Changes in ACI 318 Code Provisions for Earthquake-Resistant Structures, Part 2

Changes in sections governing the design of walls, diaphragms, foundations, and members that are not designated as part of the seismic force-resisting system

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"Building Code Requirements for Structural Concrete and Commentary (ACI 318-19)" maintains the format established in the previous edition.[1,2] However, Chapter 18—Earthquake-Resistant Structures, has substantive and consequential changes. Out of the 14 sections in Chapter 18, only 18.1—Scope, 18.5—Intermediate precast structural walls, and 18.9—Special precast moment frames, remain unchanged. Section 18.11—Special precast structural walls, was updated to include a single change prompted by a requirement added elsewhere in the chapter. However, Section 18.10—Special structural walls, underwent quite extensive changes.

Using the section numbers in the Code as headers, this and a previous article outline significant changes in Chapter 18 relative to the previous edition of the Code. Underlined and stricken texts indicate additions and deletions, respectively. Where warranted, an explanation is provided, and the significance of a change is discussed. Readers are reminded that Code sections are identified by numerals separated by decimal points, and Commentary sections are identified by the letter R plus the corresponding Code section identifier. Further, note that the ACI 318 Code uses the term "structural wall" as being synonymous with "shear wall." The terms are thus used interchangeably in this article.

The previous article, Part 1, published in March 2021, covers changes through Section 18.8, focused primarily on requirements for the design of frames. This article, Part 2, covers changes in:

- 18.10—Special structural walls;
- 18.11—Special structural walls constructed using precast concrete;
- 18.12—Diaphragms and trusses;
- 18.13—Foundations; and
- 18.14—Members not designated as part of the seismic force-resisting system.

18.10—Special structural walls

The basic design philosophy expressed in 18.10 was established in a corresponding section published in ACI 318-99.[3] The content remained essentially the same in subsequent editions, except for changes published in ACI 318-14, to incorporate lessons learned in the 2010 Chile earthquake. In ACI 318-19, the content has undergone quite extensive changes, and the scope now includes the newly introduced ductile coupled structural wall defined in 2.3.

18.10.2 *Reinforcement*

18.10.2.1 This section is modified as follows:

"The distributed web reinforcement ratios, ρ_ℓ and ρ_t, for structural walls shall be at least 0.0025, except that if V_u does not exceed $\cancel{A_{cv}}\lambda\sqrt{f_c'}\,\underline{A_{cv}}$, $\cancel{\rho_\ell}$ ~~and~~ ρ_t shall be permitted to be reduced to the values in 11.6."

A_{cv} is the gross area of the concrete section bounded by web thickness and length of section in the direction of shear force, f_c' is the specified compressive strength of concrete, ρ_ℓ is the ratio of the area of distributed longitudinal reinforcement to the gross concrete area perpendicular to that reinforcement, and ρ_t is the ratio of the area of distributed transverse reinforcement to the gross concrete area perpendicular to that reinforcement. The potential reduction in ρ_ℓ was allowed in ACI 318-14; however, the committee deemed this to have an undesirable influence on the inelastic response of the wall. If the value of ρ_ℓ is too low, there is a tendency for only a few wide, flexural cracks to form in the plastic hinge region; as ρ_ℓ is increased, the flexural cracks become more numerous and narrower. The latter behavior is desired because well-distributed flexural cracking reduces concentrated zones of cyclic inelastic strain in the longitudinal reinforcement, which in turn improves the inelastic deformation capacity of the wall.

18.10.2.3 Provisions for termination of wall longitudinal reinforcement are modified to address the perceived conservatism of the ACI 318-14 provision for low-rise walls and to address inconsistencies between engineering practice and the ACI 318-14 requirements. A new provision, summarized in Fig. 1, is intended to allow more frequent termination of longitudinal (vertical) reinforcement over the height of tall walls. ACI 318-14 specified that longitudinal bars be extended at least $0.8\ell_w$ above the point at which they are no longer required to resist flexure. This was cumbersome. For example, a 30 ft long (horizontal width) wall required a bar extension of 24 ft, which is the typical height of two stories. A provision was also added to prohibit lap splices of boundary longitudinal reinforcement in the region immediately above and below the critical section. Test results[4] have demonstrated that lap splices at the critical section tend to significantly reduce wall inelastic deformation capacity.

The text in this section was modified as follows:

"Reinforcement in structural walls shall be developed or spliced for f_y in tension in accordance with 25.4, 25.5, and (a) through (~~e~~d):

(a) ~~Longitudinal reinforcement shall extend beyond the point at which it is no longer required to resist flexure by least $0.8\ell_w$, except at the top of a wall~~
Except at the top of a wall, longitudinal reinforcement shall extend at least 12 ft above the point at which it is no longer required to resist flexure but need not extend more than ℓ_d above the next floor level.

(b) [Unchanged from ACI 318-14]

(c) Lap splices of longitudinal reinforcement within boundary regions shall not be permitted over a height equal to h_{sx} above, and ℓ_d below, critical sections where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements. The value of h_{sx} need not exceed 20 ft. Boundary regions include those within lengths specified in 18.10.6.4(a) and within a length equal to the wall thickness measured beyond the intersecting region(s) of connected walls.

(d) [Unchanged from (c) in ACI 318-14]

R18.10.2.3 is revised to note that "Bar terminations should be accomplished gradually over a wall height and should not be located close to critical sections where yielding of longitudinal reinforcement is expected... Strain hardening of reinforcement results in spread of plasticity away from critical sections as lateral deformations increase."

18.10.2.4 This new section provides requirements for longitudinal reinforcement at wall ends (Fig. 2). Minimum reinforcement ratio; bar extension beyond the critical, yielding section; and termination limits are defined. Commentary Section R18.10.2.4 notes that the requirements are intended to address two primary issues. First, if insufficient longitudinal reinforcement is provided in concrete walls, the cracking moment may exceed the nominal moment strength of the wall and sudden loss of strength and failure may occur when the wall first cracks. Additionally, the tension force generated by the longitudinal reinforcement at the ends of the wall may be insufficient to develop well-distributed secondary flexural cracks in the surrounding concrete, resulting in inelastic reinforcement strains being concentrated at only a limited number of cracks and potentially leading to reinforcement fracture.

Wall deformation capacity depends on the distribution of cracks within the plastic hinge region. Well-distributed flexural cracks over the plastic hinge region generally result in large plastic deformations prior to strength loss. Observations following the 1985 Chile earthquake[5] and the 2010-2011 New Zealand earthquakes[6] indicate that the relatively slender walls in multi-story buildings with light longitudinal reinforcement at the wall ends formed a

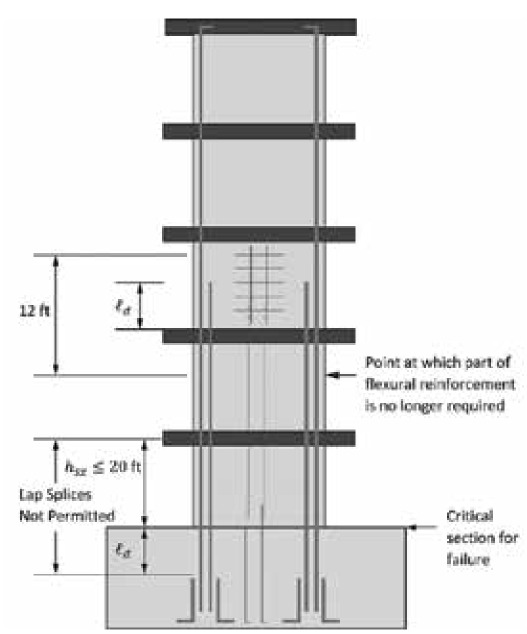


Fig. 1: Termination of wall flexural reinforcement and lap-splicing limitations at critical section

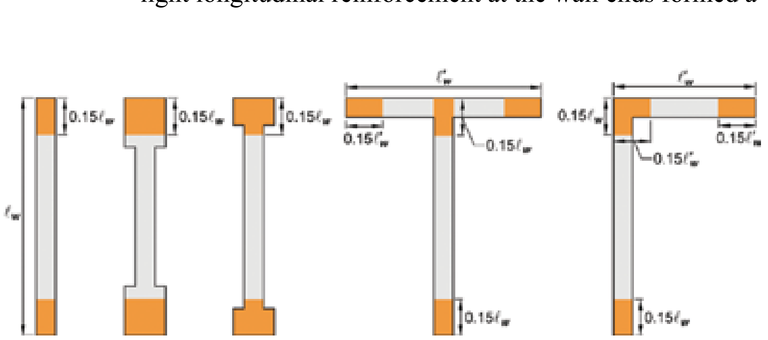


Fig. 2: Various wall configurations showing end regions requiring longitudinal reinforcement per 18.10.2.4(a) (ACI 318-19, Fig. R18.10.2.4[1])

limited number of cracks, or a single crack, in the plastic hinge region. The lack of distributed cracks led to the concentration of the inelastic deformation capacity in a significantly reduced plastic hinge length, and this was presumed to be the cause of the observed fractures of longitudinal reinforcement.

The requirements in this provision are unlikely to impact construction significantly in the United States, as the general practice is to use relatively few walls that tend to be heavily reinforced at the wall ends. Note that this provision is not specific to the "boundary elements of special structural walls" defined in 18.10.6. It applies to longitudinal reinforcement located at the wall ends in general.

18.10.2.5 This new section requires that reinforcement in coupling beams be developed or spliced for 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 [as special moment frame beams], the development length of longitudinal reinforcement is required to be 1.25 times the values calculated for f_y in tension.

(b) If coupling beams are reinforced according to 18.10.7.4, the development length of diagonal reinforcement is required to be 1.25 times the values calculated for f_y in tension."

18.10.3 *Design forces*

As with previous editions, ACI 318-19 requires beam and column sections of special moment frames to be designed for the largest shear V_e that can develop at such a beam or column section (see 18.6.5.1 and 18.7.6.1.1, respectively). In no case can V_e be smaller than the factored shear obtained at that section from an analysis of the structure under code-prescribed seismic forces.

ACI 318-14 (and prior editions of ACI 318), however, required a special shear wall section to be designed for the factored shear V_u obtained at the section from analysis of the structure that includes the shear wall under code-prescribed seismic forces. No attempt was made to determine the largest shear that can develop at the shear wall section. This has now changed in ACI 318-19.

18.10.3.1 This new section requires consideration of flexural overstrength. For flexure-controlled walls, flexural overstrength reduces collapse risk and may improve performance; however, also for flexure-controlled walls exhibiting nonlinear response, shear demand is determined in part by flexural strength. Thus, flexural overstrength contributes to increased shear demand and is considered in defining shear demand used in design by ACI 318-19. Many other design codes, standards, and guides around the world, including CSA A23.3,[7] require flexural overstrength to be accounted for in design.

Multiple studies have investigated, and multiple design codes, standards, and guides around the world specify, dynamic amplification of shear demand (due to higher mode effects) in concrete walls. Research by Pugh et al.[8] showed, using nonlinear dynamic analysis of idealized walled buildings ranging in height from 6 to 24 stories, that the dynamic amplification factor ω_v can range from 1.1 to 2.5. Pugh et al.[8] showed also that dynamic amplification equations included in the New Zealand design standard[9] as well as those recommended by SEAOC,[10] which define dynamic amplification on the basis of building height, provide fairly consistent, modest underprediction of dynamic amplification.

ACI 318-19 requires cross sections of a special shear wall to be designed for an amplified shear V_e equal to the V_u obtained from analysis of the structure under code-prescribed seismic forces, amplified by an overstrength factor Ω_v and a dynamic amplification factor ω_v:

$$V_e = \Omega_v \omega_v V_u \leq 3V_u$$

The values for Ω_v and ω_v vary with h_{wcs}/ℓ_w, where h_{wcs} is the height of the entire structural wall above the critical section for flexural and axial loads. Per Table 18.10.3.1.2, if $h_{wcs}/\ell_w > 1.5$, $\Omega_v = M_{pr}/M_u \geq 1.5$ for the load combination producing the largest value of Ω_v. For $h_{wcs}/\ell_w \leq 1.5$, $\Omega_v = 1.0$. Further, for $h_{wcs}/\ell_w < 2.0$, $\omega_v = 1.0$. For $h_{wcs}/\ell_w \geq 2.0$, ω_v varies with the number of stories n_s.

$$\omega_v = 0.9 + n_s/10 \text{ for } n_s \leq 6$$

$$\omega_v = 1.3 + n_s/30 \leq 1.8 \text{ for } n_s > 6$$

Note that $n_s \geq 0.007h_{wcs}$, with h_{wcs} in units of inches.

This is a very important code change that will have a major impact on the design of flexure-controlled shear walls.

18.10.4 *Shear strength*

18.10.4.6 This new section makes it clear that the requirements of 21.2.4.1 are not applicable for walls or wall piers designed according to the displacement-based approach of 18.10.6.2. Section 21.2.4.1 requires that the strength reduction factor ϕ used for shear strength in members designed to resist earthquake effects must be 0.6, rather than the usual 0.75, if the nominal shear strength of the member is less than the shear corresponding to the nominal flexural strength of the member.

18.10.6 *Boundary elements of special structural walls*

18.10.6.2 Item (b) in this section is new, and it introduces a separate wall drift capacity check that is in addition to the drift check required by ASCE/SEI 7.[11]

Abdullah and Wallace[12] created a database of 164 tests of walls with special boundary elements and determined that drift capacity is primarily a function of three ratios: ℓ_w/b, c/b, and $V_e/(A_{cv}\sqrt{f_c'})$, where b is the wall thickness and c is the neutral axis depth. Based on a regression analysis of the data, the mean drift capacity for walls with special boundary elements is given by:

$$\frac{\delta_c}{h_{wcs}} = \frac{1}{100}\left(4 - \frac{1}{50}\left(\frac{\ell_w}{b}\right)\left(\frac{c}{b}\right) - \frac{V_e}{8\sqrt{f_c'}A_{cv}}\right) \geq 0.015$$

where δ_c is the wall displacement capacity at the top of the wall. Item (b) in 18.10.6.2 requires that the wall drift demand, estimated as $1.5\delta_u/h_{wcs}$, must be less than or equal to the wall drift capacity determined using the aforementioned equation. This provision results in roughly a 10% probability of strength loss for design earthquake-level shaking. The new provision also includes a simplified approach to satisfying the drift capacity check by including a minimum wall compression zone width. Assuming $V_e/(8A_{cv}\sqrt{f_c'}) = 1.0$ and $\delta_u/h_{wcs} = 0.015$, the requirement that $1.5\delta_u/h_{wcs} \leq \delta_c/h_{wcs}$ can be reduced to this simple constraint:

$$b \geq \sqrt{0.025c\ell_w}$$

In general, the new provision will require redesign of walls with large values of ℓ_w/b (> 15 to 20), large values of c/b (> 3 to 4), and high shear demands as indicated by $V_u/(A_{cv}\sqrt{f_c'})$ (> 6 to 8). The provision also applies to coupled walls, and it may require the use of thicker wall piers for coupled walls.

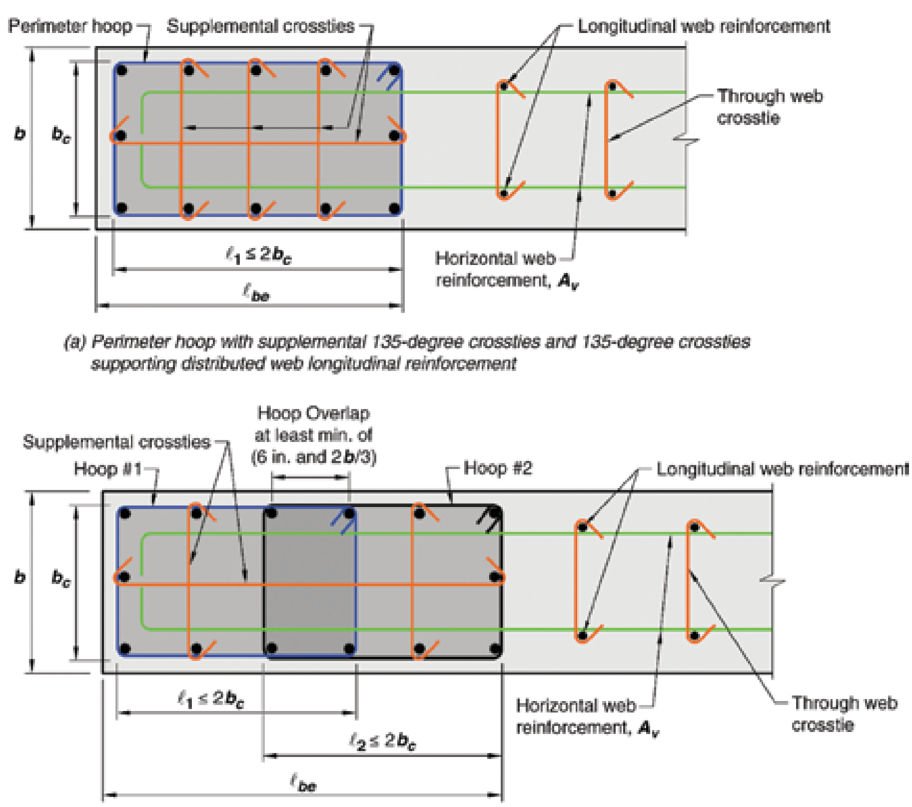
18.10.6.4 Configuration requirements for transverse reinforcement in boundary elements and webs were modified in ACI 318-19 (Fig. 3). The changes were prompted by observations following recent earthquakes as well as the results of laboratory testing. The length of a hoop leg must not exceed two times the boundary element thickness; and adjacent hoops must overlap at least 6 in. or two-thirds the boundary element thickness, whichever is smaller. Also, for a distance above and below the critical section equal to the greater of ℓ_w and $M_u/4V_u$, web vertical reinforcement must be enclosed by the corner of a hoop or by a crosstie with a seismic hook at each end.

18.10.6.5 This section provides requirements on horizontal and transverse reinforcement in walls in which special boundary elements are not required by 18.10.6.2 and 18.10.6.3. It has been updated to account for Grade 80 and Grade 100 longitudinal reinforcement. If the reinforcing ratio at the wall boundary exceeds $400/f_y$, the vertical spacing of transverse reinforcement at the wall boundary is required to be in accordance with a new Table 18.10.6.5(b), reproduced herein as Table 1. Note that the spacing has not changed for walls with Grade 60 primary flexural reinforcement.

18.10.9 *Ductile coupled walls*

The prevalent seismic force-resisting system used for modern high-rise concrete buildings in high seismic regions comprises reinforced concrete core walls. Current design practice uses the provisions of ACI 318 for design of the materials (Chapter 18) and the seismic design coefficients from ASCE/SEI 7[11] (response modification coefficient R = 5 or 6, for example) for calculation of the design lateral loads based on the ductility of the force-resisting system. The special reinforced concrete shear wall has historically included all shear wall variations. That is, differentiation in behavior is not recognized between slender versus squat walls, flanged versus rectangular walls, and coupled versus cantilever walls. As of the 2019 edition of ACI 318, a new system definition has been created in 2.3—Terminology, to recognize the ductile coupled structural wall (DCSW). The definition for a DCSW system refers to Section 18.10.9, where the requirements for the system are given.

The performance objective of the DCSW system is to dissipate most of the energy in the coupling beams—analogous to the strong column weak beam performance



(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. 3: Configurations of boundary transverse reinforcement and web crossties (ACI 318-19, Fig. R18.10.6.4a)

objective in moment frames. Studies were conducted to identify system characteristics that led to coupling beam energy dissipation of no less than 80% of total system energy dissipation under risk-targeted maximum considered earthquake (MCE_R) ground motions. In these studies, nonlinear response history analyses were conducted using spectrally matched ground motion records on a variety of coupled shear wall archetypes. Archetypes ranged from five to 50 stories in height, and they accommodated a range of longitudinal reinforcement ratios in the coupling beams as well as the shear walls. Results of these analyses are presented in Fig. 4. The x-axis represents the aspect (clear span to total depth) ratio of the coupling beams, with D designating diagonally reinforced coupling beams and M indicating coupling beams detailed as moment frame beams. The y-axis is the percentage of total system energy dissipation that occurs in the coupling beams alone. The analyses show that coupling beams with aspect ratios ranging from 2 to 5 tend to be effective at dissipating most of the system energy.

The analyses show that the primary characteristics of a DCSW system are geometry based. Squat walls are too stiff to allow sufficient story drift for coupling beams to behave inelastically, so each wall in a DCSW system needs to have a total-height-to-length ratio of at least 2.0. Squat coupling beams overcouple the seismic force-resisting system, leading to significant energy dissipation in the wall piers rather than the coupling beams. Thus, each coupling beam in a DCSW system needs to have a length-to-total-depth ratio of at least 2.0. Very slender coupling beams, designated as having aspect ratios greater than 5.0, are too weak to contribute sufficient hysteretic energy dissipation. Thus, such beams are allowed in no more than 10% of all floor levels of the building. Further, the flexural reinforcement in coupling beams conforming to these geometric constraints is required to develop $1.25f_y$ at each end to dissipate the intended hysteretic energy. This requirement is intended to preclude the use of fixed-pinned coupling beams that might be considered where one wall pier has insufficient width to develop the coupling beam reinforcement.

Lastly, the noted requirements of the DCSW system are in addition to existing requirements for special structural walls and coupling beams.

Table 1:
Maximum vertical spacing of transverse reinforcement at wall boundary (ACI 318-19, Table 18.10.6.5(b)[1])

Grade of primary flexural reinforcing bar	Transverse reinforcement required	Maximum vertical spacing of transverse reinforcement*	
60	Within the greater of ℓ_w and $M_u/4V_u$ above and below critical sections†	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†	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†	Lesser of:	$4d_b$
			6 in.
	Other locations	Lesser of:	$6d_b$
			6 in.

*In this table, d_b is the diameter of the smallest primary flexural reinforcing bar.

†Critical sections are defined as locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements.

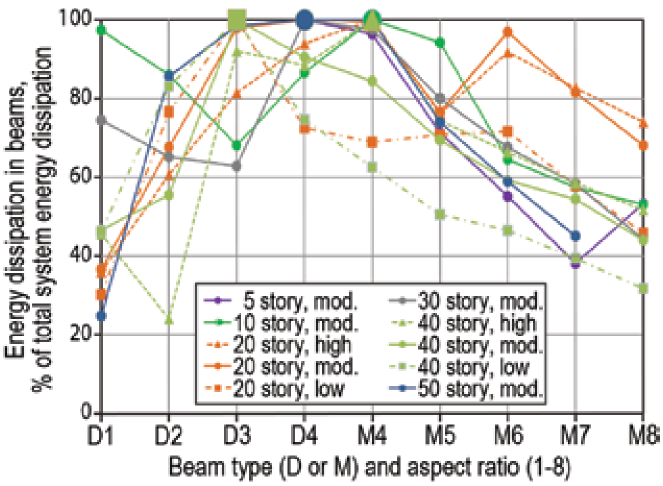


Fig. 4: Energy dissipated by coupling beams. Labels "low," "mod.," and "high" refer to the relative quantities of flexural reinforcement in the beams *(Courtesy of Magnusson Klemencic Associates)*

18.11—Special structural walls constructed using precast concrete

18.11.2 *General*

18.11.2.1 While special structural walls constructed using precast concrete are required to comply with the provisions of 18.10—Special structural walls, the new requirement for minimum longitudinal reinforcement (18.10.2.4) at the wall ends does not apply for precast walls where deformation demands are concentrated at the panel joints.

18.12—Diaphragms and trusses

18.12.7 *Reinforcement*

18.12.7.4 In ACI 318-14, this section required Type 2 splices where mechanical splices are used to transfer forces between the diaphragm and the vertical elements of the seismic force-resisting system. ACI 318-19 retains the same requirement for Grade 60 reinforcement and adds that Grade 80 and Grade 100 reinforcement must not be mechanically spliced for this purpose.

18.12.7.5 Table 20.2.2.4(a) permits the maximum design yield strength of reinforcement to be 80,000 psi for portions of a collector—for example, at and near critical sections. To control diaphragm cracking over the length of the collector, the average stress in the collector needs to be limited. This new section therefore requires longitudinal reinforcement for collectors to be proportioned such that the average tensile stress over length (a) or (b) does not exceed ϕf_y, where the value of f_y is limited to 60,000 psi:

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

(b) Length between two vertical elements.

18.12.11 *Precast concrete diaphragms*

Two subsections are new to ACI 318-19:

18.12.11.1 Diaphragms and collectors constructed using precast concrete members with composite topping slab and not satisfying 18.12.4 [interface between topping slab and precast diaphragm clean, free of laitance, and intentionally roughened], and untopped precast concrete diaphragms, are permitted provided they satisfy the requirements of ACI 550.5.[13] Cast-in-place noncomposite topping slab diaphragms are required to satisfy 18.12.5 and 18.12.6 [minimum thickness of diaphragms].

18.12.11.2 Connections and reinforcement at joints used in the construction of precast concrete diaphragms satisfying 18.12.11.1 are required to have been tested in accordance with ACI 550.4.[14]

18.13—Foundations

This section has greatly expanded from ACI 318-14 to ACI 318-19, because the seismic design provisions for deep foundations have been imported from IBC[15] and ASCE/SEI 7.[11] These changes are too voluminous to be detailed in this article.

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

18.14.2 *Design actions*

18.14.2.1 This section has been modified as shown:

"Members not designated as part of the seismic-force-resisting system shall be evaluated for gravity load combinations of ~~$(1.2D + 1.0L + 0.2S)$ or $0.9D$, whichever is critical,~~ 5.3 including the effect of vertical ground motion acting simultaneously with the design displacement δ_u. ~~The load factor on the live load, L, shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 100 lb/ft^2.~~"

The load combinations of ACI 318-14 thus have effectively become:

$(1.2 + 0.2S_{DS})D + 1.0L + 0.2S)$ or

$(0.9 - 0.2S_{DS})D$.

If gravity frame members remain elastic ($M_u < M_n$, $V_u < V_n$) under the imposed design earthquake displacements δ_u, then the sections to be satisfied are: 18.14.3.2(a) for beams, 18.14.3.2(c) for highly axially loaded columns that may fail in compression, and 18.14.3.2(b) for other columns.

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

18.14.3.2 Modifications have been made to items (a), (b), and (c), and item (d) has been added:

"(a) Beams shall satisfy 18.6.3.1. Transverse reinforcement shall be provided throughout the length of the beam at a spacing not to exceed $d/2$. Where factored axial force exceeds $A_g f_c'/10$, transverse reinforcement shall be hoops satisfying 18.7.5.2 at a spacing ~~s_o, according to 18.14.3.2(b)~~ not to exceed the lesser of $6d_b$ of the smallest enclosed longitudinal bar and 6 in.

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

(c) Columns with factored gravity axial forces exceeding $0.35P_o$ shall satisfy 18.14.3.2(b) and 18.7.5.7. The minimum amount of transverse reinforcement provided shall be, ~~one-half of that required by 18.7.5.4 and spacing shall not exceed s_o for the full column length.~~ for rectilinear hoops, one-half the greater of Table 18.7.5.4 parts (a) and (b) and, for spiral or circular hoops, one-half the greater of Table 18.7.5.4 parts (d) and (e). This transverse reinforcement shall be provided over a length ℓ_o, as defined in 18.7.5.1, from each joint face.

(d) Joints shall satisfy Chapter 15."

18.14.5 *Slab-column connections*

This section has been modified in significant ways. Only the post-tensioned slab provisions are new. The other changes essentially represent reorganization.

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

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

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

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

Note that the text following "…defined in 22.6.4.1" in the previous version of 18.14.5.1 has been deleted from this section, and a portion has been included in 18.14.5.3 as indicated in the following description.

18.14.5.2 Note that while this section includes requirements from 18.14.5.1 in the previous edition, the requirements in (b) are new.

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

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

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

18.14.5.3 "…~~if~~ $\cancel{\Delta_x/h_{sx} \geq 0.035 - (1/20)(v_{ug}/\phi v_c)}$~~.~~ Required slab shear reinforcement shall provide $v_s \geq 3.5\sqrt{f_c'}$ at the slab critical section and shall extend at least four times the slab thickness from the face of the support adjacent to the slab critical section. ~~The shear reinforcement requirements of this provision shall not apply if~~ $\cancel{\Delta_x/h_{sx} \leq 0.005}$~~. The value of~~ $\cancel{(\Delta_x/h_{sx})}$ ~~shall be taken as the greater of the values of the adjacent stories above and below the slab column connection.~~ $\cancel{v_c}$ ~~shall be calculated in accordance with 22.6.5.~~ $\cancel{v_{ug}}$ ~~is the factored shear stress on the slab critical section for two-way action due to gravity loads without moment transfer.~~"

Concluding Remarks

ACI 318-19 Chapter 18—Earthquake-Resistant Structures, includes many substantive changes. In Part 1, we summarized the changes in Sections 18.2—General, plus changes in Sections 18.3 through 18.8, which govern the design of ordinary and intermediate moment frames as well as the design of beams, columns, and joints of special moment frames.

In this, Part 2, we discussed the latter sections of Chapter 18. Key changes include:

ASTM A706 Grade 80 and Grade 100 bars are now permitted to resist bending moments, shear forces, and axial forces in special structural walls, including coupling beams and wall piers. Section 18.10—Special structural walls, has undergone by far the most substantive changes.

The corresponding section in ACI 318-95[16] underwent profound changes in ACI 318-99 but remained essentially unchanged through ACI 318-11.[17] Significant changes were made in ACI 318-14 in view of observations made after the Chile earthquake of 2010 and the Christchurch, New Zealand, earthquakes of 2010-2011. Further major changes were made in ACI 318-19 in view of observations made after recent earthquakes and tests. The biggest change is in the shear design of shear walls, which is now much more conservative. It is believed that this change will have a beneficial impact on the performance of concrete structures with seismic force-resisting systems comprising special structural walls.

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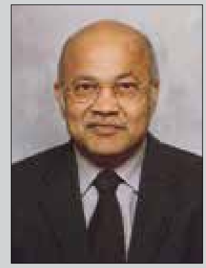

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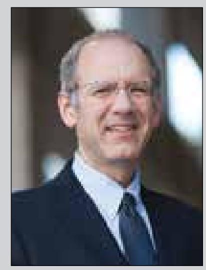

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