

APPENDIX

IMPACT OF ACI 318-08 ON THIS HANDBOOK

The *PCI Design Handbook* 7th edition is based on the *Building Code Requirements for Structural Concrete* (ACI 318-05), *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-05), and the *International Building Code* (IBC-2006). The IBC-2006 is the code adopted by most jurisdictions having legal authority over building construction at the time of publication of this edition of the handbook. The IBC-2006 references ACI 318-05 and ASCE 7-05.

IBC-2009 was published in the spring of 2009 (IBC-2009) and references ACI 318-08 and ASCE 7-05. However, it is not clear how many jurisdictions will adopt IBC-2009 and if so, when. Many may choose to wait until IBC-2012 is released. Hence, if this handbook were to use IBC-2009 as its main model building code reference, some of its provisions may not be in conformance with the applicable codes in all jurisdictions.

It is desirable that this handbook present the most current information available with respect to the design of precast/prestressed concrete.

This Appendix is intended to provide the user of the handbook an easy reference to chapters in the handbook and how ACI 318-08 affects those chapters. This is done on a chapter by chapter basis. References identified within the text are references to ACI 318-08 unless specified otherwise.

Notation for the Appendix shown at the end of the section, includes only the notation used in this Appendix, and reflects notation used in ACI 318-08 unless noted otherwise.

The material presented is directly from four articles^{1,2,3,4} written by SK Ghosh and published in the *PCI Journal* as special supplements for members.

Chapter 1 – Precast and Prestressed Concrete: Applications

This chapter provides a general discussion of the applications of precast and prestressed concrete and is not affected by either ACI 318-08 or IBC 2009.

Chapter 2 – Notation and Definitions

A new definition for specified concrete cover has been added in Chapter 2. The commentary to this definition points out that tolerances on specified concrete cover are provided

in Section 7.5.2.1.

Design load combination is now defined in Chapter 2.

Important new definitions have been added in Chapter 2 for equilibrium density, headed deformed bars, and headed shear stud reinforcement. Commentary to the definition for headed deformed bars points out important differences between such bars and headed shear stud reinforcement.

Chapter 3 – Preliminary Design of Precast, Prestressed Concrete Structures

Chapter 3 presents a number of load tables for deck components and interaction curves for columns and wall panels. These may be influenced by changes in ACI 318-08 that are identified in the section in the appendix on Chapter 5. The user of the handbook should review the changes in Chapter 5 and use judgment in how it affects these load table and interaction diagrams.

Chapter 4 – Analysis and Design of Precast, Prestressed Concrete Structures

General

In Chapter 2, new definitions for *shear cap*, *steel fiber-reinforced concrete*, and *work* have been added.

A new Section 8.8, “Effective Stiffness to Determine Lateral Deflections,” has been added. Section 8.8.2 requires that the lateral deflections of reinforced-concrete building systems resulting from factored lateral forces be determined by either:

- a detailed analysis considering the reduced stiffness of all members under the loading conditions;
- a linear analysis using (a) section properties defined in Section 10.10.4(a) through (c) (Section 10.10 deals with slenderness effects in compression members) or (b) 50% of stiffness values based on gross section properties.

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 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).

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, “appendices 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.”

Seismic Related

Commentary Section R1.1.9, “Provisions for Earthquake Resistance” (formerly Section R1.1.8) has been completely rewritten with the introduction of SDC terminology into ACI 318. Table R1.1.9.1 (formerly Table 1.1.8.3), which provides correspondence between the new terminology and the terminology used in legacy model codes (seismic performance categories, seismic zones) and in prior editions of ACI 318 (regions of low, moderate, and high seismic risk), has been updated.

The term *seismic design category* is defined in Chapter 2.

Definitions for *special precast structural wall* and *special structural wall* were added. The term *special structural wall* is now defined, and it can be made of cast-in-place or precast concrete.

Design requirements for earthquake-resistant structures have been rewritten in terms of SDC. The prior terminology of regions of low, moderate, and high seismic risk is gone. This change makes ACI 318 terminology the same as that used in the IBC and ASCE 7 *Standard Minimum Design Loads for Buildings and Other Structures*⁵ (since its 1998 edition). Because the IBC has emerged as the one model building code for the entire country on which the legal codes by most legal jurisdictions (such as cities, counties, and states) are based, this is a sensible and timely change. The IBC will no longer have to provide an interface between its own terminology and that of ACI 318, as it has in the past.

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.

The first paragraph of Section R9.3.4 of ACI 318-05 has been 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 in special moment frames and special structural walls.

“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 previously stated 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.”

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 $0.7E$.

Overall changes to Chapter 21: Earthquake-Resistant Structures

Title

The title of the chapter has changed from “Special Provisions for Seismic Design” to “Earthquake-Resistant Structures.”

Notation

In previous editions of ACI 318, the format was to have all definitions in Chapter 2, with the exception of Chapter 21, which in ACI 318-05 contained definitions. Having definitions in two places is undesirable because there can be problems in updating the definitions consistently. Alternatively, having definitions in Chapter 21 but not Chapter 2 can create difficulties locating definitions. Thus, all definitions have been transferred from Chapter 21 to Chapter 2. In addition to the transfer, a few of the definitions have been modified.

Detailing requirements by seismic design category

ACI Committee 318 originally developed seismic design provisions for regions of high seismic risk. The design provisions were placed in Appendix A in earlier versions of ACI 318 and subsequently in Chapter 21. Provisions for regions of moderate seismic risk were added later.

It has always been understood among the users of ACI 318 that the body of the document, excluding Chapter 21, provides design and detailing requirements for regions of low seismic risk. As long as the model building codes divided the United States into Seismic Zones 0 through 4, and seismic detailing requirements were triggered by seismic zones, it was relatively easy for the practicing engineer to correlate the regions of low, moderate, and high seismic risk of ACI 318 with the Seismic Zones 0 through 4 of the model codes.

But then the model building codes started triggering seismic detailing requirements by seismic performance categories, which were a function of the seismic hazard at the site and the occupancy of the structure. The IBC⁶ now triggers seismic detailing requirements by seismic design category

ries (SDCs) that are additionally functions of the soil characteristics at the site. Thus, in recent times, ACI 318 has used the awkward language: “in regions of low seismic risk or for structures assigned to low seismic performance or design categories,” “in regions of moderate seismic risk or for structures assigned to intermediate seismic performance or design categories,” and “in regions of high seismic risk or for structures assigned to high seismic performance or design categories.”

ACI 318-08 has dropped this cumbersome language. Instead, SDCs are now used directly in Section 1.1.9, “Provisions for Earthquake Resistance,” Section 21.1.1, “Scope,” and elsewhere. This is a significant positive development. Because the IBC will no longer have to provide an interface between the SDC and the regions of low, moderate, and high seismic risk of ACI 318, it will be possible to eliminate unnecessary amendments to ACI 318 requirements.

More logical organization

In ACI 318-05, design and detailing requirements for structures assigned to SDC A and B are located in Chapters 1 through 18. Additional detailing requirements for structures assigned to SDC C are given in Sections 21.12 and 21.13, and those for structures assigned to SDCs D, E, and F are given in Sections 21.2.2 through 21.2.8 and 21.3 through 21.10. This is obviously not the most logical arrangement.

In ACI 318-08, seismic detailing requirements have been organized in the order of ascending SDCs. Chapter 21 starts with two new provisions for SDC B structures and the provisions for SDC C structures (commonly referred to as intermediate detailing) follow. Appearing last in Chapter 21 are the provisions for SDC D, E, and F structures (commonly referred to as special detailing). Table A.1 shows the section

number changes that have resulted in Chapter 21 from ACI 318-05 to 318-08.

Deliberate use of special

A primary use of the term *special* in Chapter 21 is to define structural systems in which the proportions and details make them suitable as primary lateral-force-resisting systems of structures assigned to high SDCs. However, the term *special* is also used throughout Chapter 21 of ACI 318-05 for other purposes, sometimes leading to confusion in code usage. Any unnecessary or confusing use of the term *special* has now been removed from all of Chapter 21, as well as from a few locations in Chapter 1 that refer to seismic design requirements.

The term *special transverse reinforcement*, which refers to the confinement reinforcement within the region of potential plastic hinging at the ends of special-moment-frame columns, has been dropped.

Specific changes to Chapter 21

Removal of commentary sentence

When the committee was removing the word *special*, it was noted that the 2003 National Earthquake Hazards Reduction Program (NEHRP) provisions⁷ and ASCE 7-05⁵ include intermediate precast concrete walls. Consequently, the no-longer-relevant commentary sentence, “Although new provisions are provided in 21.13 for design of intermediate precast structural walls, general building codes that address seismic performance or design categories do not include intermediate structural walls,” has been struck from ACI 318-08 commentary Section R21.1.1 (previously ACI 318-05 commentary Section R21.2.1).

Table A.1 ACI 318-05 and the corresponding ACI 318-08 Chapter 21 section numbers

ACI 318-08 section numbers	ACI 318-05 section numbers
All definitions have moved to chapter 2	21.1 Definitions
21.1 General Requirements	21.2 Same title
21.2 Ordinary Moment Frames	Not included
21.3 Intermediate Moment Frames	21.12 Requirements for Intermediate Moment Frames
21.4 Intermediate Precast Structural Walls	21.13 Same title
21.5 Flexural Members of Special Moment Frames	21.3 Same title
21.6 Special Moment-Frame Members Subjected to Bending and Axial Load	21.4 Same title
21.7 Joints of Special Moment Frames	21.5 Same title
21.8 Special Moment Frames Constructed Using Precast Concrete	21.6 Same title
21.9 Special Structural Walls and Coupling Beams	21.7 Special Reinforced Concrete Structural Walls and Coupling Beams
21.10 Special Structural Walls Constructed Using Precast Concrete	21.8 Same title
21.11 Structural Diaphragms and Trusses	21.9 Same title
21.12 Foundations	21.10 Same title
21.13 Members Not Designated as Part of the Seismic-Force-Resisting System	21.11 Members Not Designed as Part of the Lateral-Force-Resisting System

High-strength transverse reinforcement

Section 21.2.5 of ACI 318-05 introduced a sentence that limits the yield strength of transverse reinforcement, including spirals, to 60,000 psi. The added sentence was part of a change that modified ACI 318-02 Sections 9.4 and 10.9.3 to allow the use of spiral reinforcement with specified yield strength up to 100 ksi. The added sentence specifically prohibits such use in members resisting earthquake-induced forces in structures assigned to SDC D, E, or F. This was largely a result of some misgiving that high-strength spiral reinforcement might be less ductile than conventional mild reinforcement and that spiral failure has been observed in earthquakes.

There are fairly convincing arguments, however, against such specific prohibition. Spiral failure, primarily observed in bridge columns, has invariably been the result of insufficient spiral reinforcement. Also, prestressing steel, which is primarily the high-strength steel available on the U.S. market, is at least as ductile as welded-wire reinforcement, which is allowed to be used as transverse reinforcement.

Under 2006 IBC Section 1908.1.5, the applicability of the ACI 318-05 restriction “The value of f_{yt} for transverse reinforcement including spiral reinforcement shall not exceed 60,000 psi” is narrowed by the clause “for computing shear strength” in front of the requirement.

Two of the functions of transverse reinforcement in a reinforcement concrete member are to confine the concrete and to act as shear reinforcement. There has been enough testing of columns⁸⁻¹⁰ with high-strength confinement reinforcement (f_{yt} ranging up to 120 ksi and beyond) to show that there is no detriment to such use. The 2006 IBC, therefore, uses the ACI 318-05 upper limit on the yield strength of transverse reinforcement solely to limit the width of possible shear cracks to acceptable levels. This does not preclude the use of high-strength transverse reinforcement for confining the core of a concrete member.

During discussion of this item within ACI 318 Subcommittee, H. Budek¹¹ introduced research results that showed equivalent shear and confinement performance of 18-in.-diameter columns with transverse reinforcement having roughly 200 ksi yield strength.

Section 21.1.5.4 of ACI 318-08 now states, “The value of f_{yt} used to compute the amount of confinement reinforcement shall not exceed 100,000 psi.”

Section 21.5.5.5 of ACI 318-08 now states, “The value of f_y or f_{yt} used in design of shear reinforcement shall conform to 11.4.2.”

Section 11.4.2 reads, “The values of f_y and f_{yt} used in design of shear reinforcement shall not exceed 60,000 psi, except the value shall not exceed 80,000 psi for welded deformed wire reinforcement.”

Thus, unlike the 2006 IBC, ACI 318-08 imposes an upper limit of 100 ksi on the yield strength of high-strength confinement reinforcement in members resisting earthquake-induced forces in structures assigned to SDC D, E, or F. Also, ACI 318-08 requires such transverse reinforcement to conform to ASTM A1035.¹² Section 3.5.3.1 requires deformed reinforcement

bars to conform to ASTM A615¹³ (carbon steel), ASTM A706¹⁴ (low-alloy steel), and ASTM A955¹⁵ (stainless steel).

Ordinary moment frames

Section 21.2, for the first time, contains specific detailing requirements for ordinary moment frames in buildings assigned to SDC B.

Both SDC A and SDC B fall under the old designation of low seismic risk. Structures assigned to SDC A are required to satisfy Chapters 1 through 18 and Chapter 22 of ACI 318. Frames in buildings assigned to SDC B are required in both the 2003 NEHRP provisions⁷ and ASCE 7-05⁵ to satisfy additional requirements. The additional requirements of ASCE 7-05 are:

- “In structures assigned to SDC B, flexural members of ordinary moment frames forming part of the seismic force-resisting system shall have at least two main flexural reinforcing bars continuously top and bottom throughout the beams through or developed within exterior columns or boundary elements.”
- “In structures assigned to SDC B, columns of ordinary moment frames having a clear height-to-maximum-plan-dimension ratio of five or less shall be designed for shear in accordance with ACI 318-05 Section 21.12.3 (ACI 318-08 Section 21.3.3).”

Intermediate moment frames

There are important changes in Section 21.3, “Intermediate Moment Frames.” Intermediate moment frames include beam-column as well as slab-column moment frames. When the requirements for intermediate moment frames were introduced in ACI 318-83, shear design requirements for beam-column and slab-column frames were grouped (Section 21.12.3 of ACI 318-05). However, it was never intended that nominal shear stresses due to shear and moment transfer in two-way slab-column frames be treated the same as beam shear in beam-column frames or one-way shear in two-way slabs, though the provisions appear to indicate such. The provisions of former Section 21.12.3 constrained slab-column designs in a way that was not intended and that was not supported by observations in laboratory tests.

Analyses of laboratory tests¹¹ indicate that the ductility or inelastic deformability of slab-column framing is better judged on the basis of the level of gravity shear stress and the presence of slab shear reinforcement. This has been recognized for gravity framing of buildings assigned to high SDCs (Section 21.13.6) and slab-column intermediate frames (Section 21.3.6.8).

The purpose of a significant change to Section 21.3.6.8 is to clarify that the nominal shear stresses due to shear and moment transfer in two-way slabs do not need to satisfy the requirements of Section 21.3.3, but instead only need to satisfy the requirements of Section 21.3.6.8. In addition, Section 21.3.6.8 has been modified to make it more consistent with current understanding of the relationship between earthquake demands and strengths, as reflected in Section 21.13.6. ACI 318-05 permits the value of eccentric shear stress to reach

ϕV_n for design load combinations including E , as long as the contribution of E does not exceed $0.5\phi V_n$. Considering that E is the linear earthquake action divided by a force-reduction factor R , ACI 318-05 is believed to permit unsafe levels of nominal shear stresses.

A modification eliminates this provision in ACI 318-08 and replaces it with a more rational one. Specifically, in Section 21.3.6.8 (formerly Section 21.12.6.8), the second sentence has been changed from “It shall be permitted to waive this requirement if the contribution of the earthquake-induced factored two-way shear stress transferred by eccentricity of shear in accordance with 11.12.6.1 and 11.12.6.2 at the point of maximum stress does not exceed one-half of the stress ϕV_n permitted by 11.12.6.2” to “It shall be permitted to waive this requirement if the slab design satisfies requirements of 21.13.6.”

Columns supporting discontinued shear walls

Discontinuous shear walls and other stiff members can impose large axial forces on supporting columns during earthquakes. Section 21.4.4.5 of ACI 318-05 contains transverse reinforcement requirements for such columns in order to improve column toughness under anticipated demands. The requirements are triggered when “the factored axial compressive force in these members, related to earthquake effect, exceeds $A_g f'_c/10$.” The same requirements continue in Section 21.6.4.6 of ACI 318-08. However, the trigger has been adjusted by adding the following sentence: “Where design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $A_g f'_c/10$ shall be changed to $A_g f'_c/4$.”

While Section 21.6.4.6 applies to columns of special moment frames, ACI 318-08 has added corresponding requirements for columns of intermediate moment frames. Section 21.3.5.6 requires the following:

Columns supporting reactions from discontinuous stiff members, such as walls, shall be provided with transverse reinforcement at the spacing, s_o , as defined in 21.3.5.2 over the full height beneath the level at which the discontinuity occurs if the portion of factored axial compressive force in these members related to earthquake effects exceeds $A_g f'_c/10$. Where design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit $A_g f'_c/10$ shall be increased to $A_g f'_c/4$. This transverse reinforcement shall extend above and below the columns as required in 21.6.4.6(b).

It should be noted that the confinement that Section 21.3.5.6 requires is considerably less than that required by Section 21.6.4.6. Also, the 2006 IBC section 1908.1.12 modifies ACI 318-05 Section 21.12.5 to introduce a provision similar to that of ACI 318-08 Section 21.3.5.6.

Beams and joints of special moment frames

Changes have been made to Sections 21.5, “Flexural Members of Special Moment Frames,” and 21.7, “Joints of Special Moment Frames,” to clarify maximum beam width, joint confinement requirements, and design rules for joints having

beam stubs extending a short distance past the joint.

Research¹⁶ has shown that the effective beam width is more closely related to the depth of the column than it is to the depth of the beam. A recommendation based on the column widths and beam widths tested to date has been adopted.

Research¹⁷ has also indicated that extensions of beams (beam stubs) that project a short distance past the joint can also be considered as confining members to joints if they extend at least one effective depth beyond the joint face and meet the dimensional and reinforcement requirements for full flexural members. A change has been made to recognize this effect.

Section 21.5.1.4 now reads, “Width of member, b_w , shall not exceed width of supporting member, c_2 , plus a distance on each side of supporting member equal to the smaller of (a) and (b):

- (a) width of supporting member, c_2 , and
- (b) 0.75 times the overall dimension of supporting member, c_1 .”

In ACI 318-05 it reads, “plus distances on each side of supporting member not exceeding three fourths of the depth of flexural member.”

Sections 21.7.3.1, 21.7.3.2, and 21.7.3.3 on transverse reinforcement within beam-column joints of special moment frames has been rewritten for added clarity. Section 21.7.3.3, in particular, represents a major improvement.

In ACI 318-05 it reads, “Transverse reinforcement as required by 21.4.4 (now 21.6.4) shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement is not provided by a beam framing into the joint.”

It now reads, “Longitudinal beam reinforcement outside the column core shall be confined by transverse reinforcement passing through the column that satisfies spacing requirements of 21.5.3.2, and requirements of 21.5.3.3 and 21.5.3.6, if such confinement is not provided by a beam framing into the joint.”

An example of transverse reinforcement through the column provided to confine the beam reinforcement passing outside the column core is now shown in Fig. R21.5.1. This figure is a welcome addition (Fig. A.1).

Finally, the following text has been added to Section 21.7.4.1: “Extensions of beams at least one overall beam thickness h beyond the joint face are permitted to be considered as confining members. Extensions of beams shall satisfy 21.5.1.3, 21.5.2.1, 21.5.3.2, 21.5.3.3, and 21.5.3.6.”

Columns of special moment frames

Significant changes have been made to Section 21.6.4 to improve the organization and expression of transverse reinforcement requirements for columns in special moment frames, as well as to eliminate one provision.

The provisions of Section 21.6 are intended to apply to a column of a special moment frame for all load combinations if, for any load combination, the axial load exceeds $A_g f'_c/10$. The wording in ACI 318-05 is often misinterpreted as meaning that the provisions applied only for those load combinations for

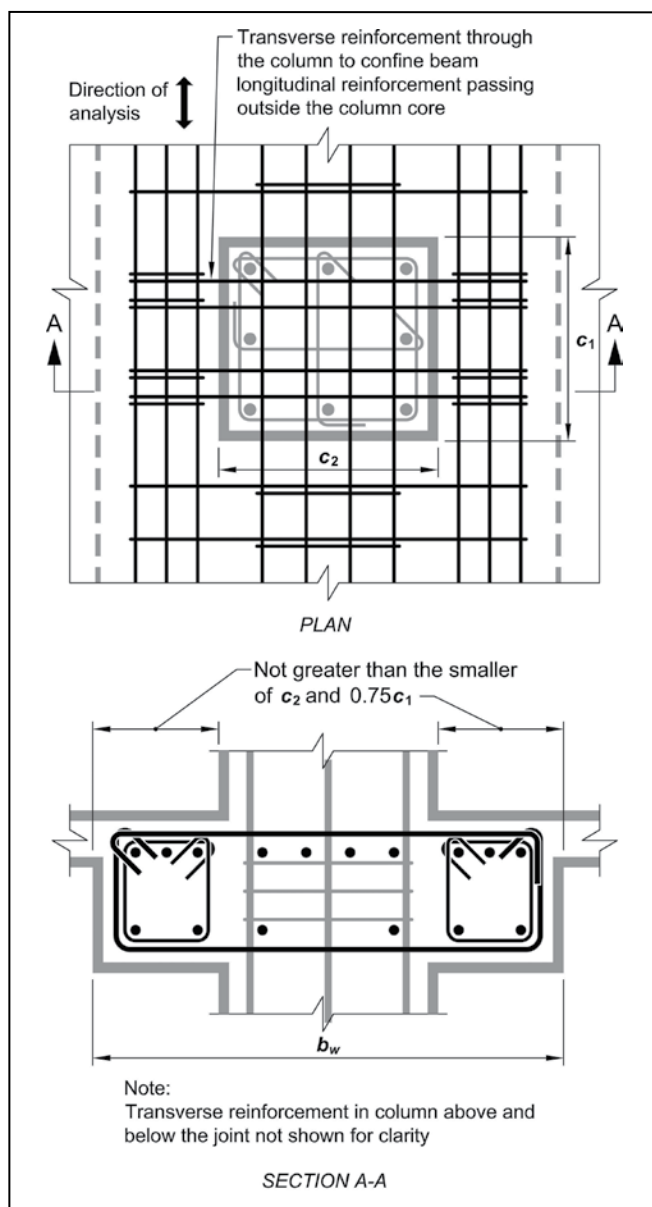


Fig. A.1 Maximum effective width of wide beam and required transverse reinforcement. Source: Reprinted by permission from Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08) (Farmington Hills, MI: ACI, 2008) p. 334, Figure R21.5.1.

which the axial load exceeded $A_g f'_c / 10$. Thus, Section 21.6.1 of ACI 318-08 has been modified to read, “Requirements of this section apply to special moment frame members...that resist a factored axial compressive force P_u under any load combination exceeding $A_g f'_c / 10$.” (emphasis added)

Several areas for potential improvement of ACI 318-05 provisions related to columns of special moment-resisting frames were first identified by ACI 318 Subcommittee H-Seismic Provisions. These included the following:

- The items listed as (a) through (e) in ACI 318-05 Section 21.4.4.1 are not expressed in parallel language. ACI 318-05 requires that the area of transverse rein-

forcement be determined using (a) or (b) but does not require that (c), (d), and (e) always be satisfied.

- The term *design strength of the member core* is used in ACI 318-05 in Section 21.4.4.1(d), but that terminology is not well defined.
- ACI 318-05 Section 21.4.4 uses both b_c and A_{sh} to determine the required amount of special transverse requirement. However, b_c is based on center-to-center dimensions and A_{sh} is based on the out-to-out dimensions of the hoop. To make it easier for the user, these items have been made consistent. In Section 2.1 of ACI 318-08, both A_{sh} and b_c are “measured to the outside edges of transverse reinforcement.” This amounts to a small increase in A_{sh} on the order of 2% to 3%.

Additional deliberations within ACI 318 Subcommittee H have led to additional changes, including:

- removing the terminology “special transverse reinforcement” to refer to the confinement reinforcement within the length ℓ_o of the column;
- eliminating ACI 318-05 Section 21.4.4.1(d), which allows the design of columns with less than the specified confinement reinforcement, if the column core satisfies design requirements;
- allowing crossties of diameter less than that of the hoops (Section 21.6.4.2).

ACI 318-05 Section 21.4.4 has been replaced with the new ACI 318-08 Section 21.6.4. The revision is outlined in Table A.2. Note that the revised Section 21.6.4 is made up of parts of ACI 318-05 Section 21.4.4, with some revisions and some reorganization.

A two-sentence paragraph has been added to Section R21.6.4.4.

“Equations (21-4) and (21-5) are to be satisfied in both cross-sectional directions of the rectangular core. For each direction, b_c is the core dimension perpendicular to the tie legs that constitute A_{sh} , as shown in Fig. R21.6.4.2 [Fig. A.2].”

This figure replaces ACI 318-05 Fig. R21.4.4 and represents a significant improvement.

Finally, the following sentence has been deleted from the commentary of ACI 318-05 Section 21.4.4.6 (now 21.6.4.5). “Field observations have shown significant damage to columns in the unconfined region near the midheight. The requirements of 21.4.4.6 are to ensure a relatively uniform toughness of the column along its length.”

Special moment frames made with precast concrete

A change has been made to commentary Section R21.8.4 to alert designers to ACI ITG-1.2,¹⁸ which provides an option to satisfy the provisions of Section 21.1.1.8.

ACI 374.1¹⁹ provides for the development of precast concrete special moment frames that can meet the requirements of Section 21.1.1.8. ACI ITG-1.2 is an industry standard that defines requirements, in addition to those of Section 21.8.4, for the design of one specific type of moment frame that con-

Table A.2 Reorganization of ACI 318-5 Section 21.4.4 into ACI 318-08 Section 21.6.4

ACI 318-08 section numbers	ACI 318-05 section numbers
21.6.4.4	21.4.4.1
No change	21.4.4.1(a)
No change	21.4.4.1(b)
21.6.4.2	21.4.4.1(c)
Deleted	21.4.4.1(d)
21.6.4.7	21.4.4.1(e)
21.6.4.3	21.4.4.2
21.6.4.2	21.4.4.3
21.6.4.2	21.4.4.4
21.6.4.6	21.4.4.5
21.6.4.5	21.4.4.6
Eq. (21-3)	Eq. (21-2)
Eq. (21-4)	Eq. (21-3)
Eq. (21-5)	Eq. (21-4)
Eq. (21-2)	Eq. (21-5)

sists of precast concrete beams post-tensioned to precast or cast-in-place concrete columns. The columns are continuous through the joints, and each beam spans a single bay.

The hybrid beam-to-column connection uses a system of post-tensioning strands that run through a duct at the center of the beam and through the column. Mild-steel reinforcement is placed in ducts in the center of the beam and through the column, and then grouted. A key feature of the hybrid frame connection is that the grouted mild reinforcing bars must be deliberately debonded for short distances in the beams adjacent to the beam-column interfaces in order to reduce the high cyclic strains that would otherwise occur at those locations. The amount of mild-steel reinforcement and post-tensioning steel are proportioned so that the frame recenters itself after a major seismic event.

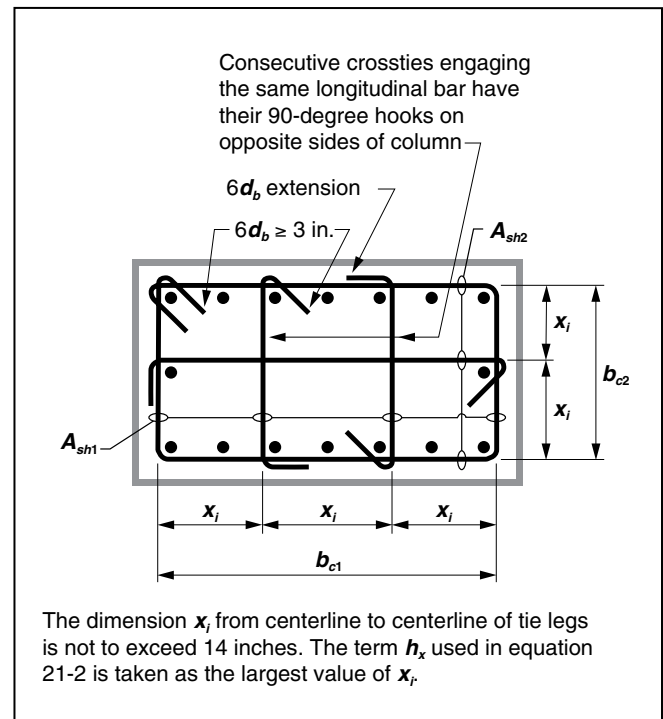
The University of Washington test results for the Third and Mission building in San Francisco, Calif., and the Precast Seismic Structural Systems (PRESSS) building test results for the frame direction can be used as a basis for special precast concrete hybrid moment frame designs in accordance with ITG-1.2. The results of the University of Washington tests are on file at ACI in conjunction with ITG-1.2. The results of the PRESSS building frame direction tests are available in a series of reports from PCI.

The following sentence has been added at the end of section R21.8.4. "ACI ITG 1.2 defines design requirements for one type of special precast concrete moment frame for use in accordance with 21.8.4."

ACI ITG-1.2 has also been added to the Chapter 21 commentary reference list.

Boundary elements of special shear walls

Section 21.9.6.4 (c) has been changed to permit increased

**Fig. A.2** This is an example of transverse reinforcement in columns.

Source: Reprinted by permission from *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)* (Farmington Hills, MI: ACI, 2008) p. 341, Figure R21.6.4.2.

spacing of transverse reinforcement in the boundary elements of walls with relatively thin boundary zones.

ACI 318-05 Section 21.7.6.4(c) requires special boundary elements to satisfy ACI 318-05 Section 21.4.4.2, which limits the spacing of transverse reinforcement to no more than one quarter of the minimum member dimension. This was an unintended consequence of referring the wall transverse reinforcement requirements to those of columns of special moment frames. For a 12-in.-thick wall, the spacing requirements could not exceed 3 in. The *Uniform Building Code*,²⁰ in its last two editions, relaxed the maximum spacing to the smaller of 6 in. and 6 times the longitudinal bar diameter, regardless of wall thickness.

Wall specimens tested by Thomsen and Wallace²¹ included rectangular walls RW1 and RW2 with boundary transverse reinforcement spaced at three quarters of the wall thickness. The walls had lateral drift capacities in excess of 2% of the wall height.

Section 21.9.6.4(c) now reads, "The boundary element transverse reinforcement shall satisfy the requirements of 21.6.4.2 through 21.6.4.4 except Eq. (21-4) need not be satisfied and the transverse reinforcement spacing limit of 21.6.4.3(a) shall be one-third of the least dimension of the boundary element."

A sentence has been added at the end of Section R21.9.6.4 that reads, "Tests show that adequate performance can be achieved using spacing larger than permitted by 21.6.4.3(a)."

Table A.3 Coupling beam detailing requirements of ACI 318-08

	ACI 318-08 section		
	21.9.7.1	21.9.7.3	21.9.7.2
Conditions	$(\ell_n/h) \geq 4$, no limit on V_u	$(\ell_n/h) < 4$, no limit on V_u	$(\ell_n/h) < 2$ and $V_u > 4\sqrt{f'_c} A_{cw}$
Conventional reinforcement per ACI 318-08 Section 21.5	Required*	Not mentioned†	Not permitted
Diagonal reinforcement per ACI 318-08 Section 21.9.7.4	Not permitted	Permitted	Required

*The provisions of Sections 21.5.1.3 and 21.5.1.4 need not be satisfied if it can be shown that the beam has adequate lateral stability.
†Only Sections 21.5.2 through 21.5.4 need be satisfied.
Note: A_{cw} = area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear; f'_c = design compressive strength of concrete; h = overall thickness or height of member; ℓ_n = length of clear span measured face to face of supports; V_n = nominal shear strength; V_u = factored shear force at section.

Table A.4 Coupling beam detailing requirements of ACI 318-05

	ACI 318-05 section		
	21.7.7.1	21.7.7.2	21.7.7.3
Conditions	$(\ell_n/h) \geq 4$, no limit on V_u	$(\ell_n/h) < 4$, no limit on V_u	$(\ell_n/h) < 2$ and $V_u > 4\sqrt{f'_c} A_{cw}$
Conventional reinforcement per ACI 318-05 Section 21.3	Required	Not mentioned	Not permitted
Diagonal reinforcement per ACI 318-05 Section 21.7.7.4	Not permitted	Permitted	Required

Note: A_{cw} = area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear; f'_c = design compressive strength of concrete; h = overall thickness or height of member; ℓ_n = length of clear span measured face to face of supports; V_n = nominal shear strength; V_u = factored shear force at section.

The Thomsen and Wallace paper²¹ has also been added to the Chapter 21 commentary reference list.

Coupling beams

In Section 21.9.7, a clarification has been provided that the provisions of Sections 21.5.2 through 21.5.4 can be applied to coupling beams of moderate aspect ratios ($2 \leq \ell_n/h < 4$).

Conventional reinforcing details for coupling beams with moderate aspect ratios ($2 \leq \ell_n/h < 4$) have at times been disallowed by building departments, even though the intent of ACI 318 has always been to allow these details. The ACI 318-08 provisions for coupling beams that are part of the lateral-force-resisting system are summarized in Table A.3.

It should be evident from Table A.3 that ACI 318 was inadvertently silent on the issue of whether conventional reinforcement could be used in beams with $2 \leq \ell_n/h < 4$. ACI 318-05 Section 21.7.7.2 reads, "Coupling beams with $(\ell_n/h) < 4$ shall be permitted to be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan."

ACI 318-08 Section 21.9.7.3 now reads, "Coupling beams not governed by 21.9.7.1 or 21.9.7.2 shall be permitted to be reinforced either with two intersecting groups of diagonally placed bars symmetrical about the midspan or according to 21.5.2 through 21.5.4."

Significant changes have been made to Section 21.9.7 to

relax the spacing requirements for transverse reinforcement confining diagonal reinforcement in coupling beams and to introduce an alternate detail involving confinement of the entire beam cross section (Fig. A.3).

ACI 318-99 first introduced diagonal reinforcement in coupling beams. Among its provisions was the requirement that spacing not exceed one quarter of the minimum member dimension. For diagonally reinforced coupling beams, this dimension is defined as the cross section of the diagonal cage (out to out) plus nominal cover. This may result in a spacing as low as 2 in. in practical situations. This appears to be unnecessarily restrictive.

Andres Lapage carried out a brief review of available diagonally reinforced coupling beam test results for the benefit of subcommittee H. Paulay and Binney²² had tested a 6 in. \times 31 in. coupling beam with a span-to-depth ratio of 1.3 and a 6 in. \times 39 in. beam with a span-to-depth ratio of 1.0. The spacing of transverse reinforcement around the diagonal reinforcement in both beams was 4 in. $4.6d_b$, $b_w/1.5$, and $d_1/1.0$. Tassios et al.²³ have tested a 5 in. \times 20 in. coupling beam with a span-to-depth ratio of 1.0 and a 5 in. \times 12 in. beam with a span-to-depth ratio of 1.7. The spacing of transverse reinforcement around the diagonal reinforcement was 2 in. $5.0d_b$, $b_w/2.5$, and $d_1/1.3$. Setting aside the difference in loading protocol, all four specimens exhibited negligible strength degradation up to total rotations of 5%.

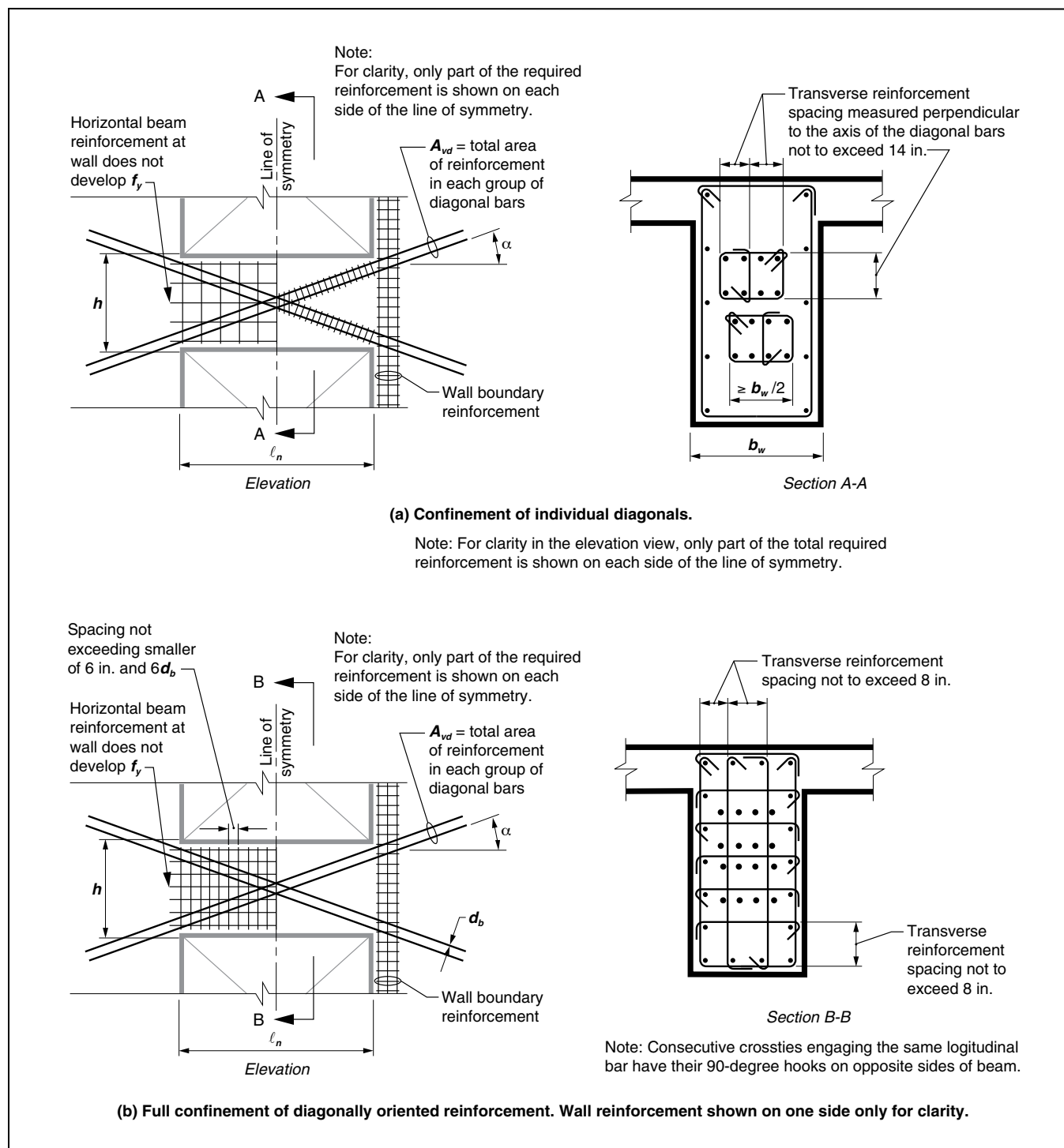


Fig. A.3. Coupling beams are shown with diagonally oriented reinforcement. Wall boundary reinforcement is shown on one side only for clarity. Source: Reprinted by permission from *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)* (Farmington Hills, MI: ACI, 2008) p. 355, Figure R21.9.7.

Galano and Vignoli²⁴ had tested two 6 in. \times 16 in. coupling beams with a span-to-depth ratio of 1.5. The spacing of transverse reinforcement around the diagonal reinforcement was 4 in., $10d_b$, $b_w/1.5$, and $d_1/1.3$. These specimens experienced buckling of the diagonal reinforcement at total rotations below 3%. It should be emphasized that the transverse reinforcement around the diagonal was spaced at $10d_b$.

The results suggest that the transverse reinforcement around the diagonals need not be spaced at one quarter the minimum dimension of the confined section as long as the spacing does not exceed $6d_b$. Thus some relaxation of the spacing requirement appears to be justified. See ACI 318-08 Section 21.9.7.4(c) for such relaxation.

ACI 318 Subcommittee H also explored whether confinement of the entire beam section might be a suitable alternative. Examples of this detailing in practice were identified. It was noted that this was the approach used successfully in early PCA tests.²⁵ This alternative detailing approach has now been incorporated into ACI 318-08 (see Section 21.9.7.4[d]). The organization and presentation of Section 21.9.7 have improved. Also, because the bars are diagonal (not longitudinal), and flexural reinforcement is not present, some sections that had been called out in earlier editions of ACI 318 were not strictly correct. These have been corrected.

The following important commentary has been added to Section R21.9.7:

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

Two confinement options are described. The first option is found in Section 21.9.7.4(c). This option is not needed but revisions have been made in the 2008 Code to relax spacing of transverse reinforcement confining the diagonal bars, to clarify that confinement is required at the intersection of the diagonals, and to simplify design of the longitudinal and transverse reinforcement around the beam perimeter; beams with these revised details are expected to perform acceptably.

Section 21.9.7.4(d) describes a second option for confinement of the diagonals introduced in ACI 318-08 (Figure R21.9.7(b)). This second option is to confine the entire beam cross section instead of confining the individual diagonals. This option can considerably simplify field placement of hoops, which can otherwise be especially challenging where diagonal bars intersect each other or enter the wall boundary.

Special structural walls made with precast concrete

A change made to Section 21.10 now allows the use of unbonded, post-tensioned precast concrete walls, coupled or uncoupled, as special structural walls, provided that the requirements of ACI ITG-5.1²⁶ are satisfied.

Testing and analysis²⁷⁻²⁹ have shown that, with appropriate limitations, unbonded post-tensioned precast concrete walls, coupled or uncoupled, can exhibit seismic performance equal to or better than that of cast-in-place special reinforced concrete shear walls. ITG-5.1 defines the protocol necessary to establish a design procedure, validated by analysis and laboratory tests, for such precast concrete walls. Provided that the requirements of ITG-5.1 are satisfied, the requirements of Section 21.1.1.8 that such walls must “have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying the chapter” are met.

Since 2002, ACI 318 has permitted in Section 21.8.3 (previously 21.6.3) the use of special moment frames constructed

using precast concrete, provided those frames met the requirements of ACI 374.1.¹⁹ The object of the recent change to Section 21.10 is to allow, in a similar manner, through a systematic program of analysis and laboratory testing, the use of one type of special precast concrete structural walls. For special precast concrete moment frames, Section 21.8.3 also contains two requirements related to the details and materials used in test specimens and the design procedure used to proportion test specimens. For walls, those latter two requirements are not needed because they are specifically included in ITG-5.1.

The ITG-5.1 document has been adopted in Section 3.8.10 of ACI 318-08. Section 21.10.3 now reads, “Special structural walls constructed using precast concrete and unbonded post-tensioning tendons and not satisfying the requirements of 21.10.2 are permitted provided they satisfy the requirements of ACI ITG-5.1.”

A new commentary section R21.10.3 has also been added.

Structural diaphragms and trusses

Terminology and design requirements for diaphragms and trusses have been updated through changes in Sections 21.1, “Definitions,” and 21.11, “Structural Diaphragms and Trusses.” ACI 318-05 Sections 21.9.2 and 21.9.3, as written, implies that a complete transfer of forces is required only for composite-topping slab diaphragms, not for non-composite diaphragms. A new Section, 21.11.3, has been added to clarify that this is required for all diaphragms. Structural trusses are separated from diaphragms because requirements differ and separating them clarifies the requirements.

In Sections 21.1, in the definition of *boundary elements*, any reference to diaphragms has been eliminated. Boundary elements are now for structural walls only.

The definition of *collector element* has been revised to “Element that acts in axial tension or compression to transmit seismic forces within a structural diaphragm or between a structural diaphragm and a vertical element of the lateral-force-resisting system.”

The definition of *structural diaphragm* has been revised to clarify that it transmits forces to “the vertical elements of the lateral-force-resisting system,” rather than to “lateral-force-resisting members.”

The definitions of *strut* and *tie elements* have been deleted. Section 21.11.2, “Design Forces,” which is new in ACI 318-08, reads, “The earthquake design forces for structural diaphragms shall be obtained from the legally adopted general building code using the applicable provisions and load combinations.”

New Section 21.11.3, “Seismic Load Path,” reads:

21.11.3.1—All diaphragms and their connections shall be proportioned and detailed to provide for a complete transfer of forces to collector elements and to the vertical elements of the lateral-force-resisting system.

21.11.3.2—Elements of a structural diaphragm system that are subjected primarily to axial forces and used to transfer diaphragm shear or flexural forces around openings or other discontinuities, shall comply with the requirements

for collectors in 21.11.7.5 and 21.11.7.6.

Any reference to continuous load path has been eliminated from Section 21.11.4, “Cast-in-Place Composite-Topping Slab Diaphragm,” because the topic is now covered in Section 21.11.3.

Section 21.11.7.2 has been changed for clarity to “Bonded tendons used as reinforcement to resist collector forces or diaphragm shear or flexural tension.”

Section 21.11.7.3 now reads, “All reinforcement used to resist collector forces, diaphragm shear, or flexural tension.”

References to structural truss elements, struts, ties, and diaphragm chords have been eliminated from Section 21.11.7.5.

The former Sections 21.9.8.1 and 21.9.8.2 have now been replaced by Section 21.11.8, “Flexural Strength,” which reads, “Diaphragms and portions of diaphragms shall be designed for flexure in accordance with 10.2 and 10.3 except that the nonlinear distribution of strain requirements of 10.2.2 for deep beams need not apply. The effects of openings shall be considered.”

This is an important change because flexure is no longer supposed to be resisted by the boundary element reinforcement only, as is implied in earlier editions of ACI 318.

The new Section 21.11.11, “Structural Trusses,” now reads:

21.11.11.1—Structural truss elements with compressive stresses exceeding 0.2 at any section shall have transverse reinforcement, as given in 21.6.4.2 through 21.6.4.4 and 21.6.4.7, over the length of the element.

21.11.11.2—All continuous reinforcement in structural truss elements shall be developed or spliced for f_y in tension.

A new commentary Section in R21.11.2 has been added. There are other significant additions to and deletions from the commentary on Section 21.11. Notable among these are the additions to Section R21.11.8.

Diaphragm shear strength

Studies of precast concrete parking structures following the 1994 Northridge earthquake³⁰ indicate that composite topping slab diaphragms depend on shear friction to transmit inertial forces to the vertical elements of the lateral-force-resisting system. The results of this research were used to develop ACI 318-05 Eq. (21-11) and are summarized in ACI 318-05 commentary Section R21.9.7. However, ACI 318-05 Eq. (21-11) referred to distributed transverse reinforcement within the diaphragm (for consistency with ACI 318-05 Eq. [21-10]) rather than to distributed longitudinal reinforcement. When the provisions in ACI 318-05 Section 21.9.7.2 were originally developed, it was assumed that the sentence “The required web reinforcement should be distributed uniformly in both directions” was sufficient to ensure that the same amount of reinforcement be used in both the longitudinal and the transverse directions. There were subsequent indications that clarification was needed.

ACI 318-05 Section 21.9.7.3 (now 21.11.9.3) has been revised to directly refer to shear friction reinforcement. Both

boundary and distributed reinforcement in the topping slab are assumed to contribute to the shear strength of the topping slab diaphragm, but connectors between the precast concrete elements are not included at this time.

Section 21.11.9.1 (formerly 21.9.7.1) has added the following text below Eq. (21-10): “For cast-in-place topping slab diaphragms on precast floor or roof members, A_{cv} shall be computed using the thickness of topping slab only for non-composite topping slab diaphragms and the combined thickness of cast-in-place and precast elements for composite topping slab diaphragms. For composite topping slab diaphragms, the value of f'_c used to determine V_n shall not exceed the smaller of f'_c for the precast members and f'_c for the topping slab.”

Sections 21.11.9.3 and 21.11.9.4 now read:

21.11.9.3—Above joints between precast elements in non-composite and composite cast-in-place topping slab diaphragms, V_n shall not exceed

$$V_n = A_{vf} f_y \mu$$

where A_{vf} is total area of shear friction reinforcement within topping slab, including both distributed and boundary reinforcement, that is oriented perpendicular to joints in the precast system and coefficient of friction, μ , is 1.0λ where λ is given in 11.6.4.3. At least one-half of A_{vf} shall be uniformly distributed along the length of the potential shear plane. Area of distributed reinforcement in topping slab shall satisfy 7.12.2.1 in each direction.

21.11.9.4—Above joints between precast elements in non-composite and composite cast-in-place topping slab diaphragms, V_n shall not exceed the limits in 11.6.5 where A_c is computed using the thickness of the topping slab only.

Commentary Section R21.9.7 has been modified to explain the changes.

Gravity columns

An error has been corrected in Section 21.13, “Members Not Designated as Part of the Seismic-Force-Resisting System.” Consider the case of a gravity column where the effects of design displacements are not explicitly checked and the member has axial load exceeding $A_g f'_c / 10$. According to ACI 318-05 Section 21.11.3.3, the member need not satisfy ACI 318-05 Section 21.4.3.2 and, therefore, the column lap splice might be located at the base of the column. However, if the effects of the design displacements were checked, then according to ACI 318-05 Section 21.11.2.2, the member must satisfy ACI 318-05 Section 21.4.3.2 and the location of the column splice must be within the center of the column length.

In other words, if a gravity column might yield under the design displacements, the designer could splice the longitudinal reinforcement at any location, but if the column was not expected to yield, the splice has to be located near mid-height. This does not make sense.

This flaw traced back to ACI 318-95. Before the 1995 edition, lap-splice locations were not prescribed for members that were not proportioned to resist forces induced by earthquake motions. The flaw has been corrected in ACI 318-08. ACI 318-05 Section 21.11.2.2 requires members with factored gravity axial forces exceeding $A_g f'_c / 10$ to satisfy Sections 21.4.3, 21.4.4.1(c), 21.4.4.3 and 21.4.5. ACI 318-08 Section 21.13.3.2 now requires such columns to satisfy Sections 21.6.3.1, 21.6.4.2, and 21.6.5. ACI 318-05 Section 21.11.3.3 requires members with factored gravity axial forces exceeding $A_g / 10$ to satisfy Sections 21.4.3.1, 21.4.4, 21.4.5, and 21.5.2.1. ACI 318-08 Section 21.13.4.3 now requires such columns to satisfy Sections 21.6.3, 21.6.4, 21.6.5, and 21.7.3.1.

Structural Integrity Related:

In Section 7.13, “Requirements for Structural Integrity,” changes have been made to the anchorage and splice requirements for structural integrity reinforcement. Continuous top and bottom structural reinforcement is now required to pass through the column core. Also, the types of transverse reinforcement used to enclose structural integrity reinforcement in perimeter beams are more clearly specified.

A new Section 12.1.3 has been added. The section specifically calls designers’ attention to the structural-integrity requirements in Section 7.13. There has been 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.

Chapter 5 – Design of Precast and Prestressed Concrete Components

In Section 9.3.2.2, the strength-reduction factor ϕ for spirally reinforced columns has been 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 excitations³¹ and is partly due to new reliability analyses.³²

In Section 9.3.5, the ϕ factor for plain concrete has been 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.³³

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:

$$\frac{k \ell_u}{4} \leq 22 \text{ in.}$$

and “for compression members braced against sidesway when:

$$\frac{k \ell_u}{r} \leq 34 - 12 \left(\frac{M_1}{M_2} \right) \leq 40 \text{ in.}$$

where

k = effective length factor

ℓ_u = unsupported length The font

r = radius of gyration

M_1 = smaller factored end moment

M_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 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 nonsway 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.^{34,35}

Commentary Section R10.13.8, “Tie Reinforcement around Structural Steel Core,” which is Section R10.16.8 in ACI 318-05, states:

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.”

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”¹⁾) 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 Non-prestressed 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 has been 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 have been made to the list of members in Section 11.4.6.1 for which minimum shear reinforcement is not required where V_u exceeds $0.5\phi V_c$. 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,³⁶ a new limit on the depth of hollow-core units of 12.5 in. is 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.”³⁷⁻³⁹ 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.” In Section 11.6.5, the upper limit on the nominal shear-friction strength V_n has been 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.^{40, 41}

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”).

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 has been moved from the numerator to the denominator. At the same time, in Section 12.2.4(d), $\lambda = 1.3$ has been 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 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.” 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.”^{42, 43} 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 # 11, and the upper limit of 60,000 psi on f_y are based on test data.⁴⁴

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.

“For bars in tension, heads allow the bars to be developed in a shorter length than required for standard hooks.”⁴²⁻⁴⁴ 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.⁴²⁻⁴⁴

“Because transverse reinforcement has been 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 has been 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.”

Section 14.3.7 of ACI 318-05 reads, “In addition to the minimum reinforcement required by 14.3.1, not less than (2) # 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 (2) # 5 bars in walls having two layers of reinforcement in both directions and (1) # 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),⁴⁵ which in turn are based on experimental research.⁴⁶ Changes have been 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 of ACI 318, 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 $h/2d$ 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 has been modified in 2008 by introducing the $h/2d$ 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 ACI 318-08, 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 data⁴⁶ indicates 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 $2/3M_{cr}$.

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.

Chapter 6 – Design of Connections

The three significant changes in Appendix D are the following:

- New definitions of reinforcement types that cross the concrete breakout surface
- Important revisions to the requirements that ductile anchor failure precede any concrete failure mode in moderate to high seismic applications
- Addition of provisions for lightweight concrete, in the computation of concrete breakout strength of anchors

ACI 318-05 defines an anchor group as “a number of anchors of approximately equal effective embedment depth with each anchor spaced at less than three times its embedment depth [$3h_{ef}$] from one or more adjacent anchors.” This definition considers anchors subject to tension but not anchors subject to shear. This deficiency has now been corrected. The 318-08 definition reads, “a number of anchors of approximately equal effective embedment depth with each anchor spaced at less than $3h_{ef}$ from one or more adjacent anchors when subjected to tension, or $3c_{a1}$ from one or more adjacent anchors when subjected to shear.” The distance from the center of an anchor shaft to the edge of concrete in one direction is represented by c_{a1} . ACI 318-08 has also added to the definition, “only those anchors susceptible to the particular failure mode under investigation shall be included in the group.”

An important new term, *anchor reinforcement*, is defined in Chapter 2 as “reinforcement used to transfer the full design load from the anchors into the structural member.” New Sections D.5.2.9 and D.6.2.9 contain provisions concerning anchor reinforcement.

ACI 318-05 defines supplementary reinforcement as “reinforcement proportioned to tie a potential concrete failure prism to the structural member.” ACI 318-08 has revised the supplementary reinforcement definition to read, “reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load from the anchors into the structural member.” The second part of the revised definition clearly indicates that supplementary reinforcement is not anchor reinforcement.

Section D.3.3 of ACI 318-05 reads, “When anchor design includes seismic loads, the additional requirements of D.3.3.1 through D.3.3.5 shall apply.” This wording has now changed to “when anchor design includes earthquake forces for structures assigned to Seismic Design Category C, D, E, or F, the additional requirements of D.3.3.1 through D.3.3.6 shall apply.” There are two differences:

- Section D.3.3.6 has been added.
- The applicability of the ACI 318-05 provision included seismic design category (SDC) B, which is no longer the case.

The change in the SDC matters for Section D.3.3.1 only because subsequent ACI 318-05 language restricted the applicability of Sections D.3.3.2 through D.3.3.5 to structures assigned SDC C, D, E, or F.

Section D.3.3.2 of ACI 318-08 specifically requires that “pullout strength N_p and steel strength of the anchor in shear V_{sa} shall be based on the results of the ACI 355.2 Simulated Seismic Tests.” This specific requirement is not part of ACI 318-05.

Section D.3.3.3 has undergone an important change. While the section in ACI 318-05 requires that “the design strength of anchors shall be taken as $0.75\phi N_n$ and $0.75\phi V_n$,” the ACI 318-08 section requires that “the anchor design strength associated with concrete failure modes shall be taken as $0.75\phi N_n$ and $0.75\phi V_n$.” The variables ϕ , N_n , and V_n represent the strength reduction factor, the nominal strength in tension, and the nominal shear strength, respectively. By making the seismic reduction apply only to concrete failure modes, it is significantly more difficult to meet the requirements of Section D.3.3.4 when anchors subjected to seismic forces in structures assigned to SDC C, D, E, or F have to be governed by the strength of a ductile steel element.

Section D.3.3.4 of ACI 318-05 waives the ductile anchor failure requirement if Section D.3.3.5 can be satisfied. The same section in ACI 318-08 waives the ductile failure requirement if either section D.3.3.5 or Section D.3.3.6 can be satisfied.

The 2006 IBC Section 1908.1.16 modified ACI 318-05 section D.3.3.5 to read, “Instead of D.3.3.4 . . . specified in D.3.3.3, *or the minimum design strength of the anchors shall be at least 2.5 times the factored forces transmitted by the attachment.*” The 2006 IBC includes the text of ACI 318-05 Section D.3.3.5 with the addition of “or the minimum design strength of the anchors shall be at least 2.5 times the factored forces transmitted by the attachment.” In other words, ductile anchor failure was declared unnecessary if the anchorage was oversized for concrete breakout. This concept has

been adopted into Section D.3.3.6 of ACI 318-08 with the wording, “as an alternative to D.3.3.4 [ductile anchor failure] and D.3.3.5 [yielding in the attachment], it shall be permitted to take the design strength of the anchors as 0.4 times the design strength determined in accordance with D.3.3.3.” For anchors of stud-bearing walls, the 0.4 factor may be taken as 0.5. Because ACI 318 is a material standard, the committee did not feel comfortable modifying the design load, as the 2006 IBC had done. It was decided instead, in effect, to modify the strength reduction factor. A 0.4 multiplier on ϕ is equivalent to a 2.5 multiplier on the design load.

Section D.3.4 concerning anchors embedded in lightweight concrete is different—but mostly in appearance, not really in substance.

A very important sentence has been added to Section D.4.2.1: “Where anchor reinforcement is provided in accordance with D.5.2.9 and D.6.2.9, calculation of the concrete breakout strength in accordance with D.5.2 and D.6.2 is not required.” This sentence was added because anchor reinforcement is reinforcement that carries all the design load when breakout failure occurs.

The commentary concerning supplementary reinforcement has changed in Section RD.4.4. Important new points are, “An explicit design of supplementary reinforcement is not required. However, the arrangement of supplementary reinforcement should generally conform to that of the anchor reinforcement shown in Fig. RD.5.2.9 and RD.6.2.9 (b). Full development is not required [of the supplemental reinforcement].”

The descriptions of conditions A and B in Section D.4.4 have been editorially modified for greater clarity. The descriptions now read, “Condition A applies where supplementary reinforcement is present except for pullout and pryout strengths. Condition B applies where supplementary reinforcement is not present, and for pullout or pryout strength.”

A distinction is now made between the effective cross-sectional area of anchor in tension $A_{se,N}$ and the effective cross-sectional area of anchor in shear $A_{se,V}$. In ACI 315-05, there is only the effective cross-sectional area of anchor A_{se} . The change is reflected in ACI 318-08 Eq. (D-3), (D-19), and (D-20). Commentary Section RD.5.1.2 reproduces an equation from American National Standards Institute (ANSI)/American Society of Mechanical Engineers (ASME) B1.1, *Unified Inch Screw Threads (UN and UNR Thread Form)*⁴⁷ for $A_{se,N}$ of threaded bolts. Commentary Section RD.6.1.2 reproduces an equation from ANSI/ASME B1.1 for $A_{se,V}$ for threaded bolts. The two equations are identical. The difference in the value obtained from the two equations is evident for postinstalled mechanical anchors, particularly torque-controlled expansion anchors with a tapered conical shape at the bottom. Approved postinstalled anchors give both effective areas in the product approval (in most cases, the areas provided are the same as those given by the equations in the ACI 318-08 commentary).

In Eq. (D-7) for basic concrete breakout strength, a lightweight concrete factor λ was introduced.

A very important new section, D.5.2.9, has been added to ACI 318-08. This section reads, “Where anchor reinforcement is developed in accordance with Chapter 12 on both

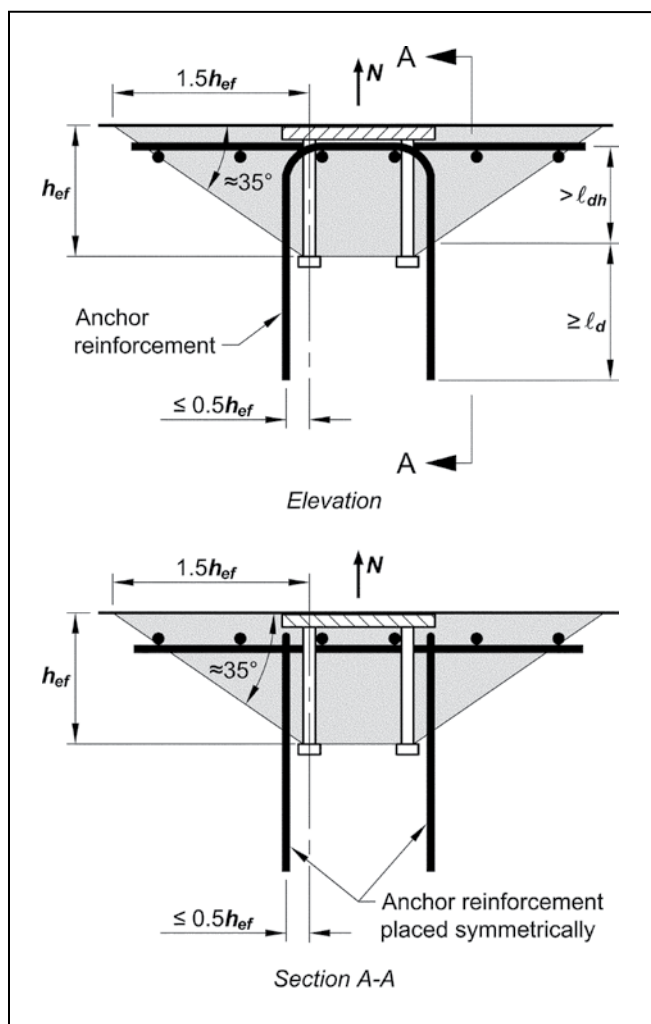


Fig. A.4 Anchor reinforcement for tension.

Source: Reprinted by permission from *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)* (Farmington Hills, MI: ACI, 2008), p. 426, Fig. RD.5.3.

sides of the breakout surface, the design strength of the anchor reinforcement shall be permitted to be used instead of the concrete breakout strength in determining ϕN_n . A strength reduction factor of 0.75 shall be used in the design of the anchor reinforcement.” An important new commentary Section RD.5.2.9 states that “for conditions where the factored tensile force exceeds the concrete breakout strength of the anchor(s) or where the breakout strength is not evaluated, the nominal strength can be that of anchor reinforcement.” The commentary includes Fig. RD.5.2.9, which is helpful and is reproduced here as Fig. A.4.

The term d_o , which represents the outside diameter or shaft diameter of a headed stud, headed bolt, or hooked bolt of ACI 318-05, has been replaced with d_a in ACI 318-08. This change is reflected in Eq. (D-16) for pullout strength in tension and in Section D.8.3.

The lightweight concrete factor λ is introduced in Eq. (D-17) for the concrete side-face blowout strength of a single anchor in tension and in Eq. (D-18) for the concrete side-face blowout strength of a group of anchors in tension.

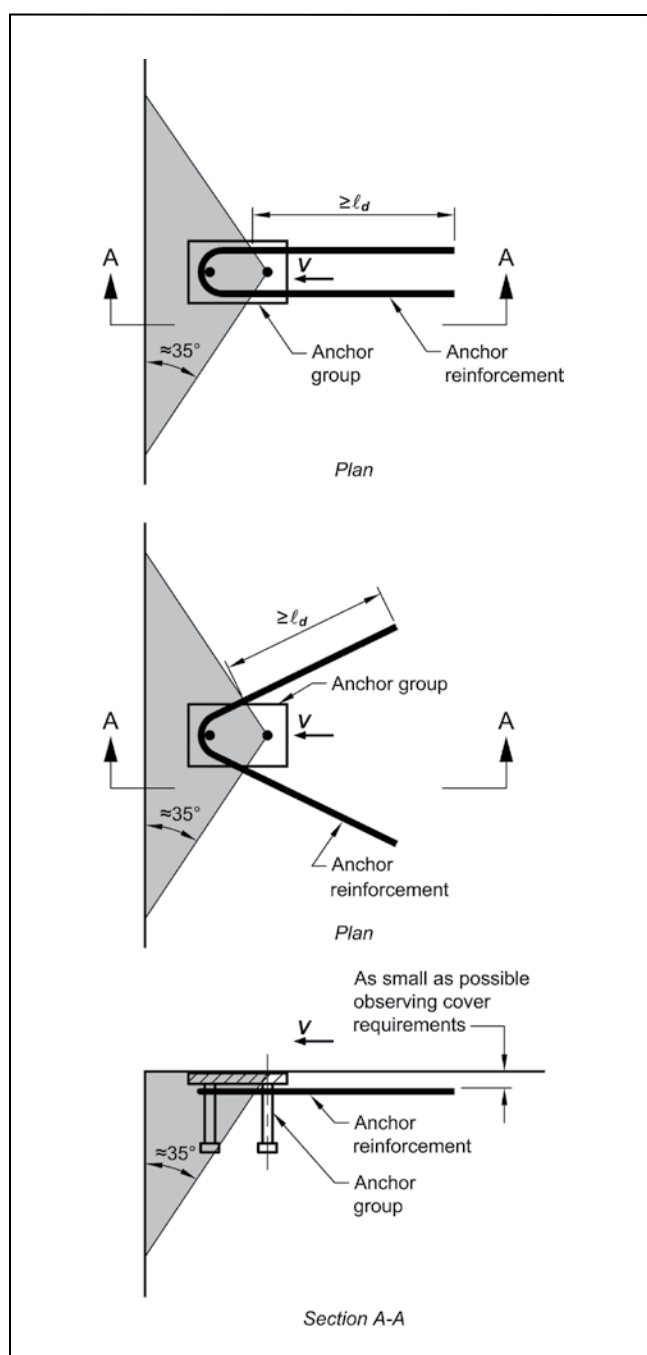


Fig. A.5 Hairpin anchor reinforcement for shear.

Source: Reprinted by permission from *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)* (Farmington Hills, MI: ACI, 2008), p. 435, Fig. RD.6.2.9(a).

A new modification factor $\psi_{h,v}$ has been added to Eq. (D-21) and (D-22) for concrete breakout strength of anchors in shear. The factor is defined by Eq. (D-29) and is for anchors located in a concrete member in which $h_a < 1.5c_{a1}$ and h_a is the thickness of the member in which an anchor is located, measured parallel to anchor axis.

The lightweight concrete factor λ is also introduced into Eq. (D-24) and (D-25) for the basic concrete breakout strength in shear.

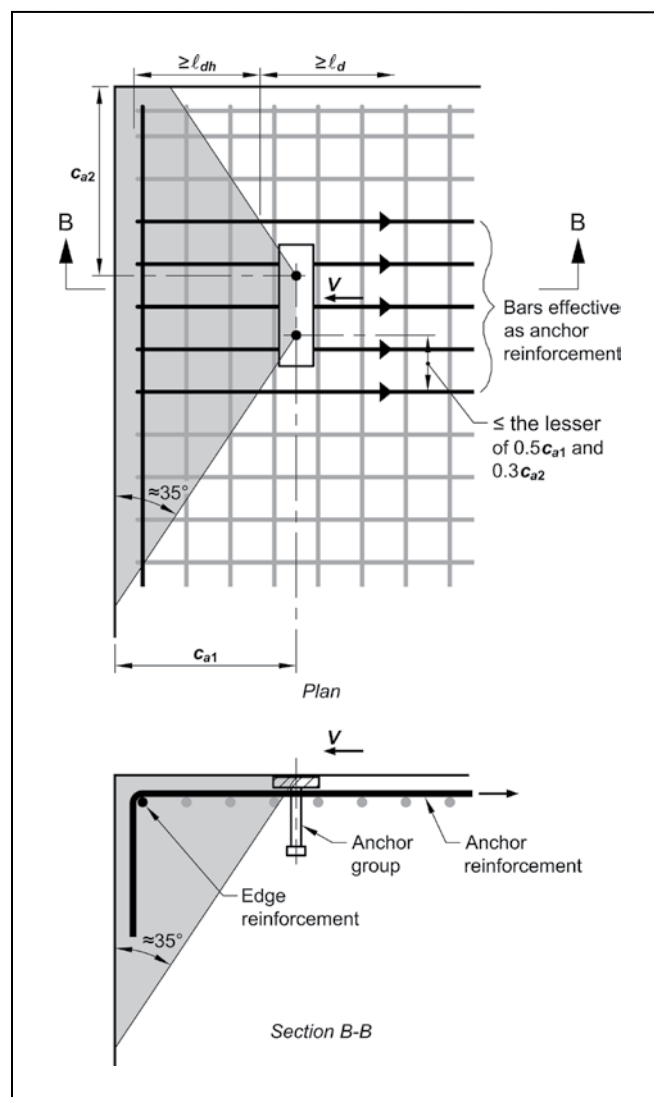


Fig. A.6 Edge reinforcement and anchor reinforcement for shear. Source: Reprinted by permission from *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)* (Farmington Hills, MI: ACI, 2008), p. 435, Fig. RD.6.2.9(b).

Paralleling Section D.5.2.9, another important new section, D.6.2.9, has been added to ACI 318-08. It reads, “Where anchor reinforcement is either developed in accordance with Chapter 12 on both sides of the breakout surface, or encloses the anchor and is developed beyond the breakout surface, the design strength of the anchor reinforcement shall be permitted to be used instead of the concrete breakout strength in determining ϕV_n . A strength reduction factor of 0.75 shall be used in the design of the anchor reinforcement.” New commentary Section RD.6.2.9 explains the provision. It includes Fig. RD.6.2.9(a) and RD.6.2.9(b), which are reproduced here as Fig. A.5 and A.6, respectively.

Chapter 7 – Structural Considerations for Architectural Precast Concrete

This chapter is not affected by ACI 318-08.

Chapter 8 – Component Handling and Erection Bracing

This chapter is not affected by ACI 318-08.

Chapter 9 – Precast and Prestressed Concrete: Materials

The definition of *lightweight aggregate concrete* in Chapter 2 now references ASTM C3307 and replaces “dry loose weight of 70 lb or less” with “loose bulk density of 70 lb/ft³ or less, determined in accordance with ASTM C29.”

All-lightweight and *sand-lightweight* concretes are now defined separately, whereas they used to be part of the definition of *lightweight concrete*. *Normalweight concrete* is now also defined. In the definition of *lightweight concrete*, “equilibrium density not exceeding 115 lb/ft³” has been replaced with “equilibrium density between 90 and 115 lb/ft³.”

Chapters 4 and 5 of earlier editions of ACI 318 were reformatted in 1989 to emphasize the importance of considering durability requirements before selecting f'_c and the specified concrete cover over the reinforcing steel. In ACI 318-08, the format of Chapter 4 is extensively revised by introducing exposure categories and classes, with applicable durability requirements given for the various classes in a unified format.

ACI 318-08 defines exposure categories and classes for concrete structures in Section 4.2.1—specifically in Tables 4.2.1.a through 4.2.1.d. Tables A.6 through A.9 are adapted from those tables and the commentary to Section 4.2.1. Associated requirements for concrete relative to the exposure classes are provided in ACI 318 Section 4.3.

Because of the concern that material properties may change with time, a limit of 12 months has been imposed on the age of the historical data used to qualify mixture proportions under Section 5.3, and proportioning on the basis of field experience or trial mixtures or both is allowed.

Under Section 5.3.3.2, the requirements that must be met by trial mixtures for concrete proportions established from such mixtures to be acceptable have been revised as indicated:

- Requirement (b) of Section 5.3.3.2 in 318-05 reads: “Trial mixtures having proportions and consistencies required for proposed work shall be made using at least three different water-cementitious materials ratios or cementitious materials contents that will produce a range of strengths encompassing f'_{cr} .”

It now reads: “Trial mixtures with a range of proportions that will produce a range of compressive strengths encompassing f'_{cr} and meet the durability requirements of Chapter 4.”

- Requirement (c) of ACI 318-05 is: “Trial mixtures shall be designed to produce a slump within ± 0.75 in.

Table A.5 ASTM reference standards added to ACI 318-08, Section 3.8

ASTM Standard	Title
A820/A820M-06	<i>Standard Specification for Steel Fibers for Fiber-Reinforced Concrete</i>
A955/A955M-07A	<i>Standard Specification for Deformed and Plain Stainless-Steel Bars for Concrete Reinforcement</i>
A970/A970M-06	<i>Standard Specification for Headed Steel Bars for Concrete Reinforcement</i>
A1022/A1022M-07	<i>Standard Specification for Deformed and Plain Stainless Steel Wire and Welded Wire for Concrete Reinforcement</i>
A1035/A1035M-07	<i>Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement</i>
A1044/A1044M-05	<i>Standard Specification for Steel Stud Assemblies for Shear Reinforcement of Concrete</i>
C29/C29M-97 (revised 2003)	<i>Standard Test Method for Bulk Density (Unit Weight) and Voids in Aggregate</i>
C231-04	<i>Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method</i>
C1012-04	<i>Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution</i>
C1116-06/C1116M-06	<i>Standard Specification for Fiber-Reinforced Concrete</i>
C1602/C1602M-06	<i>Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete</i>
C1609/C1609M-06	<i>Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading)</i>

Table A.6 Exposure Category F based on freezing and thawing exposure

Class	Description	Condition
F0	Not applicable	Concrete not exposed to freezing and thawing cycles
F1	Moderate	Concrete exposed to freezing and thawing cycles and that may be occasionally exposed to moisture before freezing (for example, exterior walls, beams, girders, and slabs not in direct contact with soil)
F2	Severe	Concrete exposed to freezing and thawing cycles and that is in continuous contact with moisture before freezing (for example, water tanks)
F3	Very severe	Concrete exposed to freezing and thawing cycles, that is in continuous contact with moisture, and where exposure to deicing chemicals is anticipated (for example, parking structures in the northern United States)
Source: Data adapted from <i>Building Code Requirements for Structural Concrete (ACI 318-08)</i> and <i>Commentary (ACI 318R-08)</i> Table 4.2.1 and commentary Section R4.2.1.		

Table A.7 Exposure Category S based on sulfate exposure

Class	Description	Water-soluble sulfate in soil, % by weight	Sulfate in water, ppm	Commentary
S0	Not applicable	$SO_4 < 0.10$	$SO_4 < 150$	Injurious sulfate attack not common
S1	Moderate	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 < 1500$ sea water	More critical value of measured water-soluble sulfate concentration in soil or the concentration of dissolved sulfate in water governs
S2	Severe	$0.20 \leq SO_4 \leq 2.00$	$1500 \leq SO_4 \leq 10,000$	Same as for S1
S3	Very severe	$SO_4 > 2.00$	$SO_4 > 10,000$	Same as for S1
Source: Data adapted from <i>Building Code Requirements for Structural Concrete (ACI 318-08)</i> and <i>Commentary (ACI 318R-08)</i> Table 4.2.1 and commentary Section R4.2.1. Note: ppm = parts per million.				

of maximum permitted, and for air-entrained concrete, within $\pm 0.5\%$ of maximum allowable air content.”

It now reads: “Trial mixtures shall have slumps within the range specified for the proposed Work; for air-

entrained concrete, air content shall be within the tolerance specified for the proposed Work.”

- Requirement (d) of ACI 318-05 is: “For each water cementitious materials ratio or cementitious materials content, at least three test cylinders for each test age

Table A.8 Exposure Category P based on requirements for low permeability

Class	Description	Condition	Commentary
P0	Not applicable	Concrete not required to have low permeability to water	—
P1	Applicable	Concrete required to have low permeability to water	When the permeation of water into concrete might reduce durability or affect the intended function of the structural element

Source: Data adapted from *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)* Table 4.2.1 and commentary Section R4.2.1.

Table A.9

Class	Description	Condition	Commentary
C0	Not applicable	Concrete that will be dry or protected from moisture in service	No additional protection required against the corrosion of reinforcement
C1	Moderate	Concrete exposed to moisture but not to an external source of chlorides in service	—
C2	High	Concrete exposed to moisture and to an external source of chlorides in service	For example, deicing chemicals, salt, brackish water, seawater, or spray from these sources

Source: Data adapted from *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)* Table 4.2.1 and commentary Section R4.2.1.

shall be made and cured in accordance with ASTM C192, *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*.²⁸ Cylinders shall be tested at 28 days or at test age designated for determination of f'_c .

It now reads: “For each trial mixture, at least two 6 in. × 12 in. or three 4 in. × 8 in. cylinders shall be made and cured in accordance with ASTM C192. Cylinders shall be tested at 28 days or at test age designated for f'_c .”

- Requirements (e) and (f) of ACI 318-05 read: “From results of cylinder tests a curve shall be plotted showing the relationship between water-cementitious materials ratio or cementitious materials content and compressive strength at designated test age. Maximum water-cementitious materials ratio or minimum cementitious materials content for concrete to be used in proposed Work shall be that shown by the curve to produce f'_c required by 5.3.2, unless a lower water-cementitious materials ratio or higher strength is required by Chapter 4.”

All of this has been replaced by a new item (e), which reads: “The compressive strength results, at designated test age, from the trial mixtures shall be used to establish the composition of the concrete mixture proposed for the Work. The proposed concrete shall achieve an average compressive strength as required in 5.3.2 and satisfy the applicable durability criteria of Chapter 4.”

It should be noted that ACI 318 has now recognized the use of three 4 in. × 8 in. cylinders as equivalent to the use

of two 6 in. × 12 in. cylinders. This change is also reflected in Section 5.6.2.4. The commentary clarifies that the confidence level of the average strength is preserved this way because 4 in. × 8 in. cylinders tend to have about 20% higher within-test variability than 6 in. × 12 in. cylinders. The commentary also points out that more than the minimum number of specimens may be desirable to allow for discarding an outlying individual cylinder strength in accordance with ACI 214R.

Commentary Section R6.3.2 now points out that Section 6.3.2 prohibits calcium chloride or any admixture containing chloride from being used in concrete with aluminum embedments.

Chapter 10 – Design for Fire Resistance of Precast and Prestressed Concrete

This chapter is not affected by ACI 318-08.

Chapter 11 – Thermal and Acoustical Properties of Precast Concrete

This chapter is not affected by ACI 318-08.

Chapter 12 – Vibration Design of Precast, Prestressed concrete Systems

This chapter is not affected by ACI 318-08.

Chapter 13 – Tolerances for Precast and Prestressed Concrete

To permit a more consistent application of tolerances in ACI 318 and other ACI documents, specified cover has replaced minimum cover throughout Chapter 7. This change affects sections 7.7.1, 7.7.2, 7.7.3, 7.7.4, 7.7.5, and 7.7.6.

Chapter 14 – Specifications and Standard Practices

The term *contract documents* is now defined in Chapter 2 and reads “Documents, including the project drawings and project specifications, covering the Work.”

The term *registered design professional* has been replaced with *licensed design professional*.

Because *registered design professional* is the term used in the IBC, the ACI 318 definition for *licensed design professional* is as follows: “An individual who is licensed to practice structural design as defined by the statutory requirements of the professional licensing laws of the state of jurisdiction in which the project is to be constructed and who is in responsible charge of the structural design; in other documents, also referred to as registered design professional.”

Chapter 15 – General Design Information

This chapter is not affected by ACI 318-08.

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Notation:

A_{brg} = net bearing area of the head of stud, anchor bolt, or headed deformed bar
 A_c = area of concrete section resisting shear transfer
 A_{cw} = area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear
 A_g = gross area of concrete section
 A_{se} = effective cross-sectional area of anchor (ACI 318-05)
 $A_{se,N}$ = effective cross-sectional area of anchor in tension
 $A_{se,V}$ = effective cross-sectional area of anchor in shear
 A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension b_c
 A_{vd} = total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam
 A_{vf} = area of shear-friction reinforcement
 b_c = cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh}
 b_w = web width or diameter of circular section
 c_1 = dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined
 c_2 = dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to c_1
 c_{a1} = distance from the center of an anchor shaft to the edge of concrete in one direction (if shear is applied to the anchor, c_{a1} is the maximum edge distance)
 c_{a2} = distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to c_{a1} , in.
 d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement
 d_1 = minimum dimension of confined section containing diagonal reinforcement
 d_a = outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt
 d_b = nominal diameter of bar, wire, or prestressing strand
 d_0 = outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt (ACI 318-05)
 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'_c = specified compressive strength of concrete
 f'_{ci} = specified compressive strength of concrete at time of initial prestress
 f_y = specified yield strength of nonprestressed reinforcement
 f_{yt} = specified yield strength of transverse reinforcement
 h = overall thickness or height of member
 h_{ef} = effective embedment depth of anchor
 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, in.

ℓ_{dh} = development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter), in.
 ℓ_{dt} = development length in tension of headed deformed bar, measured from the critical section to the bearing face of the head
 ℓ_n = length of clear span measured face-to-face of supports
 ℓ_o = length, measured from joint face along axis of structural member, over which special transverse reinforcement must be provided
 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
 N = tensile force
 N_n = nominal strength in tension
 N_p = pullout strength in tension of a single anchor in cracked concrete
 r = radius of gyration of cross section of a compression member
 s_o = center-to-center spacing of transverse reinforcement within the length ℓ_o
 V = shear force
 V_c = nominal shear strength provided by concrete
 V_n = nominal shear strength
 V_{sa} = nominal strength in shear of a single anchor or group of anchors as governed by the steel strength
 V_u = factored shear force at section
 W = wind load, or related internal moments and forces
 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
 $\psi_{h,v}$ = factor used to modify shear strength of anchors located in concrete members with $h_a < 1.5c_{a1}$