3h/4$
		300			

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Table 9.7.6.2.2—Maximum spacing of shear reinforcement

V_s		Maximum s, mm	
		Nonprestressed beam	Prestressed beam
$\leq 0.33\sqrt{f'_c}b_wd$	Lesser of:	$d/2$	$3h/4$
		600	
$> 0.33\sqrt{f'_c}b_wd$	Lesser of:	$d/4$	$3h/8$
		300	

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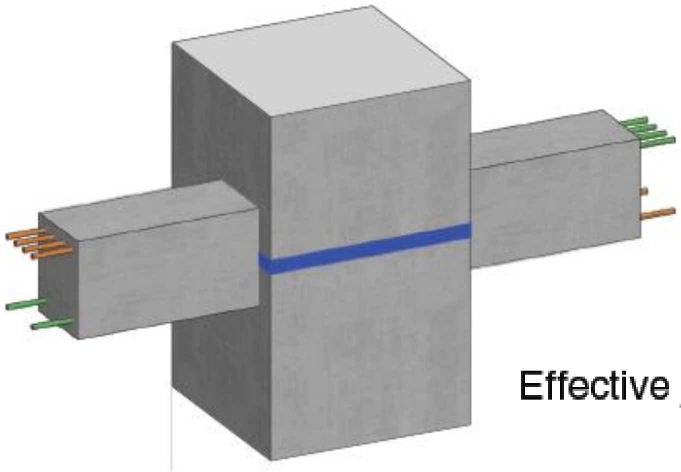

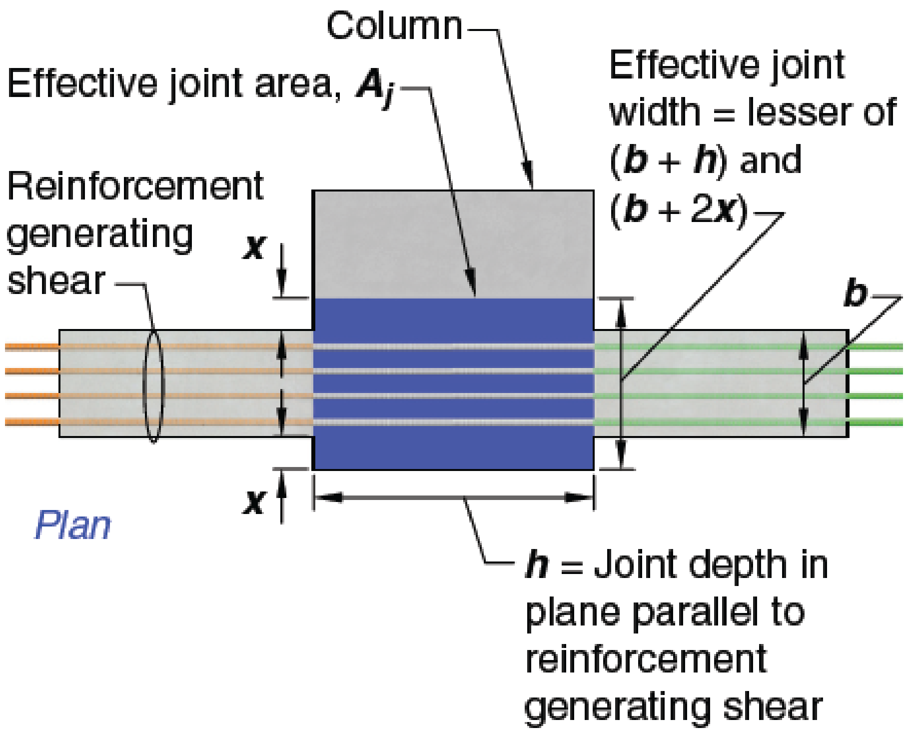

Note: Effective area of joint for forces in each direction of framing is to be considered separately.

Fig. R15.4.2—Effective joint area.

Table 15.4.2.3—Nominal joint shear strength V_n

Column	Beam in direction of V_u	Confinement by transverse beams according to 15.2.8	V_n, N[1]
Continuous or meets 15.2.6	Continuous or meets 15.2.7	Confined	$2.0\lambda\sqrt{f'_c}A_j$
		Not confined	$1.7\lambda\sqrt{f'_c}A_j$
	Other	Confined	$1.7\lambda\sqrt{f'_c}A_j$
		Not confined	$1.3\lambda\sqrt{f'_c}A_j$
Other	Continuous or meets 15.2.7	Confined	$1.7\lambda\sqrt{f'_c}A_j$
		Not confined	$1.3\lambda\sqrt{f'_c}A_j$
	Other	Confined	$1.3\lambda\sqrt{f'_c}A_j$
		Not confined	$1.0\lambda\sqrt{f'_c}A_j$

[1] λ shall be 0.75 for lightweight concrete and 1.0 for normalweight concrete.

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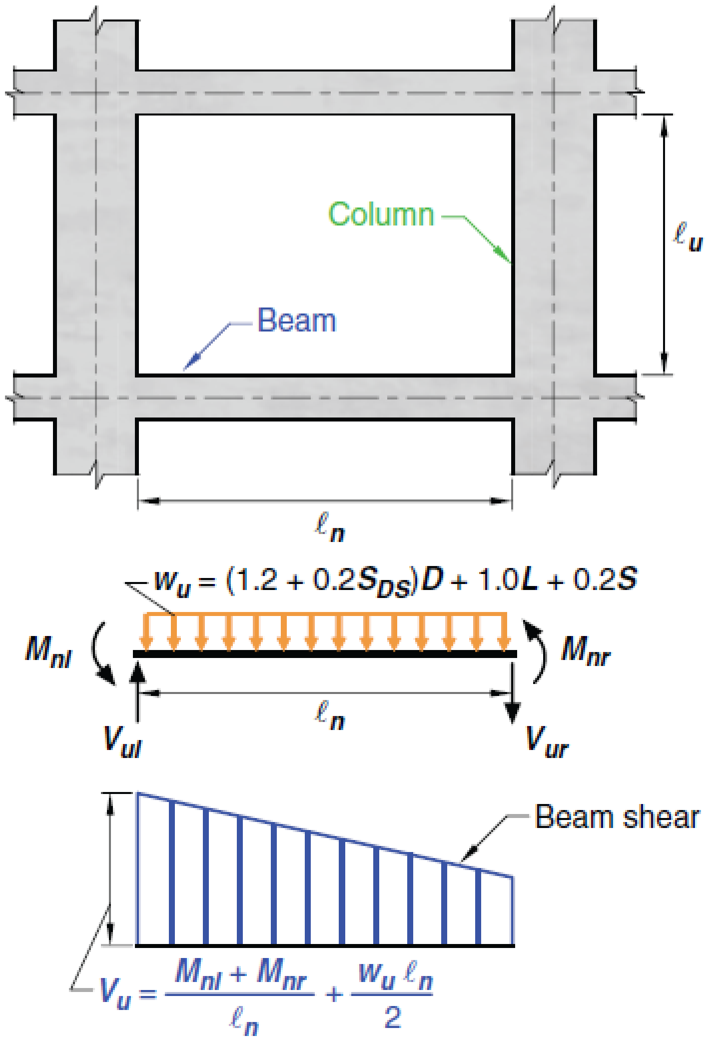

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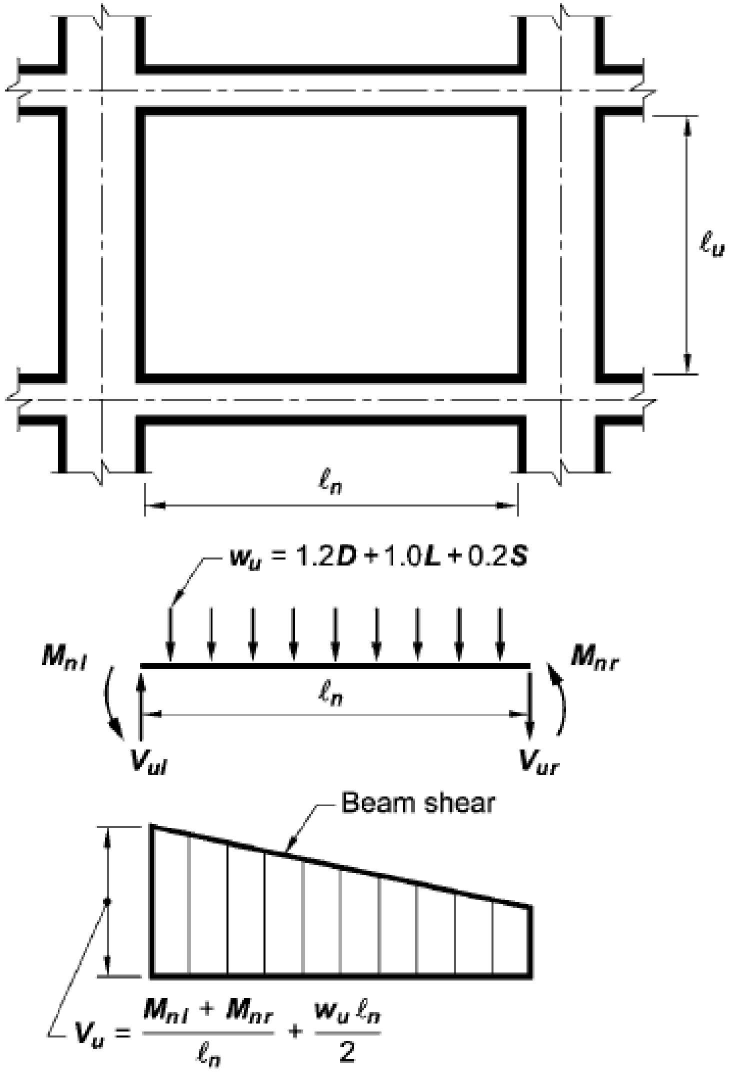

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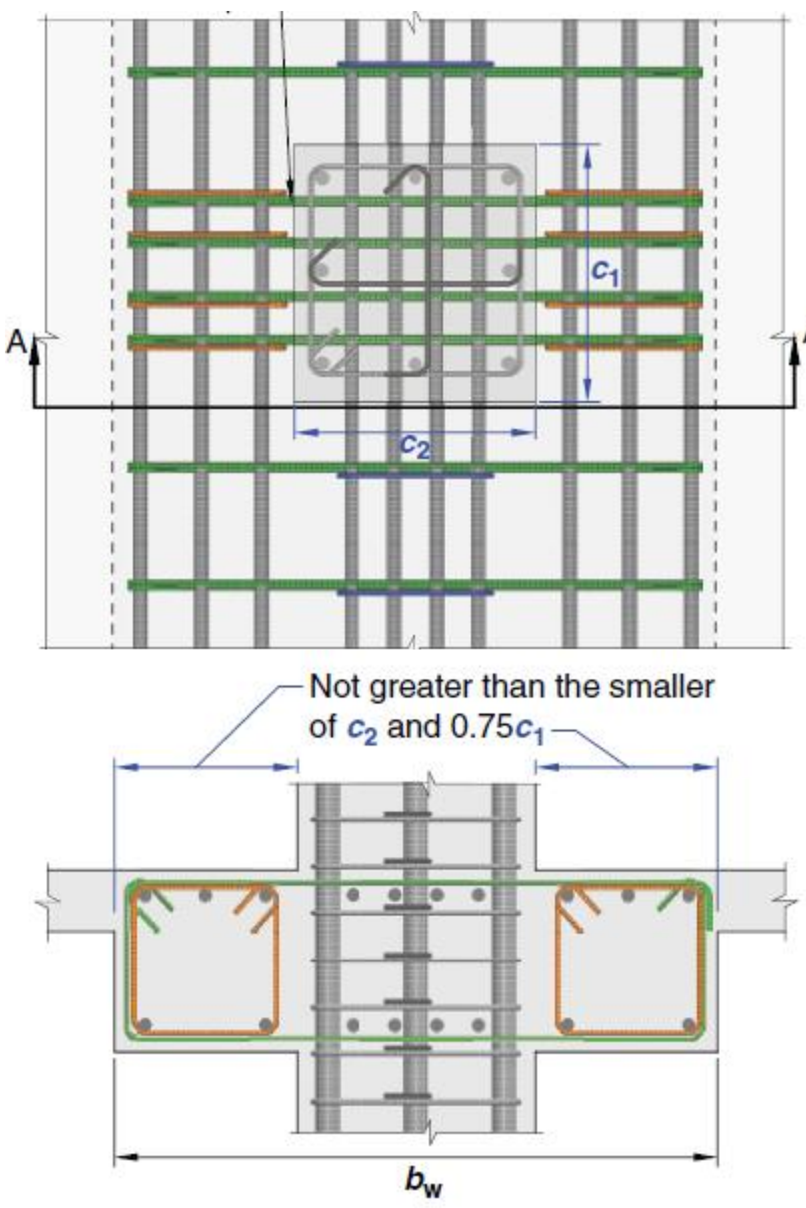

Section A-A

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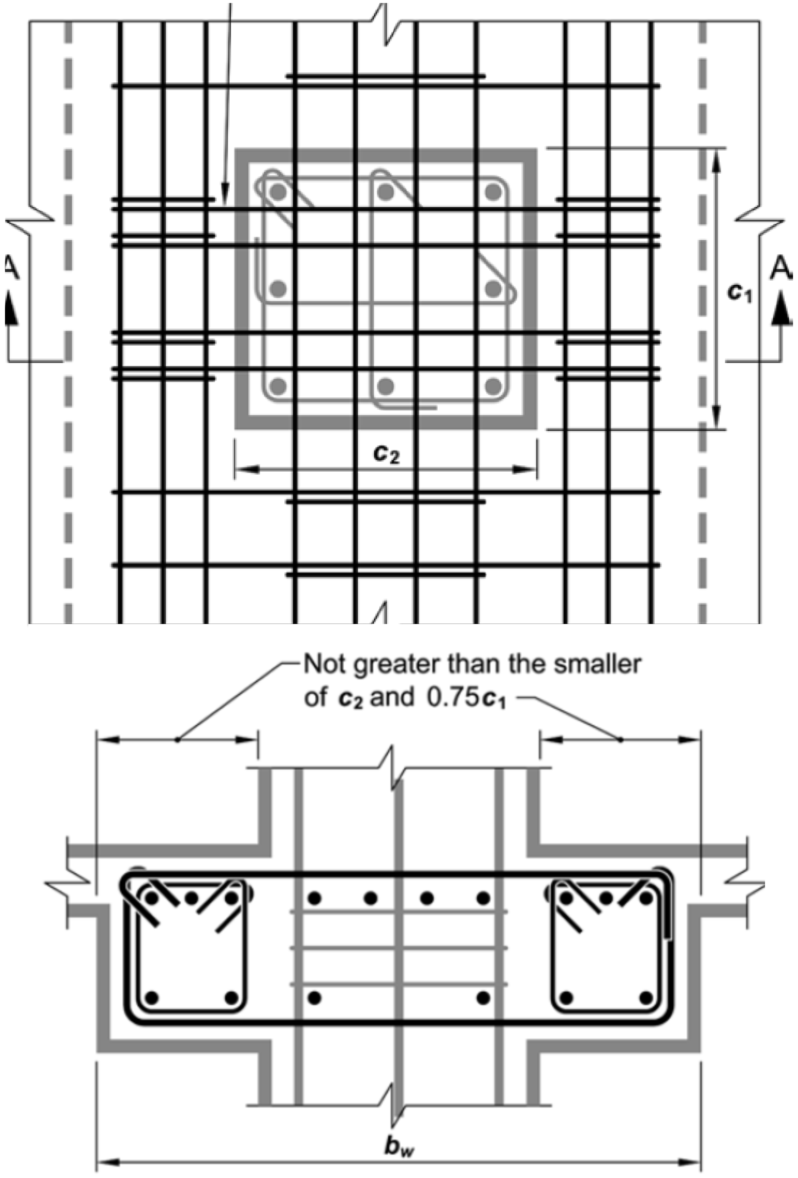

Note:
Transverse reinforcement in column above and below the joint not shown for clarity

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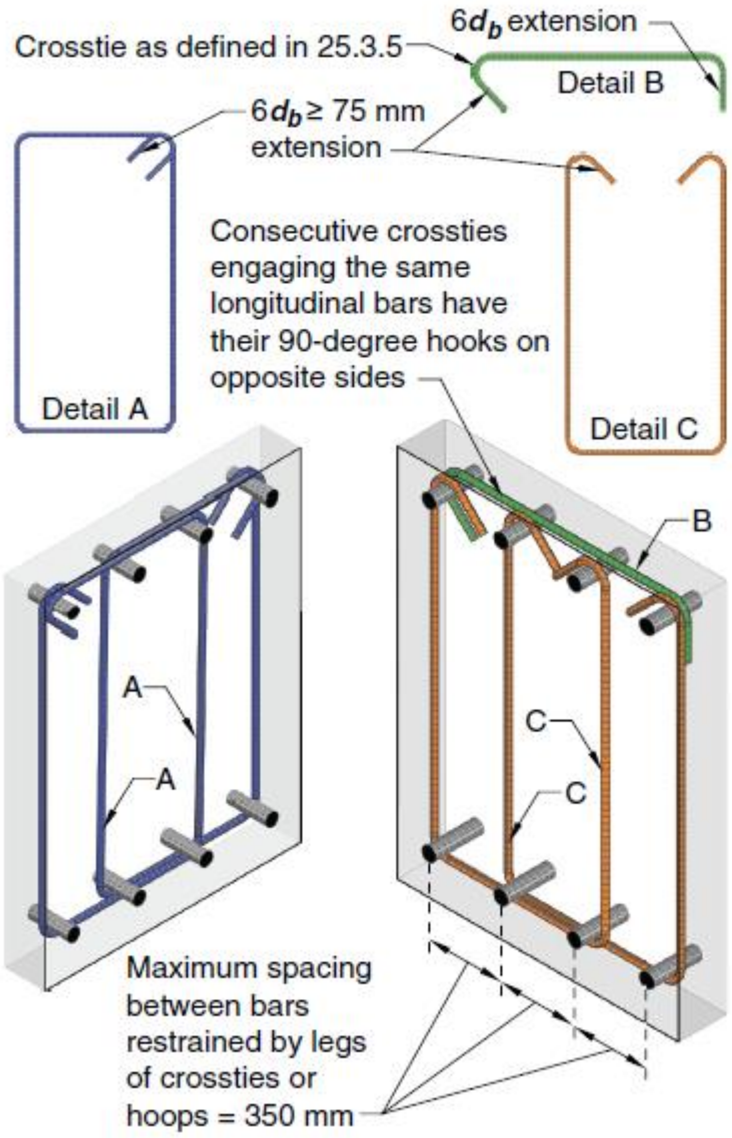

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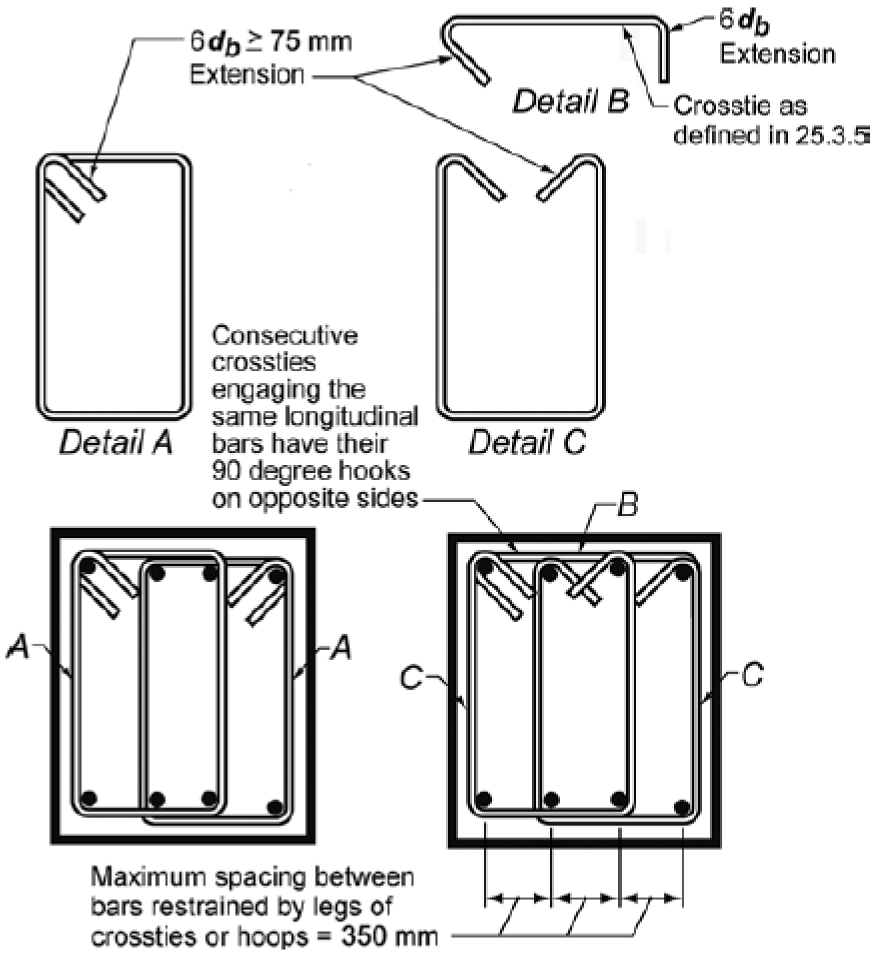

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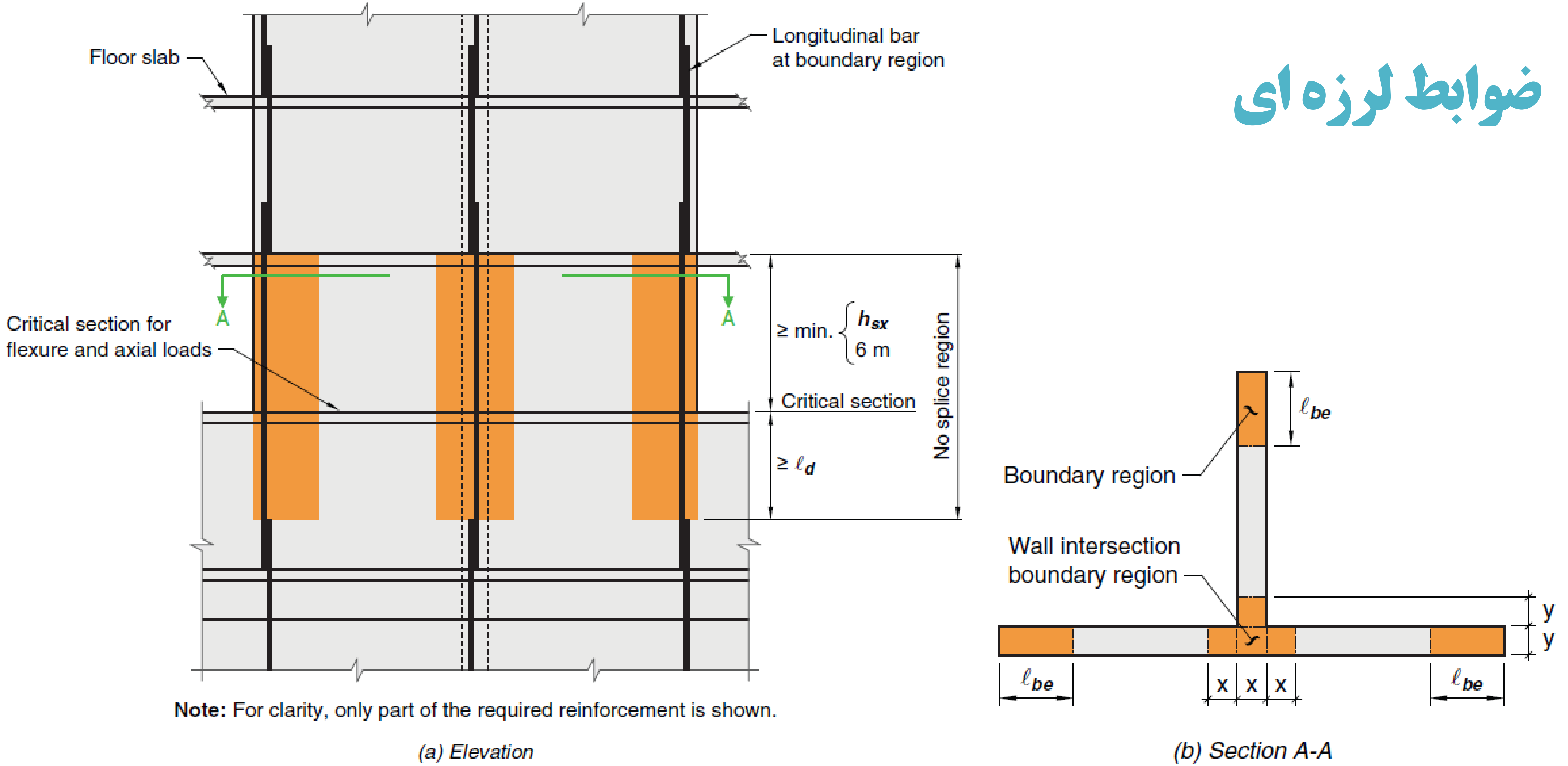

Fig. R18.10.2.3—Wall boundary regions within heights where lap splices are not permitted.

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18.10.2.4 Walls or wall piers with $h_w/\ell_w \geq 2.0$ that are effectively continuous from the base of structure to top of wall and are designed to have a single critical section for 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 $0.5\sqrt{f_c'}/f_y$.

(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 percent of the reinforcement required by 18.10.2.4(a) shall be terminated at any one section.

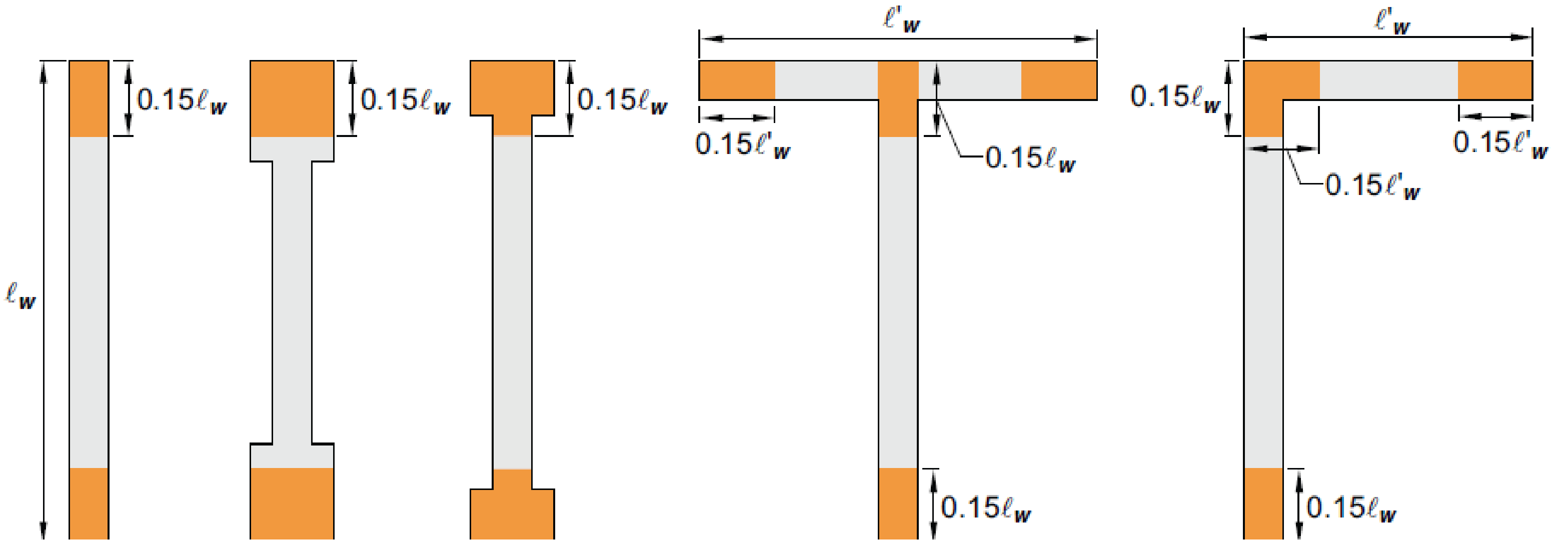

Fig. R18.10.2.4—Locations of longitudinal reinforcement required by 18.10.2.4(a) in different configurations of wall sections.

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18.10.3 *Design forces*

18.10.3.1 The design shear force V_e shall be calculated by:

$$V_e = \Omega_v \omega_v V_u \leq 3V_u \qquad (18.10.3.1)$$

where V_u, Ω_v, and ω_v are defined in 18.10.3.1.1, 18.10.3.1.2, and 18.10.3.1.3, respectively.

18.10.3.1.1 V_u is the shear force obtained from code lateral load analysis with factored load combinations.

18.10.3.1.2 Ω_v shall be in accordance with Table 18.10.3.1.2.

n_s = number of stories above the critical section

h_{wcs} = height of entire structural wall above the critical section for flexural and axial loads, mm

Table 18.10.3.1.2—Overstrength factor Ω_v at critical section

Condition	Ω_v	
$h_{wcs}/\ell_w > 1.5$	Greater of	M_{pr}/M_u[1]
		1.5[2]
$h_{wcs}/\ell_w \leq 1.5$	1.0	

[1] For the load combination producing the largest value of Ω_v.

[2] Unless a more detailed analysis demonstrated a smaller value, but not less than 1.0.

18.10.3.1.3 For walls with $h_{wcs}/\ell_w < 2.0$, ω_v shall be taken as 1.0. Otherwise, ω_v shall be calculated as:

$$\omega_v = 0.9 + \frac{n_s}{10} \qquad n_s \leq 6$$

$$\omega_v = 1.3 + \frac{n_s}{30} \leq 1.8 \qquad n_s > 6 \qquad (18.10.3.1.3)$$

where n_s shall not be taken less than the quantity $0.00028h_{wcs}$.

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.

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

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

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.

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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.

18.10.6.2 Walls or wall piers with $h_w/\ell_w \geq 2.0$ that are effectively continuous from the base of structure to top of wall and are designed to have a single critical section for flexure and axial loads shall satisfy (a) and (b) or shall be designed by 18.10.6.3:

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

$$c \geq \frac{\ell_w}{600(1.5\delta_u/h_w)} \quad (18.10.6.2)$$

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_w shall not be taken less than 0.005.

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(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 a least the greater of ℓ_w and $M_u/4V_u$, except as permitted in 18.10.6.4(i).

(ii) $b \geq \sqrt{0.025c\ell_w}$

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

$$\frac{\delta_c}{h_{wcs}} = \frac{1}{100}\left(4 - \frac{1}{50}\left(\frac{\ell_w}{b}\right)\left(\frac{c}{b}\right) - \frac{V_e}{0.66\sqrt{f_c'}A_{cv}}\right) \quad (18.10.6.2b)$$

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

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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_w shall not be taken less than 0.005.

(b) Where special boundary elements are required by (a), the 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(g).

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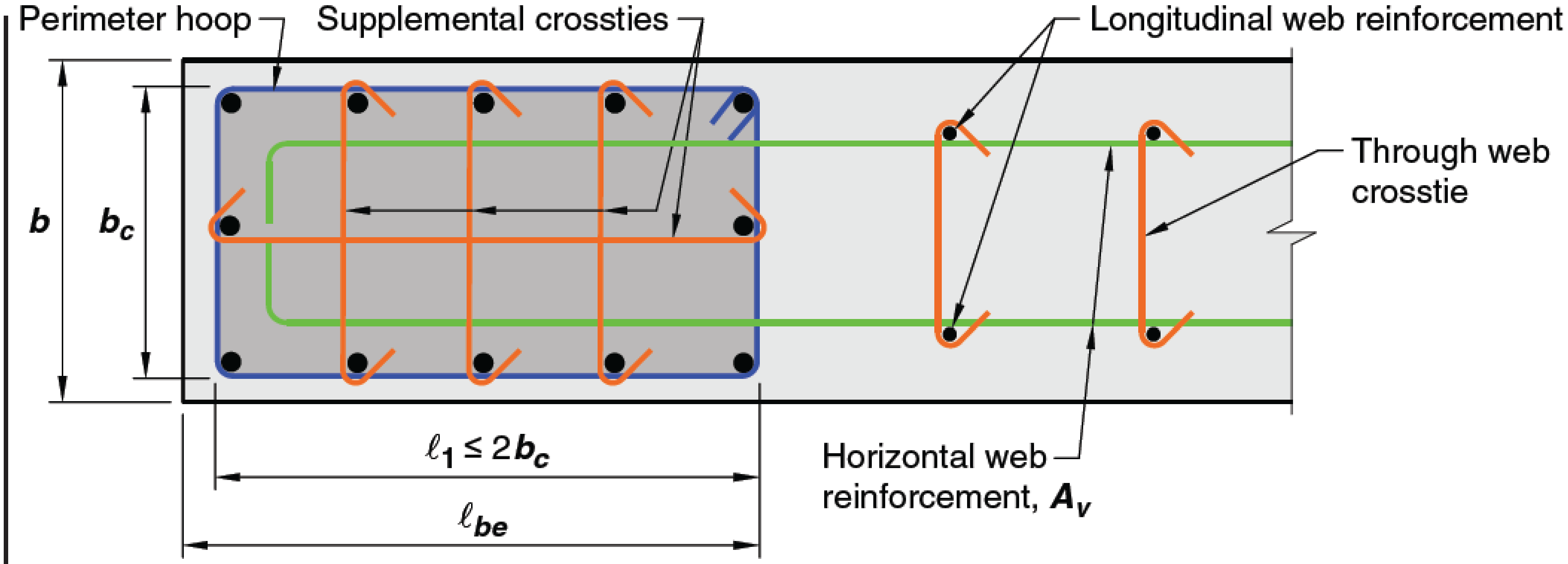

(a) *Perimeter hoop with supplemental 135-degree crossties and 135-degree crossties supporting distributed web longitudinal reinforcement*

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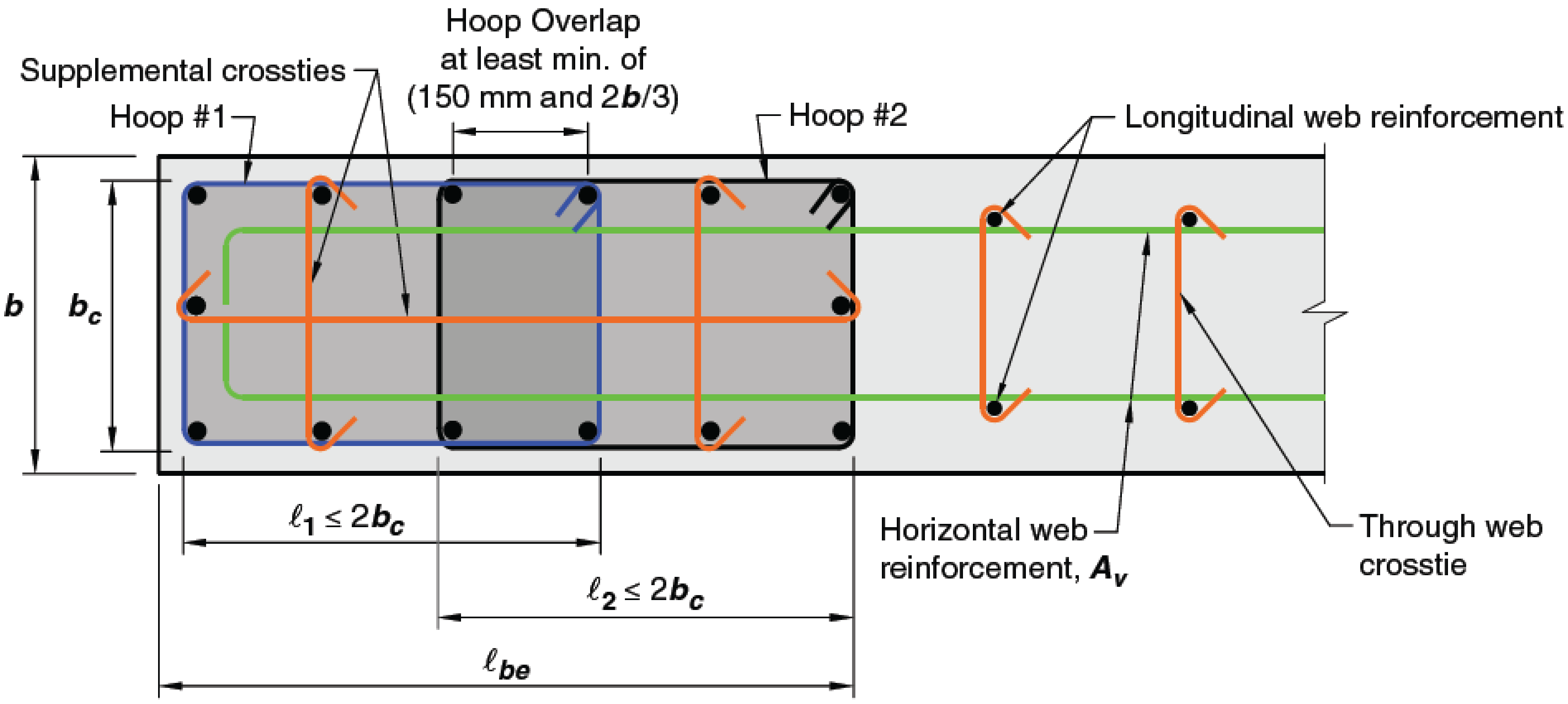

(b) Overlapping hoops with supplemental 135-degree crossties and 135-degree crossties supporting distributed web longitudinal reinforcement

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

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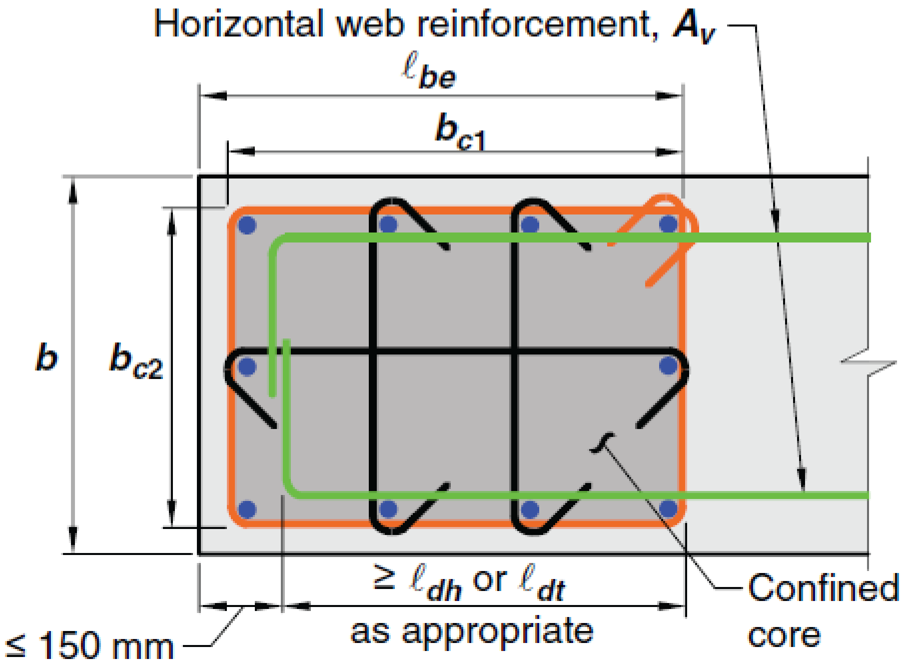
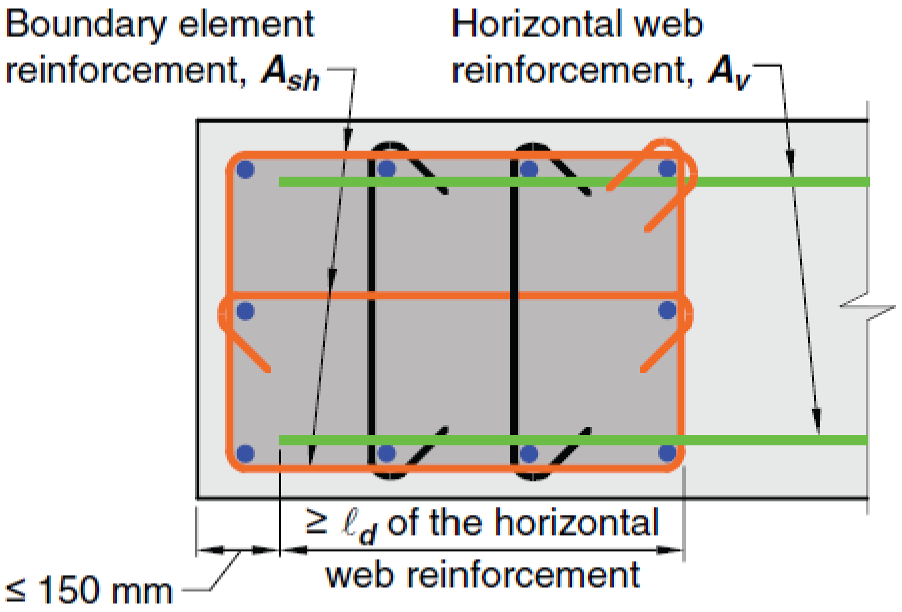


(a)
Option with standard hooks or headed reinforcement



(b)
Option with straight developed reinforcement

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

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Table 19.2.4.1(a)—Values of λ for lightweight concrete based on equilibrium density

w_c, kg/m^3	λ	
≤ 1600	0.75	(a)
$1600 < w_c \leq 2160$	$0.0075 w_c \leq 1.0$	(b)
> 2160	1.0	(c)

Table 19.2.4.1(b)—Values of λ for lightweight concrete based on composition of aggregates

Concrete	Composition of aggregates	λ
All-lightweight	Fine: ASTM C330M Coarse: ASTM C330M	0.75
Lightweight, fine blend	Fine: Combination of ASTM C330M and C33M Coarse: ASTM C330M	0.75 to 0.85[1]
Sand-lightweight	Fine: ASTM C33M Coarse: ASTM C330M	0.85
Sand-lightweight, coarse blend	Fine: ASTM C33M Coarse: Combination of ASTM C330M and C33M	0.85 to 1[2]

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Table 19.2.4.2—Modification factor λ

Concrete	Composition of aggregates	λ
All-lightweight	Fine: ASTM C330M Coarse: ASTM C330M	0.75
Lightweight, fine blend	Fine: Combination of ASTM C330M and C33M Coarse: ASTM C330M	0.75 to 0.85[1]
Sand-lightweight	Fine: ASTM C33M Coarse: ASTM C330M	0.85
Sand-lightweight, coarse blend	Fine: ASTM C33M Coarse: Combination of ASTM C330M and C33M	0.85 to 1[2]
Normalweight	Fine: ASTM C33M Coarse: ASTM C33M	1

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کاور بتن

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Table 20.5.1.3.1—Specified concrete cover for cast-in-place nonprestressed concrete members

Concrete exposure	Member	Reinforcement	Specified cover, mm
Cast against and permanently in contact with ground	All	All	75
Exposed to weather or in contact with ground	All	No. 19 through No. 57 bars	50
		No. 16 bar, MW200 or MD200 wire, and smaller	40
Not exposed to weather or in contact with ground	Slabs, joists, and walls	No. 43 and No. 57 bars	40
		No. 36 bar and smaller	20
	Beams, columns, pedestals, and tension ties	Primary reinforcement, stirrups, ties, spirals, and hoops	40

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Table 20.6.1.3.1—Specified concrete cover for cast-in-place nonprestressed concrete members

Concrete exposure	Member	Reinforcement	Specified cover, mm
Cast against and permanently in contact with ground	All	All	75
Exposed to weather or in contact with ground	All	No. 19 through No. 57 bars	50
		No. 16 bar, MW200 or MD200 wire, and smaller	40
Not exposed to weather or in contact with ground	Slabs, joists, and walls	No. 43 and No. 57 bars	40
		No. 36 bar and smaller	20
	Beams, columns, pedestals, and tension ties	Primary reinforcement, stirrups, ties, spirals, and hoops	40

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جدول۹-۶-۶ مقادیر حداقل ضخامت پوشش بتن روی میلگردها (میلیمتر) در شرایط محیطی بند ۹-۶-۴

نوع قطعه	نوع شرایط محیطی			
	متوسط	شدید	خیلی شدید	فوق‌العاده شدید
تیرها و ستون‌ها	۴۵	۵۰	۷۵	۷۵
دال ها و تیرچه‌ها	۳۰	۳۰	۶۰	۶۰
دیوار ها وپوسته‌ها	۲۵	۳۰	۵۵	۵۵
شالوده‌ها	۵۰	۶۰	۹۰	۹۰

مبحث۹ ویرایش ۱۳۹۲

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ازدیاد طول نسبی میلگرد

۹-۱۰-۷-۲-۴ به عنوان ضابطه شکل‌پذیری، ازدیاد طول نسبی دو طول معیار، یکی به طول ۱۰ برابر و دیگری به طول ۵ برابر قطر میلگرد (یعنی ε_{10} و ε_{5}) باید حداقل برابر با مقادیر مندرج در جدول ۹-۱۰-۲۱ باشد.

Table 20.2.1.3(c)—Uniform elongation requirements for ASTM A706 reinforcement

	Grade 420	Grade 550	Grade 690
Uniform elongation, minimum, percent			
Bar designation No.			
10, 13, 16, 19, 22, 25, 29, 32	9	7	6
36, 43, 57	6	6	6

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جدول ۹-۱۰-۲۱ حداقل مجاز ازدیاد طول نسبی میلگردهای فولادی در آزمایش کشش

رده فولاد / ازدیاد طول نسبی	S۲۴۰	S۳۴۰	S۴۰۰	S۵۰۰
حداقل مقدار مجاز ε_{10}	۰/۱۸	۰/۱۵	۰/۱۲	۰/۰۸
حداقل مقدار مجاز ε_{5}	۰/۲۵	۰/۱۸	۰/۱۶	۰/۱۰

مبحث۹ ویرایش ۱۳۹۲

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21.2.2.1 For deformed reinforcement, ε_{ty} shall be f_y/E_s. For Grade 420 deformed reinforcement, it shall be permitted to take ε_{ty} equal to 0.002.

ضریب کاهش مقاومت

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Table 21.2.2—Strength reduction factor ϕ for moment, axial force, or combined moment and axial force

Net tensile stain ε_t	Classification	ϕ			
		Type of transverse reinforcement			
		Spirals conforming to 25.7.3		Other	
$\varepsilon_t \le \varepsilon_{ty}$	Compression-controlled	0.75	(a)	0.65	(b)
$\varepsilon_{ty} < \varepsilon_t < 0.005$	Transition[1]	$0.75+0.15\dfrac{(\varepsilon_t-\varepsilon_{ty})}{(0.005-\varepsilon_{ty})}$	(c)	$0.65+0.25\dfrac{(\varepsilon_t-\varepsilon_{ty})}{(0.005-\varepsilon_{ty})}$	(d)
$\varepsilon_t \ge 0.005$	Tension-controlled	0.90	(e)	0.90	(f)

[1]For sections classified as transition, it shall be permitted to use ϕ corresponding to compression-controlled sections.

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Table 21.2.2—Strength reduction factor ϕ for moment, axial force, or combined moment and axial force

Net tensile stain ε_t	Classification	ϕ			
		Type of transverse reinforcement			
		Spirals conforming to 25.7.3		Other	
$\varepsilon_t \le \varepsilon_{ty}$	Compression-controlled	0.75	(a)	0.65	(b)
$\varepsilon_{ty} < \varepsilon_t < \varepsilon_{ty} + 0.003$	Transition[1]	$0.75+0.15\dfrac{(\varepsilon_t-\varepsilon_{ty})}{(0.003)}$	(c)	$0.65+0.25\dfrac{(\varepsilon_t-\varepsilon_{ty})}{(0.003)}$	(d)
$\varepsilon_t \ge \varepsilon_{ty} + 0.003$	Tension-controlled	0.90	(e)	0.90	(f)

[1]For sections classified as transition, it shall be permitted to use ϕ corresponding to compression-controlled sections.

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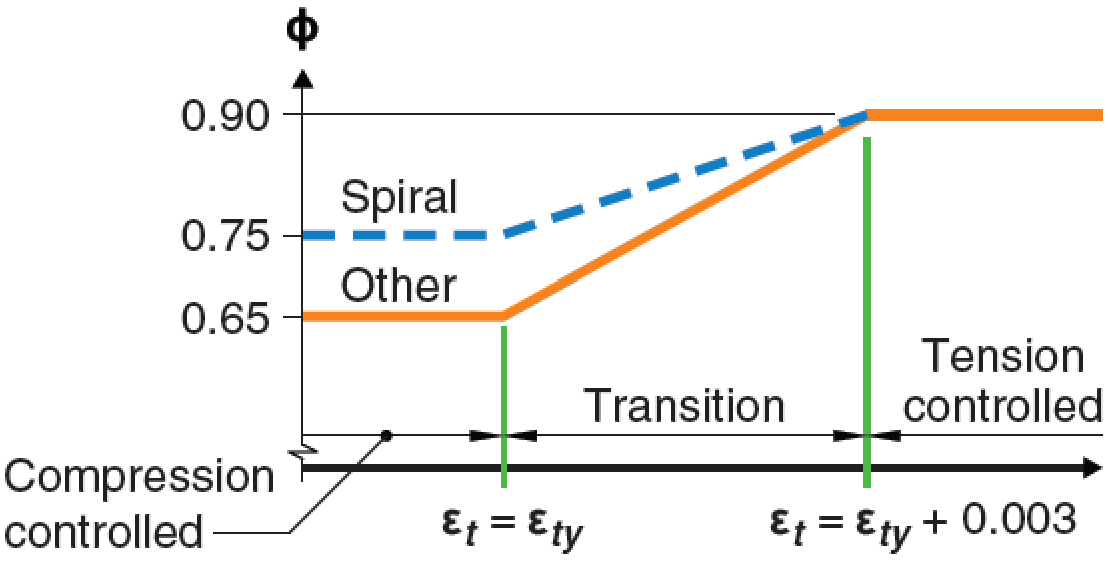

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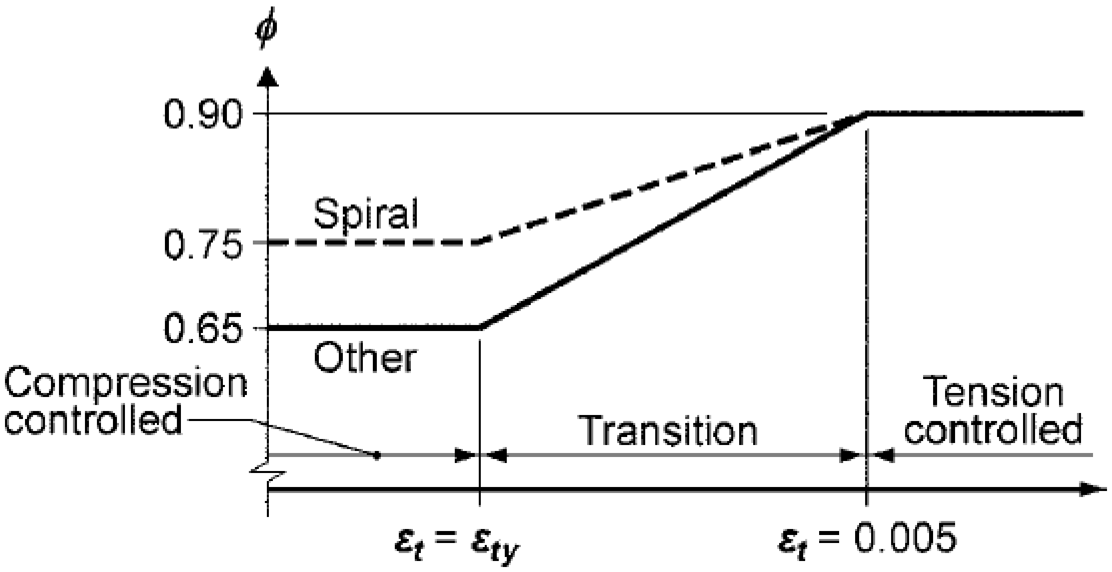

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Table 22.4.2.1—Maximum axial strength

Member	Transverse reinforcement	$P_{n,max}$	
Nonprestressed	Ties conforming to 22.4.2.4	$0.80P_o$	(a)
	Spirals conforming to 22.4.2.5	$0.85P_o$	(b)
Prestressed	Ties	$0.80P_o$	(c)
	Spirals	$0.85P_o$	(d)
Deep foundation member	Ties conforming to Ch. 13	$0.80P_o$	(e)

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Table 22.4.2.1—Maximum axial strength

Member	Transverse reinforcement	$P_{n,max}$	
Nonprestressed	Ties conforming to 22.4.2.4	$0.80P_o$	(a)
	Spirals conforming to 22.4.2.5	$0.85P_o$	(b)
Prestressed	Ties	$0.80P_o$	(c)
	Spirals	$0.85P_o$	(d)
Composite steel and concrete columns in accordance with Chapter 10	All	$0.85P_o$	(e)

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22.5.5.1 For nonprestressed members, V_c shall be calculated in accordance with Table 22.5.5.1 and 22.5.5.1.1 through 22.5.5.1.3.

Table 22.5.5.1—V_c for nonprestressed members

Criteria	V_c		
$A_v \geq A_{v,min}$	Either of:	$\left[0.17\lambda\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d$	(a)
		$\left[0.66\lambda(\rho_w)^{1/3}\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d$	(b)
$A_v < A_{v,min}$	$\left[0.66\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d$		(c)

Notes:

1. Axial load, N_u, is positive for compression and negative for tension.
2. V_c shall not be taken less than zero.

22.5.5.1.1 V_c shall not be taken greater than $\mathbf{0.42\lambda\sqrt{f_c'}\,b_w d}$.

22.5.5.1.2 In Table 22.5.5.1, the value of $N_u/6A_g$ shall not be taken greater than $\mathbf{0.05f_c'}$.

22.5.5.1.3 The size effect modification factor, λ_s, shall be determined by

$$\lambda_s = \sqrt{\frac{2}{1+0.004d}} \leq 1 \qquad (22.5.5.1.3)$$

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Table 22.6.5.2—v_c for two-way members without shear reinforcement

v_c		
Least of (a), (b), and (c):	$0.33\lambda_s\lambda\sqrt{f'_c}$	(a)
	$\left(0.17+\frac{0.33}{\beta}\right)\lambda_s\lambda\sqrt{f'_c}$	(b)
	$\left(0.17+\frac{0.083\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f'_c}$	(c)

Notes:

(i) λ_s is the size effect factor given in 22.5.5.1.3.

(ii) β is the ratio of long to short sides of the column, concentrated load, or reaction area.

(iii) α_s is given in 22.6.5.3.

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Table 22.6.5.2—Calculation of v_c for two-way shear

v_c		
Least of (a), (b), and (c):	$0.33\lambda\sqrt{f'_c}$	(a)
	$0.17\left(1+\frac{2}{\beta}\right)\lambda\sqrt{f'_c}$	(b)
	$0.083\left(2+\frac{\alpha_s d}{b_o}\right)\lambda\sqrt{f'_c}$	(c)

Note: β is the ratio of long side to short side of the column, concentrated load, or reaction area and α_s is given in 22.6.5.3.

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22.5.5.1.3 The size effect modification factor, λ_s, shall be determined by

$$\lambda_s = \sqrt{\frac{2}{1+0.004d}} \le 1 \qquad (22.5.5.1.3)$$

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Table 22.6.6.1—v_c for two-way members with shear reinforcement

Type of shear reinforcement	Critical sections	v_c		
Stirrups	All	$0.17\lambda_s\lambda\sqrt{f_c'}$		(a)
Headed shear stud reinforcement	According to 22.6.4.1	Least of (b), (c), and (d):	$0.25\lambda_s\lambda\sqrt{f_c'}$	(b)
			$0.17\left(1+\frac{2}{\beta}\right)\lambda_s\lambda\sqrt{f_c'}$	(c)
			$0.083\left(2+\frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f_c'}$	(d)
	According to 22.6.4.2		$0.17\lambda_s\lambda\sqrt{f_c'}$	(e)

Notes:

(i) λ_s is the size effect factor given in 22.5.5.1.3.

(ii) β is the ratio of long to short sides of the column, concentrated load, or reaction area.

(iii) α_s is given in 22.6.5.3.

22.6.6.2 It shall be permitted to take λ_s as 1.0 if (a) or (b) is satisfied:

(a) Stirrups are designed and detailed in accordance with 8.7.6 and $A_v/s \geq 0.17\sqrt{f_c'}\, b_o/f_{yt}$.
(b) Smooth headed shear stud reinforcement with stud shaft length not exceeding 250 mm is designed and detailed in accordance with 8.7.7 and $A_v/s \geq 0.17\sqrt{f_c'}\, b_o/f_{yt}$.

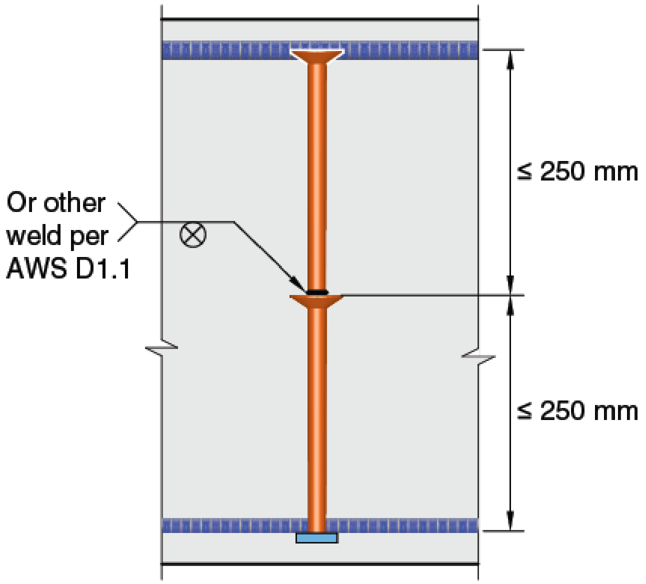

Fig. R22.6.6.2—Stacking (piggybacking) of headed shear stud reinforcement.

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24.2.3.4 Modulus of elasticity, E_c, shall be permitted to be calculated in accordance with 19.2.2.

24.2.3.5 For nonprestressed members, unless obtained by a more comprehensive analysis, effective moment of inertia, I_e, shall be calculated in accordance with Table 24.2.3.5 using:

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad (24.2.3.5)$$

Table 24.2.3.5—Effective moment of inertia, I_e

Service moment	Effective moment of inertia, I_e, mm[4]	
$M_a \le (2/3)M_{cr}$	I_g	(a)
$M_a > (2/3)M_{cr}$	$\dfrac{I_{cr}}{1-\left(\dfrac{(2/3)M_{cr}}{M_a}\right)^2\left(1-\dfrac{I_{cr}}{I_g}\right)}$	(b)

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24.2.3.4 Modulus of elasticity, E_c, shall be permitted to be calculated in accordance with 19.2.2.

24.2.3.5 For nonprestressed members, effective moment of inertia, I_e, shall be calculated by Eq. (24.2.3.5a) unless obtained by a more comprehensive analysis, but I_e shall not be greater than I_g.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (24.2.3.5a)$$

where M_{cr} is calculated by

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad (24.2.3.5b)$$

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$$\ell_d = \frac{f_y}{1.1\lambda\sqrt{f_c'}}\frac{\psi_t\psi_e\psi_s\psi_g}{\left(\frac{c_b+K_{tr}}{d_b}\right)}d_b \qquad (25.4.2.4a)$$

Table 25.4.2.3—Development length for deformed bars and deformed wires in tension

Spacing and cover	No. 19 and smaller bars and deformed wires	No. 22 and larger bars
Clear spacing of bars or wires being developed or lap spliced not less than d_b, clear cover at least d_b, and stirrups or ties throughout ℓ_d not less than the Code minimum or Clear spacing of bars or wires being developed or lap spliced at least $2d_b$ and clear cover at least d_b	$\left(\frac{f_y\psi_t\psi_e\psi_g}{2.1\lambda\sqrt{f_c'}}\right)d_b$	$\left(\frac{f_y\psi_t\psi_e\psi_g}{1.7\lambda\sqrt{f_c'}}\right)d_b$
Other cases	$\left(\frac{f_y\psi_t\psi_e\psi_g}{1.4\lambda\sqrt{f_c'}}\right)d_b$	$\left(\frac{f_y\psi_t\psi_e\psi_g}{1.1\lambda\sqrt{f_c'}}\right)d_b$

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$$\ell_d = \left(\frac{f_y}{1.1\lambda\sqrt{f_c'}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b+K_{tr}}{d_b}\right)}\right)d_b \qquad (25.4.2.3a)$$

Table 25.4.2.2—Development length for deformed bars and deformed wires in tension

Spacing and cover	No. 19 and smaller bars and deformed wires	No. 22 and larger bars
Clear spacing of bars or wires being developed or lap spliced not less than d_b, clear cover at least d_b, and stirrups or ties throughout ℓ_d not less than the Code minimum or Clear spacing of bars or wires being developed or lap spliced at least $2d_b$ and clear cover at least d_b	$\left(\frac{f_y\psi_t\psi_e}{2.1\lambda\sqrt{f_c'}}\right)d_b$	$\left(\frac{f_y\psi_t\psi_e}{1.7\lambda\sqrt{f_c'}}\right)d_b$
Other cases	$\left(\frac{f_y\psi_t\psi_e}{1.4\lambda\sqrt{f_c'}}\right)d_b$	$\left(\frac{f_y\psi_t\psi_e}{1.1\lambda\sqrt{f_c'}}\right)d_b$

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Table 25.4.2.5—Modification factors for development of deformed bars and deformed wires in tension

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Normalweight concrete	1.0
Reinforcement grade ψ_g	Grade 280 or Grade 420	1.0
	Grade 550	1.15
	Grade 690	1.3
Epoxy[1] ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement with clear cover less than $3d_b$ or clear spacing less than $6d_b$	1.5
	Epoxy-coated or zinc and epoxy dual-coated reinforcement for all other conditions	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Size ψ_s	No. 22 and larger bars	1.0
	No. 19 and smaller bars and deformed wires	0.8
Casting position[1] ψ_t	More than 300 mm of fresh concrete placed below horizontal reinforcement	1.3
	Other	1.0

[1]The product $\psi_t\psi_e$ need not exceed 1.7.

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Table 25.4.2.4—Modification factors for development of deformed bars and deformed wires in tension

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Lightweight concrete, where f_{ct} is specified	In accordance with 19.2.4.3
	Normalweight concrete	1.0
Epoxy[1] ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement with clear cover less than $3d_b$ or clear spacing less than $6d_b$	1.5
	Epoxy-coated or zinc and epoxy dual-coated reinforcement for all other conditions	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Size ψ_s	No. 22 and larger bars	1.0
	No. 19 and smaller bars and deformed wires	0.8
Casting position[1] ψ_t	More than 300 mm of fresh concrete placed below horizontal reinforcement	1.3
	Other	1.0

[1]The product $\psi_t\psi_e$ need not exceed 1.7.

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25.4.3 *Development of standard hooks in tension*

25.4.3.1 Development length ℓ_{dh} for deformed bars in tension terminating in a standard hook shall be the greater of (a) through (c):

(a) $\left(\frac{f_y \psi_e \psi_r \psi_o \psi_c}{23\lambda\sqrt{f_c'}}\right) d_b^{1.5}$ with ψ_e, ψ_r, ψ_o, ψ_c, and λ given in 25.4.3.2

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25.4.3 *Development of standard hooks in tension*

25.4.3.1 Development length ℓ_{dh} for deformed bars in tension terminating in a standard hook shall be the greater of (a) through (c):

(a) $\left(\frac{0.24 f_y \psi_e \psi_c \psi_r}{\lambda\sqrt{f_c'}}\right) d_b$ with ψ_e, ψ_c, ψ_r, and λ given in 25.4.3.2.

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Table 25.4.3.2—Modification factors for development of hooked bars in tension

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Normalweight concrete	1.0
Epoxy ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Confining reinforcement ψ_r	For No. 36 and smaller bars with $A_{th} \geq 0.4A_{hs}$ or s[1] $\geq 6d_b$[2]	1.0
	Other	1.6
Location ψ_o	For No. 36 and smaller diameter hooked bars: (1) Terminating inside column core with side cover normal to plane of hook $\geq$ 65 mm, or (2) With side cover normal to plane of hook $\geq 6d_b$	1.0
	Other	1.25
Concrete strength ψ_c	For $f_c' < 42$ MPa	$f_c'/105 + 0.6$
	For $f_c' \geq 42$ MPa	1.0

[1]s is minimum center-to-center spacing of hooked bars.

[2]d_b is nominal diameter of hooked bar.

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Table 25.4.3.2—Modification factors for development of hooked bars in tension

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Normalweight concrete	1.0
Epoxy ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Cover ψ_c	For No. 36 bar and smaller hooks with side cover (normal to plane of hook) $\geq$ 65 mm and for 90-degree hook with cover on bar extension beyond hook $\geq$ 50 mm	0.7
	Other	1.0
Confining reinforcement ψ_r[2]	For 90-degree hooks of No. 36 and smaller bars (1) enclosed along ℓ_{dh} within ties or stirrups[1] perpendicular to ℓ_{dh} at $s \leq 3d_b$, or (2) enclosed along the bar extension beyond hook including the bend within ties or stirrups[1] perpendicular to ℓ_{ext} at $s \leq 3d_b$	0.8
	For 180-degree hooks of No. 36 and smaller bars enclosed along ℓ_{dh} within ties or stirrups[1] perpendicular to ℓ_{dh} at $s \leq 3d_b$	
	Other	1.0

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A_{th} = total cross-sectional area of ties or stirrups confining hooked bars, mm^2

A_{hs} = total cross-sectional area of hooked or headed bars being developed at a critical section, mm^2

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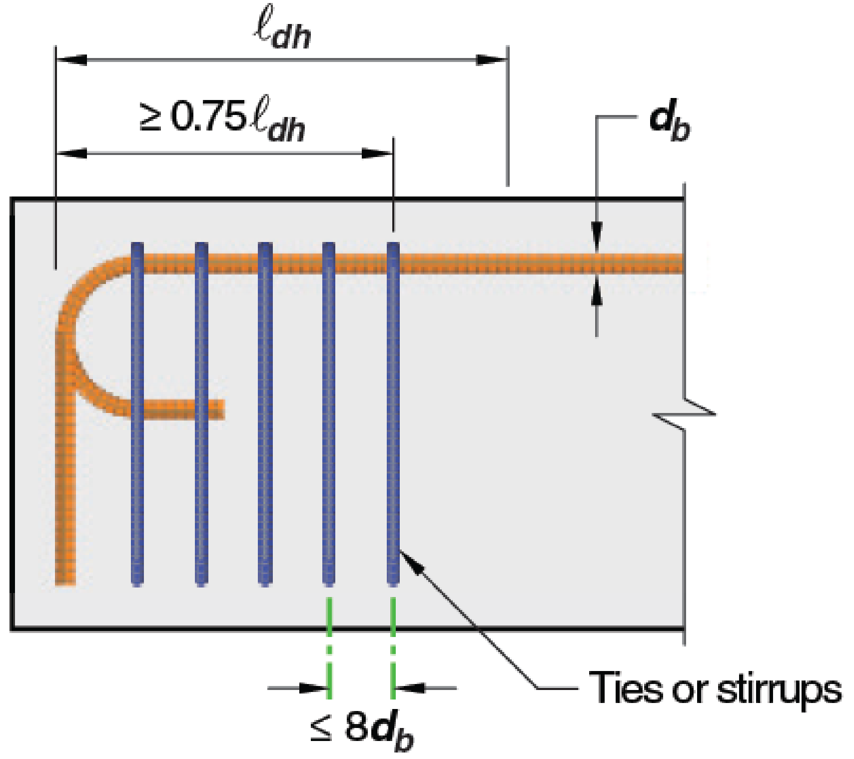

Fig. R25.4.3.3b—Confining reinforcement placed perpendicular to the bar being developed, spaced along the development length ℓ_{dh}, that contributes to anchorage strength of both 90- and 180-degree hooked bars.

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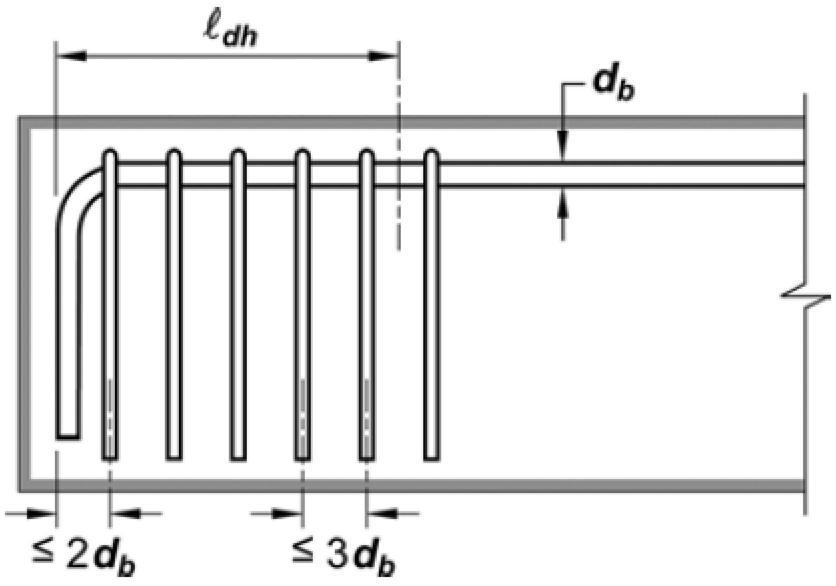

Fig. R25.4.3.2a—Ties or stirrups placed perpendicular to the bar being developed, spaced along the development length ℓ_{dh}.

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25.4.4.2 Development length ℓ_{dt} for headed deformed bars in tension shall be the longest of (a) through (c):

(a) $\left(\dfrac{f_y \psi_e \psi_p \psi_o \psi_c}{31\sqrt{f_c'}}\right) d_b^{1.5}$ with ψ_e, ψ_p, ψ_o, and ψ_c, given in 25.4.4.3

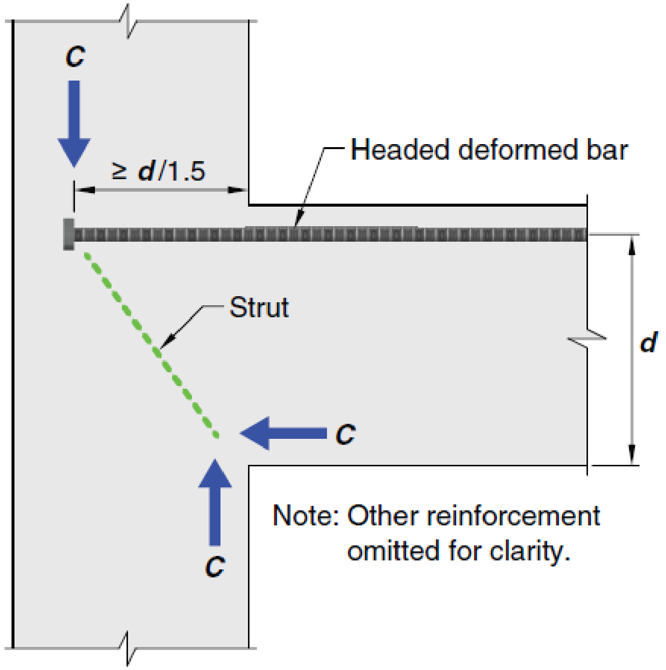

Fig. R25.4.4.2c—Breakout failure precluded in joint by keeping anchorage length greater than or equal to d/1.5.

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25.4.4.2 Development length ℓ_{dt} for headed deformed bars in tension shall be the greatest of (a) through (c):

(a) $\left(\dfrac{0.19 f_y \psi_e}{\sqrt{f_c'}}\right) d_b$, with ψ_e given in 25.4.4.3 and value of f_c' shall not exceed 40 MPa

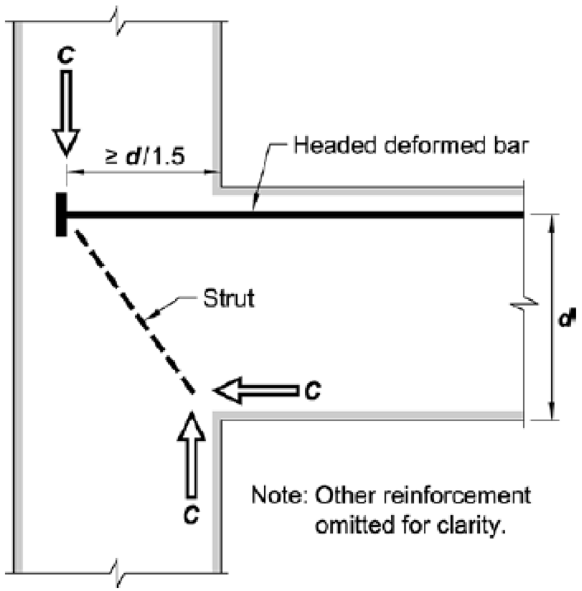

Fig. R25.4.4.2c—Breakout failure precluded in joint by keeping anchorage length greater than or equal to d/1.5.

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Table 25.4.4.3—Modification factors for development of headed bars in tension

Modification factor	Condition	Value of factor
Epoxy ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Parallel tie reinforcement ψ_p	For No. 36 and smaller bars with $A_{tt} \geq 0.3A_{hs}$ or s[1] $\geq 6d_b$[2,3]	1.0
	Other	1.6
Location ψ_o	For headed bars: (1) Terminating inside column core with side cover to bar $\geq$ 65 mm; or (2) With side cover to bar $\geq 6d_b$	1.0
	Other	1.25
Concrete strength ψ_c	For $f_c' < 42$ MPa	$f_c'/105 + 0.6$
	For $f_c' \geq 42$ MPa	1.0

[1]s is minimum center-to-center spacing of headed bars.

[2]d_b is nominal diameter of headed bar.

[3]Refer to 25.4.4.5.

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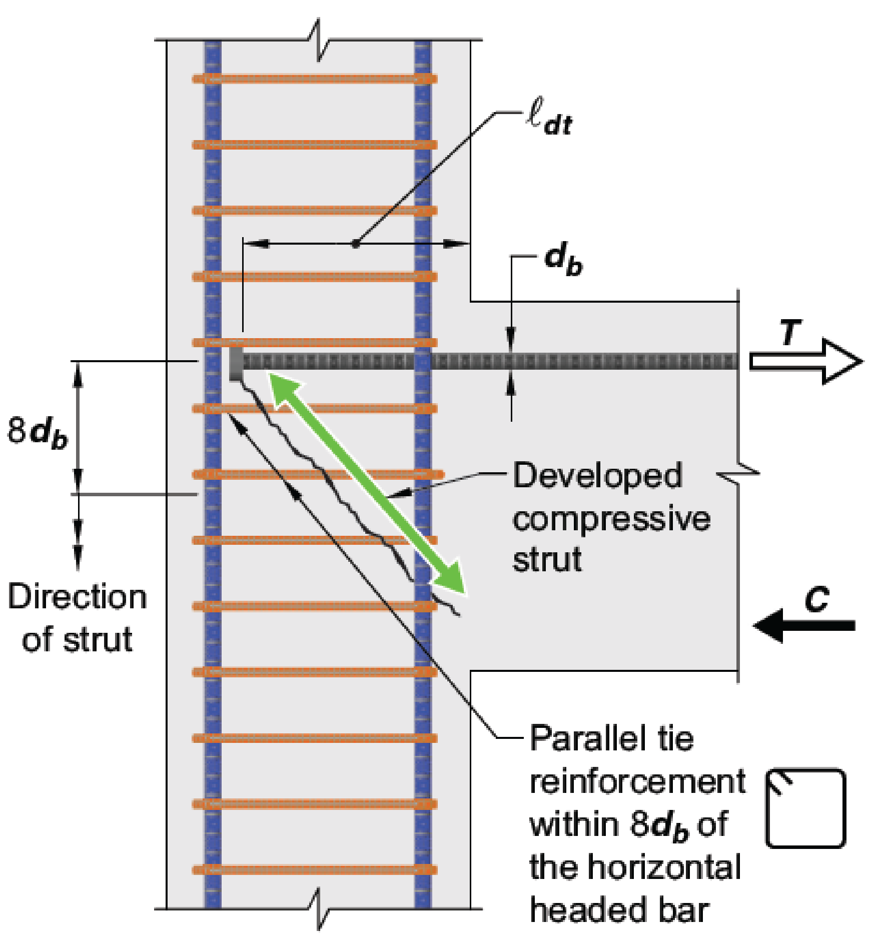
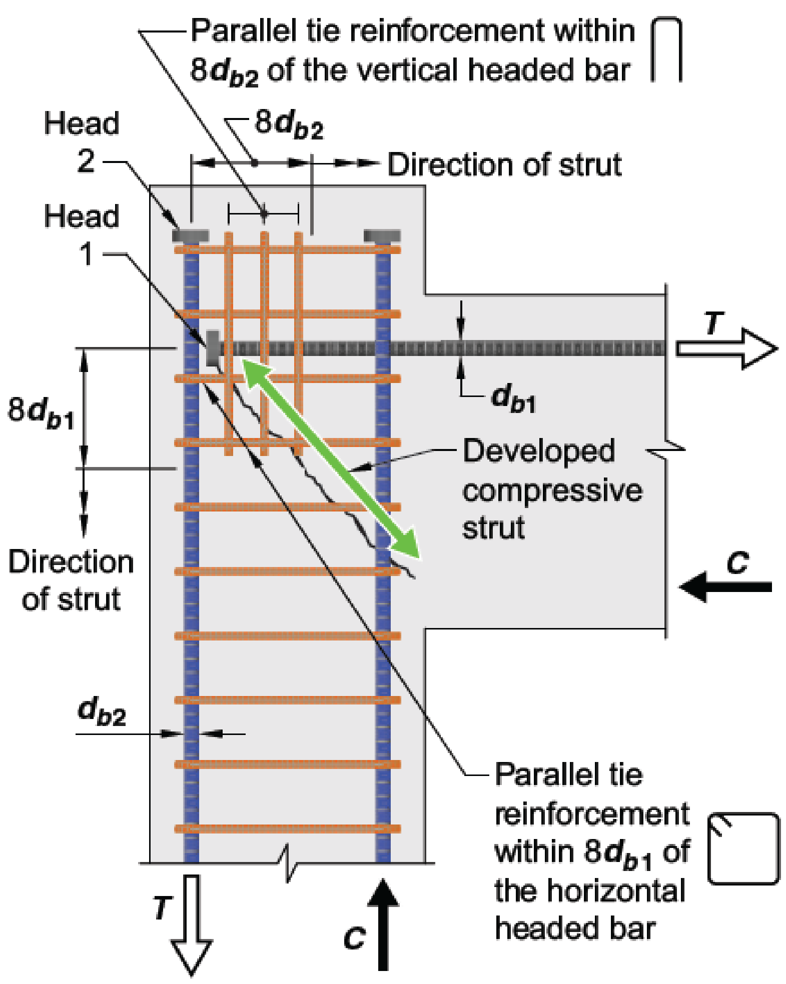
Fig. R25.4.4.4—Ties or stirrups placed parallel to the headed beam bars being developed in a beam-column joint that contribute to anchorage strength.



(a) Horizontal headed bars



(b) Vertical and horizontal headed bars

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APPENDIX A—DESIGN VERIFICATION USING NONLINEAR RESPONSE HISTORY ANALYSIS

CODE

A.1—Notation and terminology

A.1.1 *Notation*

B = bias factor to adjust nominal strength to seismic target reliabilities

D_u = ultimate deformation capacity; the largest deformation at which the hysteresis model is deemed valid given available laboratory data or other substantiating evidence

f_{ce}' = expected compressive strength of concrete, MPa

f_{ue} = expected tensile strength for nonprestressed reinforcement, MPa

f_{ye} = expected yield strength for nonprestressed reinforcement, MPa

ℓ_p = plastic-hinge length for analysis purposes, mm

R_{ne} = expected yield strength

V_{ne} = expected shear strength, N

θ_y = yield rotation, radians

ϕ_s = seismic resistance factor for force-controlled actions

COMMENTARY

RA.1—Notation and terminology

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Table A.8.4—Effective stiffness values[1]

Component		Axial	Flexural	Shear
Beams	nonprestressed	$1.0E_cA_g$	$0.3E_cI_g$	$0.4E_cA_g$
	prestressed	$1.0E_cA_g$	$1.0E_cI_g$	$0.4E_cA_g$
Columns with compression caused by design gravity loads[2]	$\geq 0.5A_gf_c'$	$1.0E_cA_g$	$0.7E_cI_g$	$0.4E_cA_g$
	$\leq 0.1A_gf_c'$ or with tension	$1.0E_cA_g$ (compression) $1.0E_sA_{st}$ (tension)	$0.3E_cI_g$	$0.4E_cA_g$
Structural walls[3]	in-plane	$1.0E_cA_g$	$0.35E_cI_g$	$0.2E_cA_g$
	out-of-plane	$1.0E_cA_g$	$0.25E_cI_g$	$0.4E_cA_g$
Diaphragms (in-plane only)[4]	nonprestressed	$0.25E_cA_g$	$0.25E_cI_g$	$0.25E_cA_g$
	prestressed	$0.5E_cA_g$	$0.5E_cI_g$	$0.4E_cA_g$
Coupling beams	with or without diagonal reinforcement	$1.0E_cA_g$	$0.07\left(\frac{\ell_n}{h}\right)E_cI_g$ $\leq 0.3E_cI_g$	$0.4E_cA_g$
Mat foundations	in-plane	$0.5E_cA_g$	$0.5E_cI_g$	$0.4E_cA_g$
	out-of-plane[5]		$0.5E_cI_g$	

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