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The most notable changes from ACI 318-14 to ACI 318-19 for precast concrete

■ This article describes the changes in the 2019 edition of the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)* that have the greatest impact on precast concrete design and compares the 2019 requirements with the 2014 edition of ACI 318.

■ The article also offers clarifications for some of the updated requirements and describes requirements that may need to be revisited in future editions of ACI 318.

The 2022 *California Building Code* (CBC),[1] based on the 2021 *International Building Code* (IBC),[2] was adopted by the State of California and the local jurisdictions within California on January 1, 2023. Adoption by California and the cities and counties within the state is typically a major milestone for a new edition of the IBC. As other states and jurisdictions across the United States adopt the 2021 IBC, more precast concrete structures will be designed to meet the new requirements.

The 2021 IBC adopts the 2016 edition of the American Society of Civil Engineers' *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16)[3] for design loads, including seismic design provisions. This aspect of the 2021 IBC is unchanged from the 2018 IBC, except that the 2021 IBC adopts Supplement No. 1 to ASCE 7-16. The standard adopted by the 2021 IBC for reinforced concrete design and construction has been changed from the 2014 edition of the American Concrete Institute's Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14),[4] which was the standard in the 2018 IBC, to ACI 318-19.[5]

It is fairly well known that many substantive changes were made between ACI 318-14 and ACI 318-19.[6,7] Following are among the significant changes relevant to the precast concrete structures industry:

- introduction of high-strength reinforcement

- modification of straight bar development length provisions
- modification of hooked/headed bar development length provisions
- simplification of shear provisions for nonprestressed beams and slabs
- recognition of size effect on shear strength provided by concrete
- introduction of screw anchors and shear lugs
- coordination of shear strength equations for ordinary and special shear walls
- extensive additions to the foundations chapter, which now covers deep foundations
- highly important changes to the design of special shear walls
- other important changes in seismic detailing requirements

These major changes are briefly described in this paper. The new hooked bar development length provisions are shown to be problematic, with consequences for the detailing of corbels and other connections. Another problematic issue involves new special shear wall design provisions that, among other things, now require shear walls to be designed for up to three times as much shear as required by ACI 318-14. The negative impact of this requirement is the most severe when ACI 318-19 is used in conjunction with ASCE 7-16, as is required by the 2021 IBC. Some relief will be provided in the 2024 IBC,[8] which references ASCE 7-22[9] and ACI 318-19. In addition to major problems with the design of special shear walls, a significant problem with special moment frames has now surfaced.

High-strength reinforcement

Considerable effort was spent to incorporate reinforcement with yield strengths greater than 60 ksi into ACI 318-19. This update includes the higher grades in both ASTM A615/A615M, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*,[10] and ASTM A706/A706M, *Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement*.[11]

ASTM A615/A615M-18, which is referenced by ACI 318-19, recognizes four grades of reinforcement: Grades 40, 60, 80, and 100. ACI 318-19 has added supplemental requirements in Table 20.2.1.3(a), "Modified Tensile Strength and Additional Tensile Property Requirements for ASTM A615 Reinforcement." The minimum tensile strength requirements for Grades 40, 60, 80, and 100 reinforcement are 60, 80, 100, and 115 ksi (420, 550, 690, and 790 MPa), respectively. The minimum ratio of the actual tensile strength to the actual yield strength is 1.10 for all four grades.

ASTM A706/A706M-16, which is referenced by ACI 318-19, recognizes Grade 60 and Grade 80 reinforcement. ACI 318-19 has added a Grade 100 reinforcement with tensile property requirements specified in Table 20.2.1.3(b). The specified properties are as follows:

- minimum tensile strength: 117 ksi (807 MPa)
- minimum ratio of actual tensile strength to actual yield strength: 1.17
- minimum yield strength: 100 ksi (690 MPa)
- maximum yield strength: 118 ksi (814 MPa)
- minimum fracture elongation in 8 in. (200 mm): 10%

Also, bend test requirements for ASTM A706 Grade 100 reinforcement are required to be the same as the bend test requirements for ASTM A706 Grade 80 reinforcement.

ACI 318-19 has also added supplementary minimum uniform elongation requirements for ASTM A706 reinforcement. For bar sizes up to no. 10 (32M), the minimum uniform elongation requirements for Grades 60, 80, and 100 (420, 550, and 690 MPa) reinforcement are 9%, 7%, and 6%, respectively. For bar sizes no. 11, 14, and 18 (36M, 43M, and 57M), the minimum uniform elongation requirement is 6% for all grades.

ACI 318-19 section 20.2.2.5 requires the following, where f_y is specified yield strength for nonprestressed reinforcement:

> *Deformed nonprestressed longitudinal reinforcement resisting earthquake-induced moment, axial force, or both, in special seismic systems and anchor reinforcement in Seismic Design Categories (SDC) C, D, E, and F shall be in accordance with (a) or (b):*
>
> *(a) ASTM A706, Grade 60, 80, or 100 for special structural walls and Grade 60 and 80 for special moment frames.*
>
> *(b) ASTM A615 Grade 60 if (i) through (iv) are satisfied. ASTM A615 Grade 80 and Grade 100 are not permitted in special seismic systems.*
>
> *(i) Actual yield strength based on mill tests does not exceed f_y by more than 18,000 psi*
>
> *(ii) Ratio of the actual tensile strength to the actual yield strength is at least 1.25*
>
> *(iii) Minimum fracture elongation in 8 in. shall be at least 14 percent for bar sizes No. 3 through No. 6, at least 12 percent for bar sizes No. 7*

through No. 11, and at least 10 percent for bar sizes No. 14 and No. 18.

(iv) Minimum uniform elongation shall be at least 9 percent for bar sizes No. 3 through No. 10, and at least 6 percent for bar sizes No. 11, No. 14, and No. 18.

Thus, ASTM A706 Grade 100, 80, or 60 (690, 550, or 420 MPa) reinforcement is permitted to be used as flexural reinforcement in special shear walls and ASTM A706 Grade 80 or 60 reinforcement is permitted to be used as flexural reinforcement in special moment frames. Also, ACI 318 Table 20.2.2.4(a) says that ASTM A615 or A706 Grade 100, 80, or 60 reinforcement is permitted to be used as shear reinforcement in special shear walls and Grade 80 or 60 reinforcement is permitted to be used as shear reinforcement in special moment frames. ASTM A955[12] (stainless steel) Grade 60 or 80 bars or ASTM A996[13] (rail or axle steel) Grade 60 bars are also permitted to be used as shear reinforcement in both special shear walls and special moment frames.

Straight bar development length

The required development length of deformed bars and deformed wires in tension can be determined from ACI 318-19 Table 25.4.2.3 or Eq. (25.4.2.4a). In both the table and the equation, a reinforcement grade factor ψ_g has been introduced because the required development length is longer for higher-grade reinforcement. The factor ψ_g is 1.15 for Grade 80 (550 MPa) reinforcement and 1.3 for Grade 100 (690 MPa) reinforcement.

Hooked bar development length

The required hooked bar development length ℓ_{dh} of deformed bars in tension in ACI 318-14 is the largest of the following:

(a) $\left(\dfrac{f_y\psi_e\psi_r\psi_c}{50\lambda\sqrt{f_c'}}\right)d_b$ with ψ_e, ψ_c, ψ_r, and λ given in section 25.4.3.2

where

ψ_e = factor used to modify development length based on reinforcement coating

ψ_r = factor used to modify development length based on confining reinforcement

ψ_c = factor used of modify development length based on cover in ACI 318-14

λ = modification factor for lightweight concrete

f_c' = specified compressive strength of concrete, psi

d_b = nominal diameter of bar, in.

(b) $8d_b$

(c) 6 in. (150 mm)

Note that this and the following equations are unit-specific and therefore would require modification to be used with metric dimensions.

In ACI 318-19, (b) and (c) remain unchanged. However, (a) has been changed to the following:

$\left(\dfrac{f_y\psi_e\psi_r\psi_o\psi_c}{55\lambda\sqrt{f_c'}}\right)d_b^{1.5}$ with ψ_e, ψ_r, ψ_o, ψ_c, and λ given in section 25.4.3.2

where

ψ_o = reinforcement location factor

ψ_c = factor used to modify development length based on concrete strength in ACI 318-19

Thus, the required hooked bar development length is, for the first time, not a multiple of the bar diameter d_b. It is a multiple of the bar diameter raised to the power of 1.5.

Only ψ_e, the epoxy coating factor, and λ, the lightweight concrete factor, remain unchanged. ψ_c was the cover factor in ACI 318-14. It is now the concrete strength factor in ACI 318-19. Although ψ_r is still the confining reinforcement factor, it is quite different from what it was in ACI 318-14. ψ_o is a completely new reinforcement location factor. The values of ψ_c, ψ_r, and ψ_o are given in **Tables 1**, **2**, and **3**, respectively.

A_{th} in Table 2 is the total cross-sectional area of ties or stirrups

Table 1. Values of concrete strength factor in ACI 318-19 ψ_c

For f_c' < 6000 psi	f_c' /15,000 + 0.6
For f_c' ≥ 6000 psi	1.0

Note: f_c' = concrete compressive strength. 1 psi = 6.895 kPa.

Table 2. Values of confining reinforcement factor ψ_r

For no. 11 and smaller bars with $A_{th} \geq 0.4A_{hs}$ or $s \geq 6_{db}$	1.0
Other	1.6

Note: A_{hs} = total cross-sectional area of hooked or headed reinforcing bars being developed at a critical section; A_{th} = total cross-sectional area of ties or stirrups confining hooked bars; d_b = nominal diameter of bar; s = center-to-center spacing of reinforcement. No. 11 = 36M.

Hooked bar development

ACI 318-14 section 25.4.3.1 specifies the development length ℓ_{dh} for standard hooks as follows:

$$\ell_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{f'_c}}\right) d_b$$

but not less than $8d_b$ or 6 in. (150 mm), where

ψ_c = 0.7 with side cover ≥ 2.5 in. (63 mm) or end cover (90-degree hook) ≥ 2 in. (50 mm) and 1.0 otherwise

ψ_r = 0.8 where hooks are confined within ties or stirrups meeting certain requirements and 1.0 otherwise

ACI 318-19 section 25.4.3.1 specifies the development length ℓ_{dh} for standard hooks as follows:

$$\ell_{dh} = \left(\frac{f_y \psi_e \psi_r \psi_o \psi_c}{55\lambda\sqrt{f'_c}}\right) d_b^{1.5}$$

but not less than $8d_b$ or 6 in. (150 mm), where

ψ_r = 1.0 where hooks are confined within ties or stirrups meeting certain dimensions and 1.6 otherwise

ψ_o = 1.0 with side cover ≥ 2.5 in. (63 mm) within column core or side cover ≥ $6d_b$ and 1.25 otherwise

The table shows the longer hook lengths required by ACI 318-19. For example, for typical corbel primary tension reinforcement developed into a column, the required hook length for a no. 8 (25M) bar increased from 8.7 in. (221 mm) in ACI 318-14 to 14.1 in. (358 mm) in ACI 318-19, assuming the column ties are sufficient for confinement. Where it would previously have been possible to develop these bars into a 12 in. (300 mm) column using ACI 318-14, these bars do not have the necessary length to fully develop in a column of that size using ACI 318-19.

Grade 60 uncoated bars in 6000 psi normalweight concrete				
ACI 318-14				
ψ_c	0.7	0.7	1.0	1.0
ψ_r	0.8	1.0	0.8	1.0
3	3.3*	4.1*	4.6*	5.6
4	4.3*	5.4*	6.2	7.7
5	5.4*	6.8	7.7	9.7
6	6.5	8.1	9.3	11.6
7	7.6	9.5	10.8	13.6
8	8.7	10.8	12.4	15.5
9	9.8	12.2	13.9	17.4
10	11.0	13.8	15.7	19.7
11	12.2	15.3	17.5	21.8
ACI 318-19				
ψ_r	1.0	1.0	1.6	1.6
ψ_o	1.0	1.25	1.0	1.25
3	3.2*	4.0*	5.2*	6.5
4	5.0	6.2	8.0	10.0
5	7.0	8.7	11.1	13.9
6	9.1	11.4	14.6	18.3
7	11.5	14.4	18.4	23.1
8	14.1	17.6	22.5	28.2
9	16.8	21.0	26.9	33.6
10	20.2	25.2	32.3	40.3
11	23.6	29.5	37.7	47.2

* Below the minimum value of $8d_b$ or 6 in. (150 mm), shown only for illustration

Note: d_b = nominal bar diameter; ψ_c = concrete cover factor in ACI 318-14 and concrete strength factor in ACI 318-19; ψ_o = reinforcing bar location factor; ψ_r = confining reinforcement factor.

confining hooked bars. ACI 318-19 devotes section 25.4.3.3 to explaining A_{th}. Two commentary figures, Fig. R25.4.3.3a and R25.4.3.3b, have been added to help with the explanation.

The calculation of hooked bar development length in ACI 318-19 is significantly more involved than it was previously. Also, the required hooked bar development length calculated by ACI 318-19 is longer than that calculated by ACI 318-14. (See "Hooked Bar Development.")

For many years now, the required hooked bar development length at exterior beam-column joints of special moment frames has been modified in ACI 318 chapter 18, "Earthquake-Resistant Structures." In ACI 318-19, as in previous editions, section 18.8.5.1 requires that for bar sizes no. 3 through 11 (10M through 36M) terminating in a standard hook, ℓ_{dh} shall be calculated by Eq. (18.8.5.1), but ℓ_{dh} shall be at least the greater of $8d_b$ and 6 in. (150 mm) for normalweight concrete.

$$\ell_{dh} = \frac{f_y d_b}{65\lambda\sqrt{f'_c}} \qquad \text{(ACI 318-19 18.8.5.1)}$$

As noted regarding previous equations, this equation is unit-specific and would therefore require modification for use with metric dimensions.

The hook shall be located within the confined core of a column or of a boundary element, with the hook bent into the joint.

Table 3. Values of hooked reinforcing bar location factor ψ_o	
For no. 11 and smaller diameter hooked bars: • terminating inside column core with side cover normal to plane of hook ≥ 2.5 in., or • with side cover normal to plane of hook ≥ $6d_b$	1.0
Other	1.25
Note: d_b = nominal bar diameter of hooked bar. No. 11 = 36M; 1 in. = 25.4 mm.	

ACI 318-19 Commentary section R18.8.5.1 says that Eq. (18.8.5.1) is based on the requirements of section 25.4.3.

> *Because Chapter 18 stipulates that the hook is to be embedded in confined concrete, the coefficients 0.7 (for concrete cover) and 0.8 (for ties) have been incorporated in the constant used in Eq. (18.8.5.1). The development length that would be derived directly from 25.4.3 is increased to reflect the effect of load reversals. Factors such as the actual stress in the reinforcement being more than the yield strength and the effective development length not necessarily starting at the face of the joint were implicitly considered in the formulation of the expression for basic development length that has been used as the basis for Eq. (18.8.5.1).*

However, that commentary is outdated. It refers to the hooked bar development length provision of ACI 318-11.[14] The ℓ_{dh} given by Eq. (18.8.5.1) is no longer "increased to reflect the effect of load reversals" from ℓ_{dh} calculated by section 25.4.3. Thus, ACI 318-19 requires a shorter hooked bar development length at exterior beam-column joints of special moment frames than at exterior joints of ordinary and intermediate moment frames. The authors are not aware of any reports of postearthquake problems associated with insufficiency of the hooked bar development length given by Eq. (18.8.5.1). The lack of such reports would suggest that the hooked bar development length of section 25.4.3 needs to be reexamined.

Headed bar development length

In ACI 318-14, the required headed bar development length ℓ_{dt} of deformed bars in tension is the largest of the following:

(a) $\left(\dfrac{0.016 f_y \psi_e}{\sqrt{f_c'}}\right) d_b$ with ψ_e given in section 25.4.4.3

and the value of f_c' restricted to no more than 6 ksi (40 MPa)

(b) $8d_b$

(c) 6 in. (150 mm)

In ACI 318-19, (b) and (c) remain unchanged. However, (a) has been changed to the following:

$$\left(\frac{f_y \psi_e \psi_p \psi_o \psi_c}{75\sqrt{f_c'}}\right) d_b^{1.5}$$

with ψ_e, ψ_p, ψ_o, and ψ_c given in section 25.4.4.3; there is no longer any restriction on concrete strength

where

ψ_p = factor used to modify development length for headed reinforcement based on parallel tie reinforcement

Thus, as in the case of hooked bars, the required headed bar development length is no longer calculated as the multiple of bar diameter d_b. Rather, it a multiple of the bar diameter raised to the power of 1.5.

ψ_e and ψ_c are the same as in the case of hooked bar development length (see the previous section, "Hooked Bar Development Length"). The parallel tie reinforcement factor ψ_p is given in **Table 4**. ψ_o for headed reinforcement is similar to the factor for hooked bars and is given in **Table 5**.

A_{tt} in Table 4 is the total cross-sectional area of ties or stirrups acting as parallel tie reinforcement for headed bars. ACI 318-19 devotes section 25.4.4.4 to explaining A_{tt}. Two commentary figures, Fig. R25.4.4.4a and R25.4.4.4b, have been added to help with this explanation.

Table 4. Values of parallel tie reinforcement factor ψ_p	
For no. 11 and smaller bars with $A_{tt} \geq 0.3A_{hs}$ or $s \geq 6d_b$*	1.0
Other	1.6
* Refer to ACI 31-19 section 25.4.4.5. Note: A_{hs} = total cross-sectional area of hooked or headed reinforcing bars being developed at a critical section; A_{tt} = total cross-sectional area of ties or stirrups confining hooked bars; d_b = nominal diameter of bar; s = center-to-center spacing of reinforcement. No. 11 = 36M.	

Table 5. Values of headed reinforcing bar location factor ψ_o	
For headed bars: • terminating inside column core with side cover to bar ≥ 2.5 in., or • with side cover to bar ≥ $6d_b$	1.0
Other	1.6
Note: d_b = nominal bar diameter of headed bar. 1 in. = 25.4 mm.	

The calculation of headed bar development length in ACI 318-19 has become significantly more involved than it was in previous editions. Fortunately, the required headed bar development length calculated by ACI 318-19 comes out to be shorter than that calculated by ACI 318-14.

Recognition of size effect on shear strength provided by concrete

It has been known for quite some time that the nominal shear strength provided by concrete V_c does not change in direct proportion to member depth, contrary to what is suggested by conventional expressions such as follows:

$$V_c = 2\lambda\sqrt{f'_c}b_w d$$

where

b_w = web width of the component, in.

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

That equation incorrectly gives the impression that doubling the member depth leads to twice the concrete contribution toward resisting shear.

ACI 318-19 introduces a new size effect modification factor λ_s, to modify the expressions to calculate V_c for the following:

- beams without shear reinforcement
- two-way slabs with or without shear reinforcement

$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \le 1 \qquad \text{(ACI 318-19 22.5.5.1.3)}$$

For example, when effective depth $d \le 10$ in. (250 mm), λ_s = 1; or for d = 30 in. (760 mm), λ_s = 0.707.

The size effect factor λ_s also applies to the two-way or punching shear strength provided by concrete in two-way slabs with or without shear reinforcement.

Modified shear provisions for nonprestressed beams and slabs

In ACI 318-14 and prior editions of the standard, the nominal shear strength provided by concrete in nonprestressed beams and one-way slabs was determined by eight different equations. In ACI 318-19, those equations are replaced by just the three equations presented in **Table 6**.

ACI 318-19 requires that V_c shall not be taken greater than $5\lambda\sqrt{f'_c}b_w d$ and that the value of $N_u/6A_g$ (where N_u is the factored axial force normal to cross section occurring simultaneously with the factored shear force or torsion at the section V_u or T_u; to be taken as positive for compression and negative for tension, lb, and A_g is the gross area of concrete section, in.^{2}) shall not be taken greater than $0.05f'_c$. $A_{v,min}$ is the minimum shear reinforcement required by ACI 318.

Table 6. V_c for nonprestressed members

Criteria	V_c		
$A_v \ge A_{v,min}$	Either of	$\left[2\lambda\sqrt{f'_c}+\frac{N_u}{6A_g}\right]b_w d$	(a)
		$\left[8\lambda(\rho_w)^{1/3}\sqrt{f'_c}+\frac{N_u}{6A_g}\right]b_w d$	(b)
$A_v < A_{v,min}$	$\left[8\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c}+\frac{N_u}{6A_g}\right]b_w d$		(c)

Source: ACI 318-19 Table 22.5.5.1.

Note: V_c shall not be taken less than zero. A_g = gross area of concrete section; A_v = area of shear reinforcement within spacing s; $A_{v,min}$ = minimum area of shear reinforcement within spacing s; b_w = web width of the component; d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement; f'_c = specified compressive strength of concrete; N_u = factored axial force normal to cross section occurring simultaneously with V_u or T_u, to be taken as positive for compression and negative for tension; λ = modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normalweight concrete of the same compressive strength; λ_s = factor used to modify shear strength based on the effects of member depth, commonly referred to as the size effect factor; ρ_w = longitudinal reinforcement ratio, equal to ratio of A_s to $b_w d$.

The new expressions work well for varying amounts of shear reinforcement provided A_v. They also work well with varying axial compressive stresses applied. Expressions (b) and (c) account for longitudinal reinforcement ratios in members. ACI 318-19 section 22.5.3.1 still limits concrete strength to 10,000 psi (69 MPa) for the purpose of calculating concrete shear strength, unless enhanced minimum transverse reinforcement is provided.

The combination of the modified shear strength expression (c) and the new size effect modification factor λ_s, has significantly reduced the available concrete shear strength for foundations and retaining walls, which generally do not include shear reinforcement.

Volume change restraint in precast concrete connections

ACI 318-14 and earlier editions did not consistently define the design horizontal force at bearing connections due to volume change effects, or applicable factored load combinations. At brackets and corbels, ACI 318-14 and prior editions specified the horizontal force as 0.2 times the factored shear force V_u, with the calculated force treated as a live load. For several editions, the *PCI Design Handbook: Precast and Prestressed Concrete*[15] has defined this force when bearing pads are used as 0.2 times the sustained load portion of V_u.

ACI 318-19 section 16.2, "Connections of Precast Members," includes a new subsection, 16.2.2.3, specifying the required strength at bearing connections. For connections on bearing pads, the force is 20% of the sustained unfactored vertical reaction multiplied by a load factor of 1.6. In addition, a maximum restraint force is also specified to provide a capacity-design limit where the bearing medium has a known slip or deformation limit.

Introduction of screw anchors and shear lugs

ACI 318-19 chapter 17 now covers screw anchors (**Fig. 1**) and shear lugs (**Fig. 2**). The design procedure for screw anchors is exactly the same as that for postinstalled expansion or undercut anchors. Separate values for various design parameters are provided.

Shear lugs consist of steel plates welded to an attachment base plate. Shear is transferred through direct bearing or concrete breakout. Tension is transferred through separate anchor bolts/studs (minimum four anchors).

Shear strength equations for ordinary and special shear walls

Ordinary shear walls, which are permitted in buildings assigned to Seismic Design Categories (SDCs) A, B, and C, are covered in chapter 11 of ACI 318-14 and ACI 318-19. Special detailing is required in buildings assigned to SDC D, E, or F. Special shear walls are covered in section 18.10 of ACI 318-14 and ACI 318-19.

The nominal shear strength of an ordinary shear wall, as given in section 11.5.4 of ACI 318-14, is quite different from that of a special shear wall, as given in section 18.10.4 of ACI 318-14. There is no reason or explanation for this difference. Sections 11.5.4 and 18.10.4 of ACI 318-19 have now been harmonized. Equation (11.5.4.3) for the nominal shear strength V_n is the same as Eq. (18.10.4.1) in ACI 318-19. This is a welcome change.

Deep foundation design provisions

ACI 318-19 contains provisions for the design of deep foundations. These provisions are basically transferred from chapter 18 of the editions prior to the 2021 IBC, although with some modifications. In ACI 318-19, the nonseismic provisions are in chapter 13 and the seismic provisions are in section 18.13.

For precast concrete piles, ACI 318-19 section 13.4.5 specifies requirements for piles supporting buildings assigned to SDC A or B, whereas section 18.13.5.10 specifies requirements for piles supporting buildings assigned to SDC C, D, E, or F. The requirements for transverse reinforcement in piles resisting

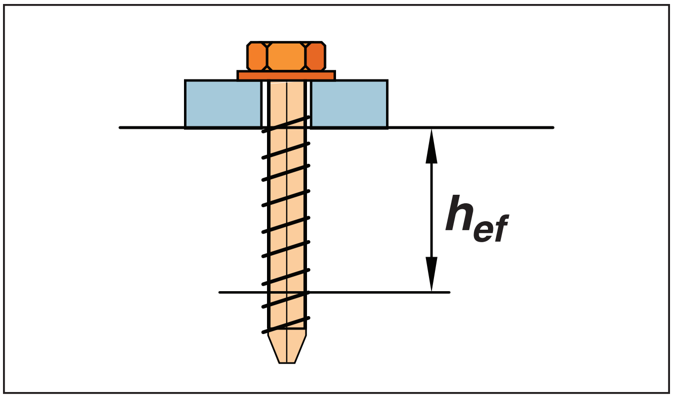

Figure 1. Screw anchor. Note: h_{ef} = effective embedment depth of anchor. Source: Adapted from ACI 318-19, Fig. R2.1.

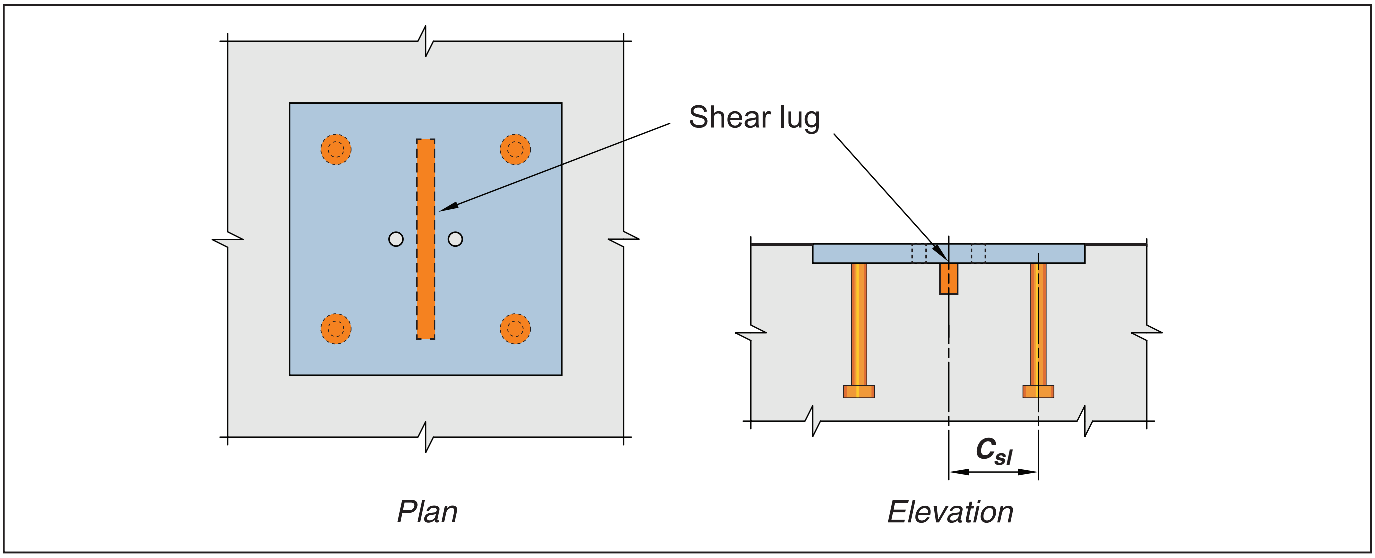

Figure 2. Shear lug. Note: C_{sl} = offset of studs from shear lug, centerline to centerline. Source: Adapted from ACI 318-19 Fig. 17.11.1.1a.

seismic effects is based on the work in Sritharan et al.[16] that relates pile curvature ductility demand to overall system ductility demand. The requirements also align with PCI's "Recommended Practice for Design, Manufacture, and Installation of Prestressed Concrete Piling."[17]

The deep foundation design requirements included in ACI 318-19 are omitted from chapter 18 of the 2021 IBC and 2024 IBC. In both editions, IBC section 1810.3.8 on precast concrete piles simply requires that they shall be designed and detailed in accordance with ACI 318. However, two exceptions are left in the 2021 IBC and 2024 IBC. The first exception permits precast and prestressed concrete piles supporting buildings in SDC C to be detailed in the same manner as piles supporting building in SDC B if they incorporate a prescribed amount of overstrength. Similarly, the second exception permits piles supporting SDC D, E, or F to be detailed in the same manner as piles supporting buildings in SDC B as long as the piles are designed for a prescribed amount of overstrength.

Important changes in seismic detailing requirements other than those for special shear walls

The following are the important changes in chapter 18 of ACI 318-19, excluding section 18.10:

- There are new design and detailing requirements for joints of ordinary moment frames that are part of the seismic-force-resisting system of an SDC B structure.
- The use of Grade 80 (550 MPa) reinforcement is recognized in specifying the spacing of column hoops in intermediate moment frames.
- The design and detailing requirements for joints of intermediate moment frames are much more extensive than in previous editions.
- The maximum longitudinal reinforcement ratio in special moment frame beams continues to be 0.025 for Grade 60 (420 MPa) reinforcement, but it is now 0.02 for Grade 80 reinforcement.
- The use of Grade 80 reinforcement is recognized in specifying hoop spacing in potential hinging regions of special moment frame beams.
- The strong column–weak beam requirement does not have to be satisfied at a beam-column joint where the column is discontinued above the joint and the earthquake-induced compressive axial force in the column is less than $A_g f'_c/10$.
- There is an important new requirement that over the clear height of a special moment frame column, the longitudinal reinforcement shall be selected such that $1.2\,\ell_d \leq \ell_u/2$, where ℓ_d is the development length and ℓ_u is the unsupported column height. This requirement is proving to be problematic from a constructibility point of view because a large number of small-diameter bars are typically required to satisfy the requirements and because these bars must be tied and cross-tied, as required for proper confinement.
- The use of Grade 80 reinforcement is recognized in specifying the maximum hoop spacing in potential hinging regions as well as the maximum tie spacing outside of the potential plastic hinge regions of special moment frame columns.
- Where longitudinal beam reinforcement simply passes through an interior column, the depth of the joint parallel to the beam longitudinal reinforcement was required in ACI 318-14 to be at least 20 times the diameter of the largest longitudinal beam bar for normalweight concrete or 26 times the same for lightweight concrete. It is now required to be the largest of (a) through (c):

  (a) $(20/\lambda)d_b$ of the largest Grade 60 longitudinal bar, where λ is 0.75 for lightweight concrete and 1.0 for all other cases

  (b) $26d_b$ of the largest Grade 80 longitudinal bar

  (c) $h/2$ of any beam framing into the joint and generating joint shear as part of the seismic-force-resisting system in the direction under consideration, where h is the beam height
- Concrete used in joints with Grade 80 longitudinal reinforcement is required to be normalweight concrete.
- The nominal joint shear strength V_n, which was given in ACI 318-14 Table 18.8.4.1, is now given in ACI 318-19 Table 18.8.4.3, which is significantly more elaborate. Table 18.8.4.3 applies to joints of special moment frames as well as intermediate moment frames. There is a similar Table 15.4.2.3 for ordinary moment frame joints.

Changes in design and detailing requirements for special shear walls

It is common knowledge that major changes in the design and detailing requirements for special shear walls were made in ACI 318-19.[6,7] Among the most important of these changes are the following:

- Shear amplification: design shear (required shear strength) V_u from ACI 318-14 must be amplified by a factor of up to 3 (similar to codes from New Zealand and Canada).
- More-stringent wall boundary and wall web detailing: the requirements are overlapping hoops if the boundary zone dimensions exceed 2:1, crossties with seismic (135-degree) hooks on both ends, and 135-degree crossties on web vertical bars.

- Check on mean top-of-wall drift capacity at 20% loss of lateral strength: a low probability of lateral strength loss at a maximum-considered-earthquake-level hazard is required.

- Minimum wall boundary longitudinal reinforcement: this is required to limit the potential of brittle tension failures of walls that are lightly reinforced.

- Severe restrictions on lap slice locations.

Some of the changes have caused serious concerns among practitioners, who are finding it difficult or impractical to implement revised requirements. Coauthor Ghosh presented on this topic at the 2022 Structural Engineers Association of California (SEAOC) Convention, and a related paper by Ghosh and Lehman is included in the SEAOC convention's *Proceedings*.[18] A near-consensus has emerged that adjustments to some of the ACI 318-19 changes are badly needed. The 2022 paper and presentation discussed a number of these needed adjustments. (Material from the 2022 convention paper is not repeated here.)

ACI has received written requests for clarification of the intent or application of the special shear wall design provisions of ACI 318-19. Requests were submitted by the Structural Engineers Association of Washington, Structural Engineers Association of Northern California, and Los Angeles Tall Buildings Structural Design Council. In response, ACI Committee 318 has developed, balloted, and approved three Code Cases to clarify the relevant code sections. These were published in the June 2023 issue of *Concrete International*.[19]

Problems remain. The Code Cases have helped but only to a certain extent. It is anticipated that ACI 318-25 will provide more significant relief. That standard will be adopted by the 2027 IBC, which will form the basis of the 2028 *California Building Code*, scheduled to go into effect in California in early January 2029. That obviously is a long time away.

Precast concrete diaphragms subjected to earthquake effects

ACI 318-19 fully incorporates research findings from the DSDM (Diaphragm Seismic Design Methodology) into the design standards. The incorporation consists of the following four parts:

- specifying alternative diaphragm design forces in ASCE 7-16 and ASCE 7-22 section 12.10.3

- adding section 18.12.11 to ACI 318-19, which "shall apply to diaphragms constructed using precast concrete

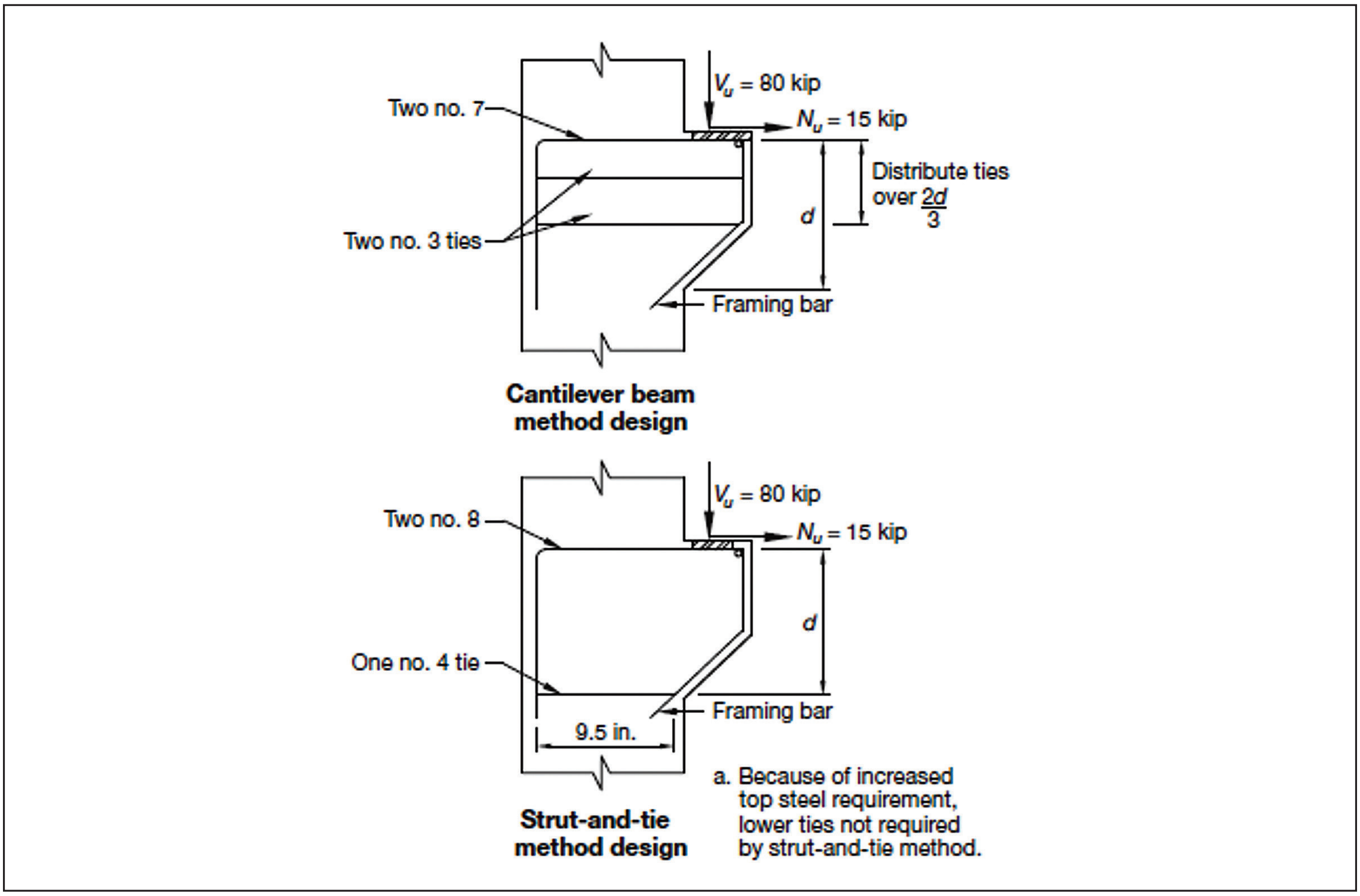

Figure 3. Comparison of corbel design methods. Source: Reproduced from *PCI Design Handbook: Precast and Prestressed Concrete*, Fig. 5.7.4. Note: d = effective depth; N_u = factored horizontal force applied to corbel; V_u = factored shear (vertical) force applied to corbel. No. 3 = 10M; no. 4 = 13M; no. 7 = 22M; no. 8 = 25M; 1 in. = 25.4 mm; 1 kip = 4.448 kN.

members and forming part of the seismic-force-resisting system for structures assigned to SDC C, D, E, or F"

- specifying the design criteria for precast concrete diaphragms via *Code Requirements for the Design of Precast Concrete Diaphragms for Earthquake Motions (ACI 550.5) and Commentary (ACI 550.5R)*[20]
- specifying the connector testing criteria via *Qualification of Precast Concrete Diaphragm Connections and Reinforcement at Joints for Earthquake Loading (ACI 550.4-18) and Commentary (ACI 550.4R-18)*[21]

Ghosh[22] provides additional background on the codification of the DSDM research and discusses additional refinements of the design criteria for precast concrete diaphragms.

Minimum reinforcement in discontinuity regions

The design of corbels may be based on either the cantilever beam method (ACI 318-19 section 16.5) or the strut-and-tie method (ACI 318-19 chapter 23). Corbels designed in accordance with the strut-and-tie method of ACI 318-14 require a larger amount of top reinforcement to resist the top tension but do not require distributed ties over the corbel depth (**Fig. 3**). In ACI 318-19, a requirement was added for minimum distributed reinforcement across struts in discontinuity regions, except where the strut is laterally restrained (such as in a continuous corbel or ledge). The result of this change is indicated in the ACI 318-19 commentary (**Fig. 4**).

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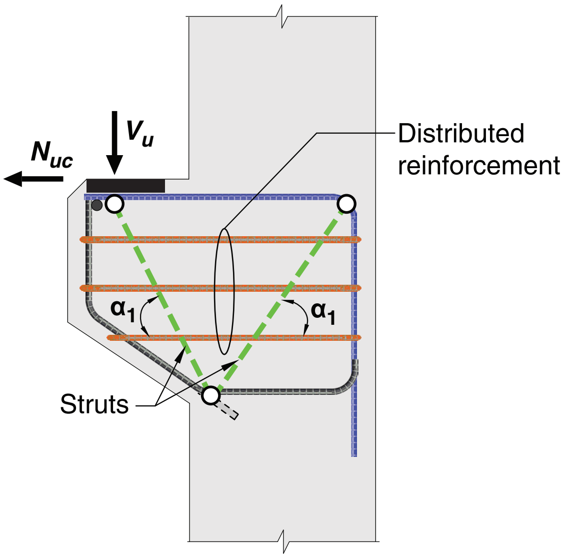

Distributed reinforcement crossing interior struts. Note that α_1 *is different for the two struts above; the minimum distributed reinforcement ratio is controlled by the smaller angle* α_1.

Figure 4. Distributed reinforcement in discontinuity regions. Source: Reproduced from ACI 318-19, Fig. R23.5.1. Note: N_{uc} = factored axial force normal to cross section occurring simultaneously with V_u or T_u; to be taken as positive for compression and negative for tension, lb; V_u = factored shear force at section, lb; α_1 = minimum angle between unidirectional distributed reinforcement and a strut.

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Notation

A_g = gross area of concrete section, in.^{2}

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

A_s = area of nonprestressed longitudinal tension reinforcement, in.^{2}

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

A_{tt} = total cross-sectional area of ties or stirrups acting as parallel tie reinforcement for headed bars, in.^{2}

A_v = area of shear reinforcement within spacing s, in.^{2}

$A_{v,min}$ = minimum area of shear reinforcement within spacing s, in.^{2}

b_w = web width of the component

C_{sl} = offset of studs from shear lug from centerline to centerline

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

d_b = nominal diameter of bar, in.

f'_c = specified compressive strength of concrete, psi

f_y = specified yield strength for nonprestressed reinforcement, psi

h = overall thickness, height, or depth of member, in.

h_{ef} = effective embedment depth of anchor, in.

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 outside end of hook, point of tangency, toward critical section, in.

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

ℓ_u = unsupported length of column or wall, in.

N_u = factored axial force normal to cross section occurring simultaneously with V_u or T_u; to be taken as positive for compression and negative for tension, lb

N_{uc} = factored restraint force applied to a bearing connection acting perpendicular to and simultaneously with V_u, to be taken as positive for tension, lb

s = center-to-center spacing of reinforcement, in.

T_u = torsion at the section

V_c = nominal shear strength provided by concrete, lb

V_n = nominal shear strength, lb

V_u = factored shear force at section, lb

α_1 = minimum angle between unidirectional distributed reinforcement and a strut

λ = modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normalweight concrete of the same compressive strength

λ_s = factor used to modify shear strength based on the effects of member depth, commonly referred to as the size effect factor

ρ_w = longitudinal reinforcement ratio, equal to ratio of A_s to $b_w d$

ψ_c = factor used to modify development length based on cover in ACI 318-14 and factor used to modify development length based on concrete strength in ACI 318-19

ψ_e = factor used to modify development length based on reinforcement coating

ψ_g = factor used to modify development length based on grade of reinforcement

ψ_o = factor used to modify development length of hooked and headed bars based on side cover and confinement

ψ_p = factor used to modify development length for headed reinforcement based on parallel tie reinforcement

ψ_r = factor used to modify development length based on confining reinforcement

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Abstract

This paper discusses selected changes from ACI 318-14 to ACI 318-19 that are likely to have the largest impact on precast concrete design and construction. Some of these changes have proved to be somewhat problematic. These concerns are described, and in a few cases, suggestions are offered that might mitigate some of the negative impact.

Keywords

ACI 318, design, development length, shear strength.

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