Significant changes to ACI 318-08 relative to precast/prestressed concrete: Part 2

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Editor's quick points

- This second of three papers describes the changes from the 2005 edition to the 2008 edition of ACI 318, *Building Code Requirements for Structural Concrete and Commentary*, for chapters 9 through 20.
- ACI 318 underwent a major revision with this version.
- Part 3 will follow in a subsequent issue of the *PCI Journal*.

Significant changes have been made since American Concrete Institute (ACI) Committee 318 published the 2005 *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05).*[1] The changes in the new 2008 edition[2] are summarized in this paper.

The intent of this article is to provide a summary of significant changes affecting conventionally reinforced concrete, precast concrete, and prestressed concrete (including post-tensioned concrete). This information should be useful to building officials, design engineers, practitioners, and the academic community. Changes to chapters 1 through 8 of ACI 318-08 were discussed in part 1 of this article series, published as a member supplement to the March–April 2008 issue of the *PCI Journal*. Changes to chapter 9 through 20 of ACI 318-08 are discussed in this part 2 of the article series. Changes to chapter 21 and the appendices will be discussed in part 3, which will appear in a subsequent issue of the *PCI Journal*.

ACI 318-08 will be the reference document for concrete design and construction in the 2009 edition of the *International Building Code* (IBC),[3] which will continue to reference ASCE 7-05.[4]

All section and chapter numbers used in this paper refer to those of ACI 318-08 unless otherwise noted.

Chapter 9: "Strength and Serviceability Requirements"

The new commentary section, R9.2.1(a), provides valuable and much-needed clarification. It points out that the load-factor modification of section 9.2.1(a) is different from the live-load reduction based on the loaded area that is typically allowed in the legally adopted general building code. The live-load reduction in the code adjusts the nominal load *L*. The lesser load factor in section 9.2.1(a) reflects the reduced probability of the joint occurrence of maximum values of multiple transient loads at the same time. The reduced live loads specified in the legally adopted general building code can be used simultaneously with the 0.5 load factor specified in section 9.2.1(a).

In section 9.3.2.2, the strength-reduction factor φ for spirally reinforced columns was increased from 0.70 to 0.75. Commentary section R9.3.2 notes that this increase is partly due to the superior performance of spirally reinforced columns when subjected to excessive loads or extreme excitations[5] and is partly due to new reliability analyses.[6]

The φ-factor modifications of section 9.3.4(a)–(c) are now also applicable to structures that rely on intermediate precast concrete structural walls to resist earthquake effects in seismic design categories (SDC) D, E, or F. Previously, the modifications applied only to structures that rely on special moment frames or special structural walls to resist earthquake effects.

In section 9.3.5, the φ-factor for plain concrete was increased from 0.55 to 0.60. As stated in commentary section R9.3.5, this is partly due to recent reliability analysis and a statistical study of concrete properties.[6]

The first paragraph of section R9.3.4 of ACI 318-05 was eliminated. In section R9.4, it is clarified that the maximum specified yield strength of nonprestressed reinforcement f_y in section 21.1.5 is 60,000 psi (420 MPa) in special moment frames and special structural walls.

Chapter 10: "Flexure and Axial Loads"

"For a compression member with a cross section larger than required by considerations of loading," section 10.8.4 permits the minimum reinforcement to be based on a reduced effective area A_g not less than one-half the total area. ACI 318-05 used to state that the provision does not apply in regions of high seismic risk. ACI 318-08 now states that this provision does "not apply to special moment frames or special structural walls designed in accordance with chapter 21."

The most significant change in chapter 10 is a rewriting of sections 10.10 through 10.13 of ACI 318-05 into the new section 10.10, "Slenderness Effects in Compression Members." The slenderness provisions are reorganized to reflect "current practice where second-order effects are considered primarily using computer analysis techniques," while the style of presentation used by ACI 318 since 1971 is retained. The moment magnifier method is also retained as an alternate procedure.

Section 10.10.1 permits slenderness effects to be neglected

"for compression members not braced against sidesway when:

$$\frac{kl_u}{r} \le 22 \text{"}$$

and

"for compression members braced against sidesway when:

$$\frac{kl_u}{r} \le 34 - 12\left(\frac{M_1}{M_2}\right) \le 40 \text{"}$$

where

k = effective length factor

l_u = unsupported length

r = radius of gyration

M_1 = smaller factored end moment

$_2$ = larger factored end moment

M_1/M_2 = positive if a compression member is bent in single curvature

A new feature permits a compression member to be considered braced against sidesway when "bracing elements have a total stiffness, resisting lateral movement of that story, of at least 12 times the gross stiffness of the columns within the story."

Section 10.10.2 requires that when slenderness effects are not neglected as permitted by section 10.10.1, "the design of compression members, restraining beams, and other supporting members be based on the factored forces and moments from a second-order analysis satisfying [section] 10.10.3, 10.10.4, or 10.10.5."

Section 10.10.3 is titled "Nonlinear Second-Order Analysis," section 10.10.4 contains requirements for elastic second-order analysis, and section 10.10.5 details moment magnification procedure. The members being discussed are also required to satisfy sections 10.10.2.1 and 10.10.2.2. Section 10.10.2.1 requires that second-order effects in compression members, restraining beams, or

Figure 1. Stud rails are used as slab shear reinforcement. Photo courtesy of Decon U.S.A. Inc.

other structural members not exceed 40% of the moment due to first-order effects. Section 10.10.2.2 requires that second-order effects "be considered along the length of compression members." This can be done using the non-sway moment magnification procedure outlined in section 10.10.6.

Section 10.10.4 on elastic second-order analysis includes new equations (10-8) and (10-9), which provide more-refined values of *EI* considering axial load, eccentricity, reinforcement ratio, and concrete compressive strength, as presented in the two Khuntia and Ghosh *ACI Structural Journal* articles.[7,8]

Commentary section R10.13.8, "Tie Reinforcement around Structural Steel Core," which was section R10.16.8 in ACI 318-05, used to state:

> Concrete that is laterally confined by tie bars is likely to be rather thin along at least one face of a steel core section. Therefore, complete interaction between the core, the concrete, and any longitudinal reinforcement should not be assumed. Concrete will probably separate from smooth faces of the steel core. To maintain the concrete around the structural steel core, it is reasonable to require more lateral ties than needed for ordinary reinforced concrete columns. Because of probable separation at high strains between the steel core and the concrete, longitudinal bars will be ineffective in stiffening cross sections even though they would be useful in sustaining compression forces.

This text has now been replaced with, "Research has shown that the required amount of tie reinforcement around the structural steel core is sufficient for the longitudinal steel bars to be included in the flexural stiffness of the composite column."[9]

Chapter 11: "Shear and Torsion"

The revisions to achieve a consistent treatment of lightweight concrete throughout ACI 318 (see discussion of section 8.6 in "Significant Changes to ACI 318-08 Relative to Precast/Prestressed Concrete: Part 1"[10]) have led to the deletion of section 11.2 of ACI 318-05. The revisions to ACI 318 also affect several of the equations in chapter 11. Those equations are found in sections 11.2, "Shear Strength Provided by Concrete for Nonprestressed Members"; 11.3, "Shear Strength Provided by Concrete for Prestressed Members"; 11.5.1, "Threshold Torsion"; 11.5.2, "Calculation of Factored Torsional Moment"; 11.9, "Provisions for Walls"; and 11.11, "Provisions for Slabs and Footings".

In addition, in section 11.6.4.3 (11.6 is the section on shear friction), $\lambda = 0.85$ for sand-lightweight concrete was changed to "Otherwise, λ shall be determined based on volumetric proportions of lightweight and normalweight aggregates as specified in [section] 8.6.1, but shall not exceed 0.85." Although the equations in the sections noted previously have different appearances, there have not been any significant changes related to the shear strength of structural members made of lightweight concrete.

Significant changes were made to the list of members in section 11.4.6.1 for which minimum shear reinforcement is not required where V_u exceeds $0.5\varphi V_c$. Solid slabs, footings, and joists are excluded from the minimum shear-reinforcement requirement because there is a "possibility of load sharing between weak and strong areas." Section 11.4.6.1, under item (a), has now clarified that the slabs must be solid. Based on experimental evidence,[11] a new limit on the depth of hollow-core units was established in item (b) of section 11.4.6.1.

"Research has shown that deep, lightly reinforced one-way slabs and beams, particularly if constructed with high-strength concrete, or concrete with a small coarse aggregate size, may fail at shear demands less than V_c computed using Eq. (11-3) especially when subjected to concentrated loads."[12–14] Because of this, "the exclusion for certain beam types in 11.4.6.1(e) is restricted to cases in which the total depth h does not exceed 24 in."

Commentary section R11.4.6.1 further advises that "for beams where f_c' is greater than 7000 psi, consideration should be given to providing minimum shear reinforcement when h is greater than 18 in. and V_u is greater than $0.5\varphi V_c$." The new exception in item (f) in section 11.4.6.1 provides a design alternative to the use of shear reinforcement, as defined in section 11.4.1.1, for members with longitudinal flexural reinforcement in which f_c' does not exceed 6000 psi, h is not greater than 24 in., and V_u does not exceed $\varphi 2\sqrt{f_c'}\, b_w d$. Fiber-reinforced concrete beams with hooked or crimpled steel fibers in dosages greater than or equal to 100 lb/yd^3 (59 kg/m^3) "have been shown through laboratory tests to exhibit shear strengths larger than $3.5\sqrt{f_c'}\, b_w d$."[15] Commentary section R11.4.6.1(f) points out that the use of steel fibers as shear reinforcement is not recommended when corrosion of fiber reinforcement is of concern.

In section 11.6.5, the upper limit on the nominal shear-friction strength V_n was significantly increased for both monolithically placed concrete and concrete placed against intentionally hardened concrete. Commentary section R11.6.5 points out that the increase is justified in view of test data.[16,17] Section 11.6.5 now clarifies that if a lower-strength concrete is cast against a higher-strength concrete, the value of f_c' used to evaluate V_n must be the f_c' for the lower-strength concrete. The increase in the upper limit on the nominal shear-friction strength is also reflected in section 11.8.3.2.1 (part of section 11.8, "Provisions for Brackets and Corbels").

One of the most significant changes in chapter 11 is the addition of code requirements to permit the use of headed stud assemblies as shear reinforcement in slabs and footings (section 11.11.5). "Using shear stud assemblies, as shear reinforcement in slabs and footings, requires specifying the stud shank diameter, the spacing of the studs, and the height of the assemblies for the particular applications" (**Fig. 1**). Tests[18] have shown that "vertical studs mechanically anchored as close as possible to the top and bottom of slabs are effective in resisting punching shear. . . . Compared with a leg of a stirrup having bends at the ends, a stud head exhibits smaller slip, and thus results in smaller shear crack widths. The improved performance results in larger limits for shear strength and spacing between peripheral lines of headed shear stud reinforcement."

Both the amount of shear assigned to the concrete V_c and the nominal shear strength $V_n = V_c + V_s$ are permitted to be larger for headed stud assemblies than for other forms of slab or footing shear reinforcement at $3\sqrt{f_c'}\, b_o d$ and $8\sqrt{f_c'}\, b_o d$, respectively. Section 11.11.5.1 clarifies that in the calculation of $V_d = A_v f_{yd}/s$, A_v is equal to the "cross-sectional area of all the shear reinforcement on one peripheral line that is approximately parallel to the perimeter of the column section, where s is the spacing of the peripheral lines of headed shear stud reinforcement."

Commentary section R11.11.5.1 clarifies that "when there is unbalanced moment transfer, the design must be based on stresses. The maximum shear stress due to a combination of V_u and the fraction of unbalanced moment $\gamma_v M_u$ should not exceed φv_n, where v_n is taken as the sum of $3\lambda\sqrt{f_c'}$ and $A_v f_{yt}/(b_o s)$."

"The specified spacings between peripheral lines of shear reinforcement [**Fig. 2**] are justified by experiments."[18] Commentary section R11.11.5.2 cautions that the "clear spacing between the heads of the studs should be adequate to permit placing of the flexural reinforcement."

Chapter 12: "Development and Splices of Reinforcement"

A new section 12.1.3 was added. The section specifically calls designers' attention to the structural-integrity requirements in section 7.13. There was concern within ACI Committee 318 that many designers were simply not aware of these requirements, though they have existed since the 1989 edition of ACI 318.

In all of the equations for development length of deformed bars and deformed wire in tension and compression, in sections 12.2.2 and 12.2.3, respectively, the lightweight-aggregate factor λ was moved from the numerator to the denominator. At the same time, in section 12.2.4(d), λ = 1.3 was replaced by "λ shall not exceed 0.75 unless f_{ct} is specified (see [section] 8.6.1)." All of this is consistent with the definition of λ in section 8.6 and is explained clearly in commentary section R12.2.4.

Before ACI 318-08, Eq. (12-2) for K_{tr} included the yield strength of the transverse reinforcement f_{yt}. The current expression assumes that f_{yt} = 60 ksi (414 MPa) and includes only the area and the spacing of the transverse reinforcement and the number of bars being developed or lap spliced. This is because tests have shown that transverse reinforcement rarely yields during bond failure.

By far the most significant change in chapter 12 is the introduction of section 12.6, "Development of Headed and Mechanically Anchored Deformed Bars in Tension." The use of headed deformed bars is attractive as an alternative to hooked bar anchorages in regions where reinforcement is heavily congested.

The term *development*, as used in section 12.6, indicates that "the force in the bar is transferred to the concrete through a combination of a bearing force at the head and bond forces along the bar." The term *anchorage*, as used in section 12.6, indicates that "the force in a bar is transferred to the concrete through bearing of the head alone."

Commentary section R12.6 states that the "provisions for headed deformed bars were written with due consideration of the provisions for anchorage in Appendix D and the bearing strength provisions of [section] 10.4.[19,20] Appendix D contains provisions for headed anchors related to the individual failure modes of concrete breakout, side-face blowout, and pullout, all of which were considered in the formulation of [section] 12.6.2. The restriction that the concrete must be normalweight, the maximum bar size of no. 11, and the upper limit of 60,000 psi on f_y are based on test data.[21]

Commentary Fig. R12.6(a) shows the length of headed deformed bar l_{dt} measured from the critical section to the bearing face of the head, which is given in section 12.6.2 for developing headed deformed bars.

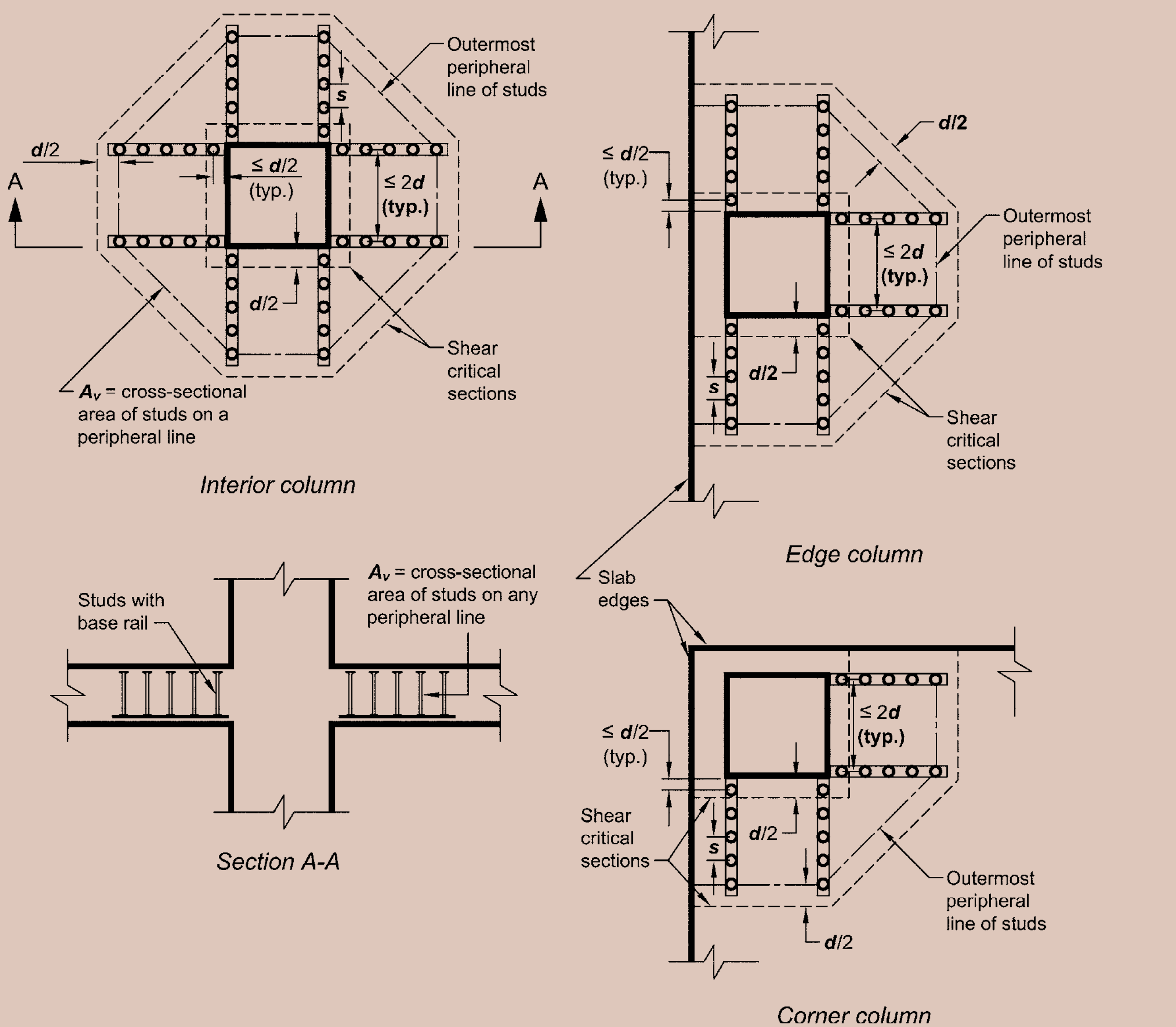

Figure 2. Typical arrangements are shown for headed shear-stud reinforcement and critical sections. Reproduced with permission from ACI 318-08 Figure R11.11.5.

"For bars in tension, heads allow the bars to be developed in a shorter length than required for standard hooks.[19–21] The minimum limits on clear cover, clear spacing, and head size are based on the lower limits on these parameters used in the tests to establish the expression for l_{dt} in [section] 12.6.2. … Headed bars with $A_{brg} < 4A_b$ have been used in practice, but their performance may not be accurately represented by the provisions of [section] 12.6.2." Headed bars may be used only in compliance with the requirements of section 12.6.4.

"A factor of 1.2 is consistently used for epoxy-coated headed reinforcing bars, the same value as used for epoxy-coated standard hooks." The upper limit of 6000 psi on the value of f_c' in section 12.6.2 is based on the concrete strengths used in the Texas tests.[19–21]

"Because transverse reinforcement was shown to be largely ineffective in improving the anchorage of headed deformed bars, additional reductions in development length … are not used for headed deformed reinforcing bars." The sole exception to this is a reduction for excess reinforcement.

Commentary section R12.6 indicates that where longitudinal headed deformed bars from a beam or slab terminate at a column or other supporting member, as shown in Figure R12.6(b), "the bars should extend through the joint to the far face of the supporting member, allowing for cover and avoiding interference with column reinforcement, even though the resulting anchorage length exceeds l_{dt}." Extending the bar to the far face of the supporting member improves the performance of the joint.

Section 12.6.3 requires that heads "not be considered effective in developing bars in compression" because there are no available test data demonstrating that "the use of heads adds significantly to anchorage strength in compression."

In section 12.8, Eq. (12-3) for the development length of plain welded-wire reinforcement in tension now shows the lightweight-aggregate factor λ in a position that is consistent with its definition in section 8.6. In section 12.13, the expression for the embedment length of web reinforcement between the midheight of a member and outside end of the hook now contains λ. In section 12.15, "Splices of Deformed Bars and Deformed Wire in Tension," section 12.15.3 was added. The section states that "when bars of different size are lap spliced in tension, splice length shall be the larger of l_d of larger bar and tension lap splice length of smaller bar."

Chapter 13: "Two-Way Slab Systems"

Section 13.2.6 has also been added. This section states that "when used to increase the critical concrete section for shear at a slab-column joint, a shear cap shall project below the slab and extend a minimum horizontal distance from the face of the column that is equal to the thickness of the projection below the slab soffit."

In section 13.3, what used to be called *special reinforcement* is now called *corner reinforcement*. Corner reinforcement is now required "at exterior corners of slabs supported by edge walls or where one or more edge beams have a value of α_f greater than 1.0."

New, useful commentary is provided in section R13.3.6. In section 13.3.8.5, *column core* was replaced by *region bounded by the longitudinal reinforcement of the column*.

Section 13.5.3.3 on transfer of unbalanced moments to columns was editorially rewritten for clarity. Two substantive changes have also been made. The limit of 37.5% of the balanced steel ratio on the amount of reinforcement within the effective slab width was updated to refer to a minimum net tensile strain of 0.010 to be consistent with the unified design approach, and the requirement for the minimum net tensile strain was eliminated for moment transfer about the slab edge for edge and corner connections based on the original recommendation from ACI Committee 352.

New commentary section R13.6.7 explains the moment redistribution of up to 10% that is permitted to occur in slabs that are analyzed using the direct design method.

Chapter 14: "Walls"

Section 14.3.7 used to read, "In addition to the minimum reinforcement required by 14.3.1, not less than two no. 5 bars shall be provided around all window and door openings. Such bars shall be extended to develop the bar beyond the corner of the openings but not less than 24 in."

The section now reads, "In addition to the minimum reinforcement required by 14.3.1, not less than two no. 5 bars in walls having two layers of reinforcement in both directions and one no. 5 bar in walls having a single layer of reinforcement in both directions shall be provided around window, door, and similar sized openings. Such bars shall be anchored to develop f_y in tension at the corners of the openings."

Section 14.8, "Alternative Design of Slender Walls," was introduced in the 1999 edition of ACI 318, and the provisions are based on similar provisions in the *Uniform Building Code* (UBC),[22] which in turn are based on experimental research.[23] Changes were made in the 2008 edition to reduce differences in the serviceability provisions between ACI 318 and the UBC to ensure that the intent of the UBC provisions is included in future editions of the IBC.

Before the 2008 edition, under section 14.8.3, the effective area of longitudinal reinforcement in a slender wall for obtaining an approximate cracked moment of inertia was calculated using an effective area of tension reinforcement defined by Eq. (14-7) without the *h*/2*d* modifier. The contribution of the axial load to the cracked moment of inertia was overestimated in many cases where two layers of reinforcement were used in a slender wall. The effective area of longitudinal reinforcement was modified in 2008 by introducing the *h*/2*d* modifier. "The neutral axis depth *c* in Eq. (14-7) corresponds to this effective area of longitudinal reinforcement."

Section 14.8.4 has undergone significant changes. Prior to this edition, out-of-plane deflections of wall panels were calculated using the effective moment of inertia given in section 9.5.2.3. "However, reevaluation of the original test data[23] indicated that out-of-plane deflections increase rapidly when the service-level moment exceeds $2/3M_{cr}$. A linear interpolation between Δ_{cr} [given by Eq. (14-10)] and Δ_n [given by equation (14-11)] is used to determine Δ_s to simplify the design of slender walls" if the maximum moment in member due to service loads M_a exceeds $2/3M_{cr}$.

Commentary section R14.8.4 states that "service-level load combinations are not defined in Chapter 9 of ACI 318." However, they are discussed in appendix C of ASCE/SEI 7-05, although, unlike ACI 318, "appendixes to ASCE/SEI 7 are not considered to be mandatory parts of the standard." Appendix C of ASCE 7-05 recommends the following load combination for calculating service-level lateral deflections of structures:

$$D + 0.5L + 0.7W$$

"which corresponds to a 5% annual probability of exceedance."

According to section R14.8.4, "if a slender wall is designed to resist earthquake effects, *E*, and *E* is based on strength-level seismic forces," a conservative estimate of service-level seismic forces is 0.7*E*.

Chapter 15: "Footings"

An important new section 15.10.4 was added, stating unequivocally that the minimum reinforcing steel in nonprestressed mat foundations shall meet the requirements of section 7.12.2 in each principal direction and that the maximum spacing shall not exceed 18 in. The new commentary section R15.10.4 also supplies important clarification. It states that "minimum reinforcing steel may be distributed near the top or bottom of the section, or may be allocated between the two faces of the section as deemed appropriate for specific conditions, such that the total area of continuous reinforcing steel satisfies [section] 7.12.2."

Chapter 16: "Precast Concrete"

No substantive changes were made to this chapter.

Chapter 17: "Composite Concrete Flexural Members"

No change was made to this chapter.

Chapter 18: "Prestressed Concrete"

One important change in section 18.4.1 permits an increase in the allowable concrete compressive stress immediately after prestress transfer at the ends of pretensioned, simply supported members from $0.60f'_{ci}$ to $0.70f'_{ci}$. This change was made based on research results and common practice in the precast/prestressed concrete industry. The remainder of section 18.4.1 was editorially rewritten.

Section 18.8.2 on minimum flexural reinforcement was editorially rewritten.

A sentence added to commentary section R18.8.2 points out that the requirement of section 18.8.2 does not apply to members with unbonded tendons because the "transfer of force between the concrete and the prestressing steel, and abrupt flexural failure immediately after cracking, does not occur when prestressing steel is unbonded."[24]

Changes were made in section 18.10.4 on redistribution of moments in continuous prestressed flexural members, which are very similar to the corresponding changes made in section 8.4 on redistribution of moments in continuous nonprestressed flexural members.

Section 18.12.4 provides "specific guidance concerning tendon distribution that will permit the use of banded tendon distributions in one direction. . . . The minimum average effective prestress of 125 psi was used in two-way test panels in the early 1970s to address punching shear concerns in lightly reinforced slabs."

A sentence was added to clearly indicate that if the slab thickness varies along or perpendicular to the span of a slab "resulting in a varying slab cross section, the 125 psi minimum effective prestress and the maximum tendon spacing are required at every cross section tributary to the tendon or group of tendons along the span, considering both the thinner and the thicker slab sections."

There are significant modifications of the requirements for structural integrity steel in two-way, unbonded, post-tensioned slab systems, previously in section 18.12.4, now in sections 18.12.6 and 18.12.7.

Section 18.12.6 requires that "in slabs with unbonded tendons, a minimum of two ½-in. diameter or larger, seven-wire post-tensioned strands shall be provided in each direction at columns, either passing through or anchored within the region bounded by the longitudinal reinforcement of the column." Such reinforcement provided at any location over the depth of the slab suspends the slab following a punching shear failure, provided the tendons "are prevented from bursting through the top surface of the slab."[25]

"Where the two structural integrity tendons are anchored within the region bounded by the longitudinal reinforcement of the column," the anchorage is required to be located beyond the column centroid and away from the anchored span. "Outside column and shear cap faces, these two structural integrity tendons are required to pass under any orthogonal tendons in adjacent spans" so that vertical movements of the integrity tendons are restrained by the orthogonal tendons.

"Where tendons are distributed in one direction and banded in the orthogonal direction, this requirement can be satisfied by first placing the integrity tendons for the distributed tendon direction and then placing the banded tendons." Commentary section R18.12.6 states that "where tendons are distributed in both directions, weaving of tendons is necessary and the use of [section] 18.12.7 may be an easier approach." That section allows the structural integrity tendons to be replaced by deformed-bar bottom reinforcement.

In section 18.13.4 (section 18.13 is on "Post-tensioned Tendon Anchorage Zones"), *nominal tensile stress* was changed to *tensile stress at nominal strength*, *nominal compressive strength of concrete* to *compressive stress in concrete at nominal strength*, and *design drawings* to *contract documents*.

Chapter 19: "Shells and Folded Plate Members"

Section 19.4.2, instead of providing values of the coefficient of friction μ, now refers to section 11.6.4.3 for those values.

A few other changes, which are essentially editorial, were made in the chapter.

Chapter 20: "Strength Evaluation of Existing Structures"

Since the 1995 edition of ACI 318, section 20.2.3 referenced section 5.6.5 for determining concrete strength from cores when evaluating the strength of an existing structure. However, section 5.6.5 was developed for investigating low-strength test results, not evaluating the strength of existing structures. ACI Committee 214 has developed procedures for estimating an equivalent f_c' from core test data. The requirements of section 20.2.3 were changed to require an estimate of an equivalent f_c' and the commentary references the ACI 214.4R-03 methods.[26]

The test load intensity in section 20.3.2 of ACI 318-05, 0.85 ($1.4D + 1.7L$), was not changed when the ASCE/SEI 7 load combinations were brought into the main body of ACI 318-02 because Committee 318 did not want to reduce the fundamental level of structural safety. However, the ACI 318-05 format was confusing to practitioners because it appeared to refer only to the traditional ACI load combinations, which are now in appendix C. Also, test load combinations including snow and rain loads were not provided. To correct these deficiencies without substantially changing the test load intensity, the required test load intensity was revised to be not less than the largest value given by three different load combinations.

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Notation

A_b = area of an individual bar or wire

A_{brg} = net bearing area of the head of stud, anchor bolt, or headed deformed bar

A_g = gross area of concrete section

A_v = area of shear-reinforcement within spacing s

b_o = perimeter of critical section for shear in slabs and footings

b_w = web width or diameter of circular section

c = distance from extreme compression fiber to neutral axis

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

D = dead loads or related internal moments and forces

E = load effects of earthquake or related internal moments and forces

EI = flexural stiffness of compression member

f_y = specified yield strength of nonprestressed reinforcement

f_{yt} = specified yield strength of transverse reinforcement

f'_c = specified compressive strength of concrete

f'_{ci} = specified compressive strength of concrete at time of initial prestress

h = overall thickness or height of member

I = moment of inertia of section about centroidal axis

k = effective length factor for compression members

K_{tr} = transverse reinforcement index

l_d = development length in tension of deformed bar, deformed wire, plain and deformed welded-wire reinforcement, or pretensioned strand

l_{dt} = development length in tension of headed deformed bar, measured from the critical section to the bearing face of the head

l_u = unsupported length of compression member

L = live loads or related internal moments and forces

M_1 = smaller factored end moment on a compression member

M_2 = larger factored end moment on a compression member

M_a = maximum moment in member due to service loads

M_{cr} = cracking moment

M_u = factored moment at section

r = radius of gyration of cross section of a compression member

s = center-to-center spacing of items, for example, spacing of the peripheral lines of headed shear-stud reinforcement

V_c = nominal shear strength provided by concrete

V_d = shear force at section due to unfactored dead load

V_n = nominal shear strength

V_s = nominal shear strength provided by shear reinforcement

V_u = factored shear force at section

α_f = ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panes (if any) on each side of the beam

γ_v = factor used to determine the unbalanced moment transferred by eccentricity of shear at slab-column connections

Δ_{cr} = computed out-of-plane deflection at midheight of wall corresponding to cracking moment

Δ_n = computed out-of-plane deflection at midheight of wall corresponding to nominal flexural strength

Δ_s = computed out-of-plane deflection at midheight of wall due to service loads

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete

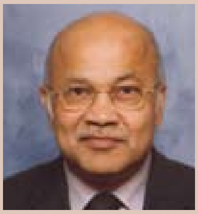
μ = coefficient of friction

ϕ = strength-reduction factor

v_n = nominal shear-stress capacity

$= 3\lambda\sqrt{f'_c} + A_v f_{yt}/(b_o s)$

About the author



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Synopsis

Significant changes were made since American Concrete Institute (ACI) Committee 318 published the 2005 *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)*. The changes in the upcoming 2008 edition are summarized here. In addition to changes affecting conventionally reinforced concrete, provisions affecting precast/prestressed concrete, including post-tensioned concrete, are enumerated. Only changes to chapters 9 through 20 of ACI 318-08 are discussed in this article.

Keywords

ACI 318, code, structural concrete.

Reader comments

Please address any reader comments to *PCI Journal* editor-in-chief Emily Lorenz at elorenz@pci.org or Precast/Prestressed Concrete Institute, c/o *PCI Journal*, 209 W. Jackson Blvd., Suite 500, Chicago, IL 60606.