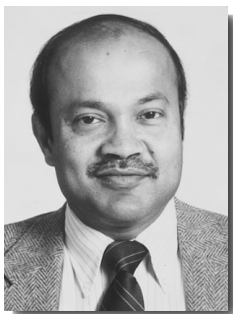


Significant Changes in the 2005 ACI Code Including Changes Affecting Precast/Prestressed Concrete – Part 1



S. K. Ghosh, Ph.D., FPCI

President
S. K. Ghosh Associates, Inc.
Palatine, Illinois

Significant changes made since the publication of the 2002 ACI 318 Building Code, which are reflected in the upcoming 2005 edition of the Code, are summarized. In addition to changes impacting conventionally reinforced concrete, provisions affecting precast/prestressed concrete including post-tensioned concrete are enumerated. Changes in Appendix D, Anchoring to Concrete, are not discussed in this Part 1 article.

The 2005 edition of the American Concrete Institute's Building Code Requirements for Structural Concrete (ACI 318-05) is in the final stages of preparation. The significant changes from the previous edition of the ACI Code (ACI 318-02) are summarized in this article.

The complete changes were published in the July 2004 issue of ACI's *Concrete International*.¹ They were also posted on the ACI website until August 31, 2004, when the public comment period ended. Pertinent discussion received by the deadline of August 31, 2004 will be published in a future issue of *Concrete International*.

ACI Committee 318 is required to respond in writing to all the discussion that is submitted. In the process of responding to public comments, the Committee may decide to make modifications to the published changes. However, major changes are not anticipated at this stage.

The intent of this article is to provide a summary of significant changes impacting 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.

ACI 318-05 will be the reference document for concrete design and construction in the 2006 edition of the *International Building Code*,² Supplement No. 1 to the 2005 edition of the ASCE 7 Standard *Minimum Design Loads for Buildings and Other Structures*,³ and the second (2006) edition of the NFPA 5000 *Building Construction and Safety Code*,⁴ issued by the National Fire Protection Association.

All section numbers refer to the 2005 Code, unless otherwise noted. In the following paragraphs, strike-out marks indicate deletion of existing (ACI 318-02) text, and underlining indicates addition of new text. Chapters

not discussed do not have any changes in them. Changes in Appendix D, Anchoring to Concrete, will be discussed in Part 2 of this paper, to appear in the next issue of the PCI JOURNAL.

Change of Notation

Perhaps the most important change in ACI 318-05 is a thorough clean-up of the notation used in the Code. A Notation and Terminology Task Group was formed within ACI 318 under the leadership of Sharon Wood to review ACI 318-02 and develop a unified set of notation. The feeling was almost universal that the Code will be easier to use if the notation is consistent throughout. The Task Group identified six specific tasks:

1. Consolidate similar terms as appropriate.
2. Eliminate unnecessary terms.
3. Provide a unique definition of each term used in ACI 318-02.
4. Move prescriptive requirements from the list of notation into the Code.
5. Use notation, rather than text, whenever possible in the Code.
6. Move the list of notation from Appendix E to Chapter 2.

The Task Group effort resulted in the following changes:

1. Notation has been consolidated. Four hundred and six terms were included in Appendix E of ACI 318-02, while ACI 318-05 includes 305 terms.
2. Duplicate definitions of terms are eliminated. In some cases, the definitions for terms were slightly different in different chapters. In those cases, the most general definition is given in the list of notation and the definition is clarified in the Code, as needed.
3. All terms related to stress in reinforcement are expressed in units of psi. The applicable equations have been modified.
4. The list of notation at the beginning of each chapter has been deleted in ACI 318-05.
5. Most of the notation-related changes within the Code are editorial in nature and are motivated by the objective to use notation, rather than text, within the Code provisions.
6. The Task Group did not review the notation in the Commentary in detail, and the list of Commentary nota-

tion is incomplete. However, a few changes have been made to be consistent with the notation in the Code and to eliminate duplicate definitions.

Change in Terminology

“Welded wire fabric” is now called “welded wire reinforcement” throughout the Code. This has given rise to a large number of editorial changes throughout ACI 318-05.

Chapter 2, Definitions

The definition of structural lightweight concrete has been revised so that it refers to “equilibrium density,” as specified in ASTM C 567-00,⁵ and uses the correct title of C 567.

The definition and required dimensions of a drop panel are currently given within Section 13.3 – Slab Reinforcement. Users of the Code cannot find this definition easily. The primary drop panel definition has been moved to Chapter 2, where it is defined as a projection below the slab at least one-quarter the slab thickness beyond the drop. The additional dimensional requirements are given in Chapter 13, in a new Section 13.2.5.

Development length is now defined as “length of embedded reinforcement, including pretensioned strand, required to develop...”

Transfer length has been newly defined as length of embedded pretensioned strand required to transfer the effective prestress to the concrete.

Chapter 3, Materials

A new paragraph has been added at the beginning of Commentary Section 3.5.1 pointing out that Fiber Reinforced Polymer (FRP) reinforcement is not addressed in this Code and that ACI Committee 440 has developed guidelines for the use of FRP Reinforcement.^{6,7}

The referenced standards listed in Section 3.8.1 have been updated.

Sections 2.3.3 (Load Combinations Including Flood Loads) and 2.3.4 (Load Combinations Including Atmospheric Ice Loads) of SEI/ASCE 7-02 Standard *Minimum Design Loads for Buildings and Other Structures*³ are declared to be part of ACI 318-05.

The 17th, rather than the 16th, edition of AASHTO Standard Specifications, dated 2002,⁸ are partially adopted in Section 3.8.5.

The 2004, rather than the 2002, edition of ACI 355.2 *Qualification of Post-Installed Mechanical Anchors in Concrete*⁹ is adopted in Section 3.8.6.

The 2002, rather than the 2000 edition of AWS D1.1 *Structural Welding Code – Steel*¹⁰ has been adopted in Section 3.8.7.

Chapter 5, Concrete Quality, Mixing, and Placing

When an acceptable record of field test results is not available, concrete proportions established from trial mixtures meeting certain restrictions are permitted in Section 5.3.3.2. The first restriction has been modified to read: “~~Combinations of~~ Materials shall be those for proposed work.” This is to clarify original intent.

Chapter 6, Formwork, Embedded Pipes, and Construction Joints

Section 6.4.4 of ACI 318-02 read: “Construction joints in floors shall be located within the middle third of spans of slabs, beams, and girders. Joints in girders shall be offset a minimum distance of two times the width of intersecting beams.” The two sentences have now been placed in separate Sections 6.4.4 and 6.4.5, so that Section 6.4.4 (not 6.4.5) can be waived for prestressed concrete construction in Chapter 18.

Chapter 9, Strength and Serviceability Requirements

Commentary Sections R9.1 and R9.2 now make references to SEI/ASCE 7-02,³ rather than to ASCE 7-98.

Section 9.2.4 has been modified as follows: “~~For~~ If a structure is in a flood zone, or is subjected to forces from atmospheric ice loads, the flood or ice loads and the appropriate load combinations of SEI/ASCE 7 shall be used.”

The expressions contained in Fig. R9.3.2 of ACI 318-02 for interpolation of ϕ within ϵ_t values of 0.002 and 0.005 did not produce accurate values,

according to some code users. Editorial changes have been made to modify the expressions to solve the inaccuracy.

The ϕ of 0.75 in ACI 318-02 for flexural sections in pretensioned members where strand embedment is less than the development length, as provided in Section 12.9.1.1, is now applicable from the end of the member to the end of the transfer length. From the end of the transfer length to the end of the development length, ϕ may be linearly increased from 0.75 to 0.9. Where bonding of a strand does not extend to the end of the member, strand embedment begins at the end of the debonded length. See also Section 12.9.3 and Fig. R.9.3.2.7(a).

The following second paragraph has been added to Commentary Section R.9.3.2.7:

Where bonding of one or more strands does not extend to the end of the member, in lieu of a more rigorous analysis, ϕ may be conservatively taken as 0.75 from the end of the member to the end of the transfer length of the strand with the largest debonded length. Beyond this point, ϕ may be varied linearly to 0.9 at the location where all strands are developed, as shown in Fig. R9.3.2.7(b). Alternatively, the contribution of the debonded strands may be ignored until they are fully developed. Embedment of debonded strand is considered to begin at the termination of the debonding sleeves. Beyond this point, the provisions of Section 12.9.3 are applicable.

Confinement reinforcement often creates congestion in reinforced concrete members. Research has shown^{11,12} that reinforcement with a yield strength up to 100,000 psi can be used for confinement, without any detriment to member performance. Spiral reinforcement with specified yield strength up to 100,000 psi is, therefore, permitted by Section 10.9.3 of ACI 318-05. Section 9.4 has accordingly been modified as follows: “The values of f_y and f_{yt} used in design calculations shall not exceed in excess of 80,000 psi except for prestressing steel and for spiral transverse reinforcement in 10.9.3.”

Chapter 10, Flexure and Axial Loads

The axial load limit of $0.10f'_cA_g$ in Section 10.3.5 is clarified to be a limit on factored axial compression load.

Commentary Section R10.6.1 has been rewritten in places to provide editorial clarification.

The maximum spacing of reinforcement closest to the tension force, for purposes of crack control, is given by:

$$s = 15 \left(\frac{40,000}{f_s} \right) - 2.5c_c \leq 12 \left(\frac{40,000}{f_s} \right)$$

with f_s in psi, whereas in ACI 318.02 it was given by:

$$s = \frac{540}{f_s} - 2.5c_c \leq 12 \left(\frac{36}{f_s} \right)$$

with f_s in ksi units. This change reflects the higher service stresses that occur in flexural reinforcement with the use of the load combinations introduced in the 2002 Code.

Note that f_s is permitted to be taken equal to $(2/3)f_y$, rather than 60 percent of f_y , as in ACI 318-02.

The crack control provisions were updated to reflect the higher service stresses that occur in flexural reinforcement with the use of the load combinations introduced in ACI 318-02.

Section 10.6.7 on skin reinforcement in deep members has been modified as follows:

“If the effective depth d of a beam or joist exceeds 36 in., longitudinal skin reinforcement shall be uniformly distributed along both side faces of the members. Skin reinforcement shall extend for a distance $d/2$ nearest $h/2$ from the tension face. nearest the flexural tension reinforcement. The spacing s ~~the spacing s_{sk} between longitudinal bars or wires of the skin reinforcement~~ shall be as provided in 10.6.4, where c_c is the least distance from the surface of the skin reinforcement or prestressing steel to the side face, not exceed the least of $d/6$, 12 in., and $1000A_p/(d-30)$. It shall be permitted to include such reinforcement in strength computations if a strain compatibility analysis is made to determine stress in the individual bars or wires. ~~The total area of longitudinal skin reinforcement in both faces need not ex-~~

ceed one-half of the required flexural tensile reinforcement.”

The changes in Section 10.6.7 are intended to simplify the crack control provisions for skin reinforcement and make these provisions consistent with those required for flexural tension reinforcement. The size of skin reinforcement is not specified; research¹³ has indicated that the spacing rather than bar size is of primary importance.

As indicated earlier, Section 10.9.3 has been modified to permit the use of spiral reinforcement with specified yield strength of up to 100,000 psi. For spirals with f_{yt} greater than 60,000 psi, only mechanical or welded splices may be used.

Section 10.13.6 requires that in addition to load combinations involving lateral loads, the strength and stability of the structure as a whole under factored gravity heads must be considered. In Items (a) and (b) of that section, “1.4 dead load and 1.7 live load” of ACI 318-02 has been replaced by “factored dead and live loads” in ACI 318-05, thus supplying a much-needed clarification.

Chapter 11, Shear and Torsion

A change of much significance to the precast concrete industry is that an alternative design procedure for torsion design has been introduced in Section 11.6.7, which more realistically addresses L-shaped beams. Design for torsion now must be in accordance with Section 11.6.1 through 11.6.6, or 11.6.7. The design for torsion in Sections 11.6.1 through 11.6.6 is based on a thin-walled tube, space truss analogy.

Section 11.6.7, titled “Alternative design for torsion,” states: “For torsion design of solid sections within the scope of the Code with an aspect ratio, h/b_t (h = overall thickness or height of member, b_t = width of that part of cross section containing the closed stirrups resisting torsion), of three or greater, it shall be permitted to use another procedure, the adequacy of which has been shown by analysis and substantial agreement with results of comprehensive tests. Sections 11.6.4 (Details of torsional reinforcement) and 11.6.6 (Spacing of torsion rein-

forcement) apply.” Commentary Section R11.6.7 states that examples of such procedures are to be found in References 14, 15, and 16, which have been extensively and successfully used for design of precast prestressed concrete beams with ledges.

The procedure described in References 14 and 15 is an extension to prestressed concrete sections of the torsion design procedures of pre-1995 editions of ACI 318. The fourth edition of the PCI Design Handbook¹⁷ describes the procedure of References 14 and 15. The procedure was experimentally verified by the tests described in Reference 18.

Section 11.6.4.2, which requires transverse torsional reinforcement to be anchored in ways indicated by Items (a) or (b), has had Item (a) modified as follows: “(a) A 135-deg standard hook or seismic hook, as defined in 21.1, around a longitudinal bar;”

Chapter 13, Two-Way Slab Systems

A new Section 13.2.5 prescribes the dimensional requirements for drop panels that were in ACI 318-02 Sections 13.3.7.1 and 13.3.7.2, but makes them applicable only when the drop panel is used to reduce the amount of negative reinforcement over a column or minimum required slab thickness. A new Commentary Section R13.2.5 points out that drop panels with dimensions less than those specified in 13.2.5 may be used to increase slab shear strength.

Chapter 14, Walls

The ϕ in Eq. (14-1), giving the design axial load strength of a wall eligible to be designed by the empirical design method, was 0.7 in ACI 318-02. Now the same ϕ must correspond to compression-controlled sections in accordance with Section 9.3.2.2. This is for consistency with Chapter 9.

For similar reasons, under Section 14.8, Alternative design of slender walls, the previous requirement that the reinforcement ratio should not exceed $0.6\rho_{bal}$ was replaced by the requirement that the wall be tension-controlled, leading to approximately the same reinforcement ratio.

Chapter 15, Footings

An important clarification of Section 15.5.3 has been provided by replacing “Other pile caps shall satisfy one of 11.12, 15.5.4, or Appendix A” with “Other pile caps shall satisfy either Appendix A, or both 11.12 and 15.5.4.” Section 15.5 deals with shear design of footings.

Chapter 18, Prestressed Concrete

Tendons of continuous post-tensioned beams and slabs are usually stressed at a point along the span where the tendon profile is at or near the centroid of the concrete cross section. Therefore, interior construction joints are usually located within the end thirds of the span, rather than the middle third of the span, as required by Section 6.4.4. This has had no known detrimental effect on the performance of such beams. Thus, Section 6.4.4 is now excluded from application to prestressed concrete.

ACI 318-02 required prestressed two-way slab systems to be designed as Class U, which meant that f_t could be up to $7.5\sqrt{f'_c}$. ACI 318-05 restricts f_t in such slabs to $6\sqrt{f'_c}$, thus limiting the permissible flexural tensile stress in two-way prestressed slabs to the same value as in ACI 318-99 and prior codes. Section 18.4.4.4 has been modified as follows: “~~Where if h the effective depth~~ of a beam exceeds 36 in., the area of longitudinal skin reinforcement consisting of reinforcement or bonded tendons shall be provided as required by 10.6.7.”

In Commentary Section R18.10.3, the statement that for statistically indeterminate structures, the moments due to reactions induced by prestressing forces, referred to as secondary moments, are significant in both elastic and inelastic states is now supported by three added references.¹⁹⁻²¹ The sentence, “When hinges and full redistribution of moments occur to create a statically determinate structure, secondary moments disappear.” has been deleted. This removes an unnecessary and potentially confusing sentence.

Section 18.12.4 no longer refers to “normal live loads,” because it is largely meaningless.

Chapter 21, Special Provisions for Seismic Design

A new term, design story drift ratio, is defined as the relative difference of design displacements between the top and the bottom of a story, divided by the story height. This is part of a change in Section 21.11 that is discussed later.

As mentioned earlier, Sections 9.4 and 10.9.3 have been modified to allow the use of spiral reinforcement with specified yield strength of up to 100,000 psi. A sentence added to Section 21.2.5 specifically prohibits such use in members resisting earthquake-induced forces in structures assigned to Seismic Design Category D, E, or F. This is largely the result of some misgiving that high strength spiral reinforcement may be less ductile than conventional mild steel reinforcement and that spiral failure has in fact been observed in earthquakes. There are fairly convincing arguments, however, against such specific prohibitions. Spiral failure, primarily observed in bridge columns, have invariably been the result of insufficient spiral reinforcement, rather than the lack of ductility of the spiral reinforcement. Also, prestressing steel, which is the only high-strength steel available on this market, is at least as ductile as welded wire reinforcement, which is allowed to be used as transverse reinforcement.

Section 21.5.4 modifies the development length requirements of Chapter 12 for longitudinal beam bars terminating at exterior beam-column joints of structures assigned to high seismic design categories. But then Section 21.7.2.3 of ACI 318-02 required that all continuous reinforcement in structural walls be anchored or spliced in accordance with the provisions for reinforcement in tension in Section 21.5.4. Section 21.9.5.4 of ACI 318-02 further required that all continuous reinforcement in diaphragms, trusses, ties, chords, and collector elements be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in Section 21.5.4. Sections 21.7.2.3 and 21.9.5.4 were very confusing to the user, because Section 21.5.4 is really not applicable to situa-

tions covered by those sections. This problem existed with ACI 318 editions prior to 2002 as well.

In a very significant and beneficial change, the requirements of Section 21.7.2.3 were modified to remove the reference to beam-column joints in Section 21.5.4. Because actual forces in longitudinal reinforcement of structural walls may exceed calculated forces, it is now required that reinforcement in structural walls be developed or spliced for f_y in tension in accordance with Chapter 12. The effective depth of member referenced in Section 12.10.3 is permitted to be taken as $0.8l_w$ for walls. Requirements of Sections 12.11, 12.12, and 12.13 need not be satisfied, because they address issues related to beams and do not apply to walls.

At locations where yielding of longitudinal reinforcement is expected, $1.25f_y$ is required to be developed in tension, to account for the likelihood that the actual yield strength exceeds the specified yield strength, as well as the influence of strain-hardening and cyclic load reversals. Where transverse reinforcement is used, development lengths for straight and hooked bars may be reduced as permitted in Sections 12.2 and 12.5, respectively, because closely spaced transverse reinforcement improves the performance of splices and hooks subjected to repeated cycles of inelastic deformation. The requirement that mechanical splices of reinforcement conform to Section 21.2.6, and welded splices to Section 21.2.7, has now been placed in

Section 21.7.2.3. Consequently, Sections 21.7.6.4(f) of 21.7.6.6 of ACI 318-02 have been deleted.

In a companion change, Section 21.9.5.4 now requires that all continuous reinforcement in diaphragms, trusses, struts, ties, chords, and collector elements be developed or spliced for f_y in tension.

Structural truss elements, struts, ties, diaphragm chords, and collector elements with compressive stresses exceeding $0.2f'_c$ at any section are required to be specially confined by Section 21.9.5.3. The special transverse reinforcement may be discontinued at a section where the calculated compressive stress is less than $0.15f'_c$. Stresses are calculated for factored forces using a linear elastic model and gross-section properties of the elements considered.

In recent seismic codes and standards, collector elements of diaphragms are required to be designed for forces amplified by a factor Ω_0 , to account for the overstrength in the vertical elements of the seismic-force-resisting system. The amplification factor Ω_0 ranges between 2 and 3 for concrete structures, depending upon the document selected and on the type of seismic system. To account for this, Section 21.9.5.3 now additionally states that where design forces have been amplified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limits of $0.2f'_c$ and $0.15f'_c$ shall be increased to $0.5f'_c$ and $0.4f'_c$, respectively.

In a very significant change, provisions for shear reinforcement at slab-column joints have been added in a new Section 21.11.5, to reduce the likelihood of punching shear failure in two-way slabs without beams. A prescribed amount and detailing of shear reinforcement is required unless either Section 21.11.5(a) or (b) is satisfied.

Section 21.11.5(a) requires calculation of shear stress due to the factored shear force and induced moment according to Section 11.12.6.2. The induced moment is the moment that is calculated to occur at the slab-column joint where subjected to the design displacement defined in Section 21.1. Section 13.5.1.2 and the accompanying commentary provide guidance on the selection of slab stiffness for the purpose of this calculation.

Section 21.11.5(b) does not require the calculation of induced moments, and is based on research^{22,23} that identifies the likelihood of punching shear failure considering interstory drift and shear due to gravity loads. The requirement is illustrated in the newly added Fig. R21.11.5. The requirement can be satisfied in several ways: adding slab shear reinforcement, increasing slab thickness, designing a structure with more lateral stiffness to decrease interstory drift, or a combination of two or more of these factors.

If column capitals, drop panels, or other changes in slab thickness are used, the requirements of Section 21.11.5 must be evaluated at all potential critical sections.

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