



HIGH-STRENGTH CONCRETE IN U.S. CODES AND STANDARDS

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ABSTRACT

The latest edition of ACI 318 Building Code Requirements for Structural Concrete contains several provisions related specifically to high-strength concrete, that partly reflect insufficient knowledge concerning the shear (diagonal tension) strength of concrete and the bond between reinforcing steel and concrete in the ranges of very high concrete strength. This paper details and discusses these provisions. It then explores areas where changes in ACI 318 provisions may be needed to ensure continued satisfactory usage of high-strength concrete in structural applications.

DEFINITION OF HIGH-STRENGTH CONCRETE

The definition of high-strength concrete has changed over the years, and should not be considered static. The precise strength defining high-strength concrete also tends to vary by geographic location. In a 1984 American Concrete Institute (ACI) committee report, revised and reissued in 1992¹, 6,000 psi (41 MPa) was selected as a lower limit for high-strength concrete. According to that report, although 6,000 psi (41 MPa) was selected as the lower limit, it was not intended to imply that there is a drastic change in material properties or in production techniques that occur at this compressive strength. In reality, all changes that take place above 6,000 psi (41 MPa) represent a process which starts with the lower-strength concretes and continues with high-strength concretes. ACI Committee 363 is now in the process of updating their state-of-the-art report on high-strength concrete. In the draft update of ACI 363R-92, high-strength concrete has been defined as concrete having a specified compressive strength for design of 8,000 psi (55 MPa), or greater.

As the strength of concrete increases, it becomes more and more sensitive to quality control procedures. Close attention to each facet of concrete production becomes more important with increasing concrete strength.

UPPER LIMIT ON THE STRENGTH OF CONCRETE

Neither ACI 318-02² nor any of the model codes in the United States (the *Uniform Building Code*³, the *BOCA/National Building Code*⁴, the *Standard Building Code*⁵, the *International Building Code*⁶, and the *NFPA 5000 Building Construction and Safety Code*⁷) imposes an upper limit on the strength of normal-weight concrete that can be used in construction, even in regions of high seismicity. Only the City of Los Angeles in the past informally imposed a limit of 6,000 psi (41 MPa) on the specified compressive strength (f'_c) of concrete used in special (meaning specially detailed) moment frames.

ACI 318-02 requires that the specified compressive strength of lightweight-aggregate concrete used in the design of members of special moment frames in any seismic zone³ or seismic performance category^{4,5} or seismic design category,^{6,7} and of members of seismic-force-resisting systems in seismic zones 3, 4,³ seismic performance categories D and E^{4,5} or seismic design categories D, E, and F^{6,7} shall not exceed 5,000 psi (35 MPa). Lightweight-aggregate concrete with higher design compressive strength may be used if demonstrated by experimental evidence that structural members made with that lightweight-aggregate concrete provide strength and toughness equal to or exceeding those of comparable members made with normal-weight aggregate concrete of the same strength. The *Uniform Building Code* imposes an absolute upper limit of 6,000 psi (41 MPa) on the specified compressive strength of lightweight concrete.

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MODULUS OF ELASTICITY

According to ACI 318-02 Section 8.5.1, the modulus of elasticity E_c for concrete may be taken as $w_c^{1.5} 33 \sqrt{f'_c}$ (in psi) or $w_c^{1.5} 0.043 \sqrt{f'_c}$ (in MPa) for values of w_c (unit weight of concrete) between 90 and 155 pounds per cubic foot or 1,500 and 2,500 kg/m³. This formula does not make any distinction between high-strength and lower-strength concretes.

In view of test results from Cornell University⁷ indicating that the moduli of elasticity of concrete with very high levels of strength might be lower than values given by the ACI 318 provision, the following formula proposed by Cornell researchers was endorsed by ACI Committee 363¹:

$$E_c = 40,000 \sqrt{f'_c} + 1,000,000 \text{ psi} \quad \text{for } 3,000 \text{ psi} < f'_c < 12,000 \text{ psi} \quad (1)$$

$$(E_c = 3,320 \sqrt{f'_c} + 6,900 \text{ MPa} \quad \text{for } 21 \text{ MPa} < f'_c < 83 \text{ MPa})$$

It should be noted that ACI 318-02 has not adopted the ACI 363 formula. There is evidence⁹ suggesting that this was probably a prudent decision.

MODULUS OF RUPTURE

According to ACI 318-02 Section 9.5.2.3, the modulus of rupture of normal-weight concrete is:

$$f_r = 7.5 \sqrt{f'_c} \quad (2)$$

$$(f_r = 0.7 \sqrt{f'_c})$$

Again, this formula does not make any distinction between high-strength and normal-strength concretes, although there are indications from Cornell research¹⁰ that the constant relating f_r and f'_c might be higher for concretes with very high strength levels.

LONG-TERM DEFLECTION MULTIPLIER

ACI 318-02 Section 9.5.2.5 requires that unless values are obtained by a more comprehensive analysis, additional long-term deflection resulting from creep and shrinkage of flexural members (normal-weight or lightweight concrete) shall be determined by multiplying the immediate deflection, caused by the sustained load considered, by the factor:

$$\lambda = \frac{\xi}{1 + 50\rho'} \quad (3)$$

where the compression reinforcement ratio ρ' shall be the value at midspan for simple and continuous spans, and at support for cantilevers. The time-dependent factor ξ for sustained loads may be taken equal to:

| | | |
|-----|---------------------------|-----------------|
| 2.0 | for a loading duration of | 5 years or more |
| 1.4 | for a loading duration of | 12 months |
| 1.2 | for a loading duration of | 6 months |
| 1.0 | for a loading duration of | 3 months |

Shrinkage values for normal-strength and high-strength concretes are basically comparable. Creep per unit stress (specific creep), however, decreases significantly as concrete strength increases.¹ This fact is not reflected in the long-term deflection multiplier (ξ) given by the Code. It is believed that the multiplier might be in the right range for high-strength concrete members, and might be unconservative for moderate-to-low-strength concrete members.

STRENGTH DESIGN OF MEMBERS FOR FLEXURE AND AXIAL LOADS

The strength design of members subject to flexure and axial loads is usually based on the equivalent rectangular concrete stress distribution defined in ACI 318. According to Section 10.2.7.1, a concrete stress of $0.85f'_c$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1c$ from the fiber of maximum compressive strain (Fig. 1). Factor β_1 shall be taken as 0.85 for concrete strengths f'_c up to and including 4,000 psi (28 MPa). For strengths above 4,000 psi (28 MPa), β_1 shall be reduced continuously at a rate of 0.05 for each 1,000 psi (7 MPa) of strength in excess of 4,000 psi (28 MPa), but β_1 shall not be taken less than 0.65.

The applicability of the rectangular stress block defined above for the strength computation of high-strength members subject to flexure with or without axial loads has often been questioned. ACI 318-02 does not specify an upper concrete strength limit beyond which the rectangular stress block becomes inapplicable. This is justified for

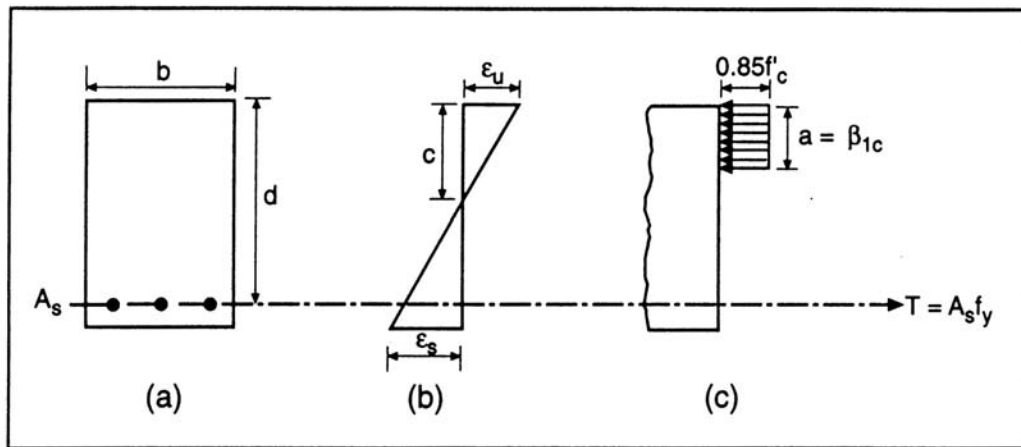


Figure 1 Equivalent rectangular stress block of ACI 318

flexural members not subject to an axial compression. That is because the ACI 318 standard generally requires flexural members to be underreinforced, which translates into shallow neutral axis depths and small concrete compression zones. The behavior of underreinforced concrete flexural members is governed almost entirely by the tension reinforcement. What shape is assumed for the compression concrete stress block becomes largely immaterial.

The ACI rectangular stress block yields acceptable strength prediction in flexure and axial compression as long as $f'_c < 8,000$ psi (55 MPa). This can be seen in Fig. 2 from Ref. 11, which compares 93 tests of eccentrically loaded columns to the strengths predicted by the ACI Code. Because some of the tests are for unreinforced concrete, and others for reinforced, only the strength contributed by concrete has been compared, the steel contribution having been subtracted from both the test and predicted values. In this figure the comparisons are made in terms of $\delta = (\text{test strength}/\text{calculated strength}) - 1.0$.

For $f'_c > 8,000$ psi (55 Mpa), in the absence of substantive enhancement in the current confinement requirements, accurate strength prediction for members subject to combined bending and axial compression would appear to require an adjustment of the ACI rectangular stress block. The importance of confinement is illustrated in Fig. 3 from Ref. 12, which shows the relationship between the parameter $\rho_s f_{yh}/f'_c$ and the ratio of experimentally obtained strength of 111 high-strength concrete columns to that predicted by the ACI Code. ρ_s is the volumetric ratio of transverse reinforcement and f_{yh} is the yield strength of transverse

reinforcement. It is clear that columns with a low volumetric ratio of transverse reinforcement may not achieve their strength as predicted by the ACI Code; however, well confined columns can develop strength well in excess of that predicted by the Code. It should be noted that excess strength of columns with relatively high amounts of transverse reinforcement is generally obtained after spalling of the cover concrete. This strength enhancement comes as a result of an increase in strength of the confined core concrete.

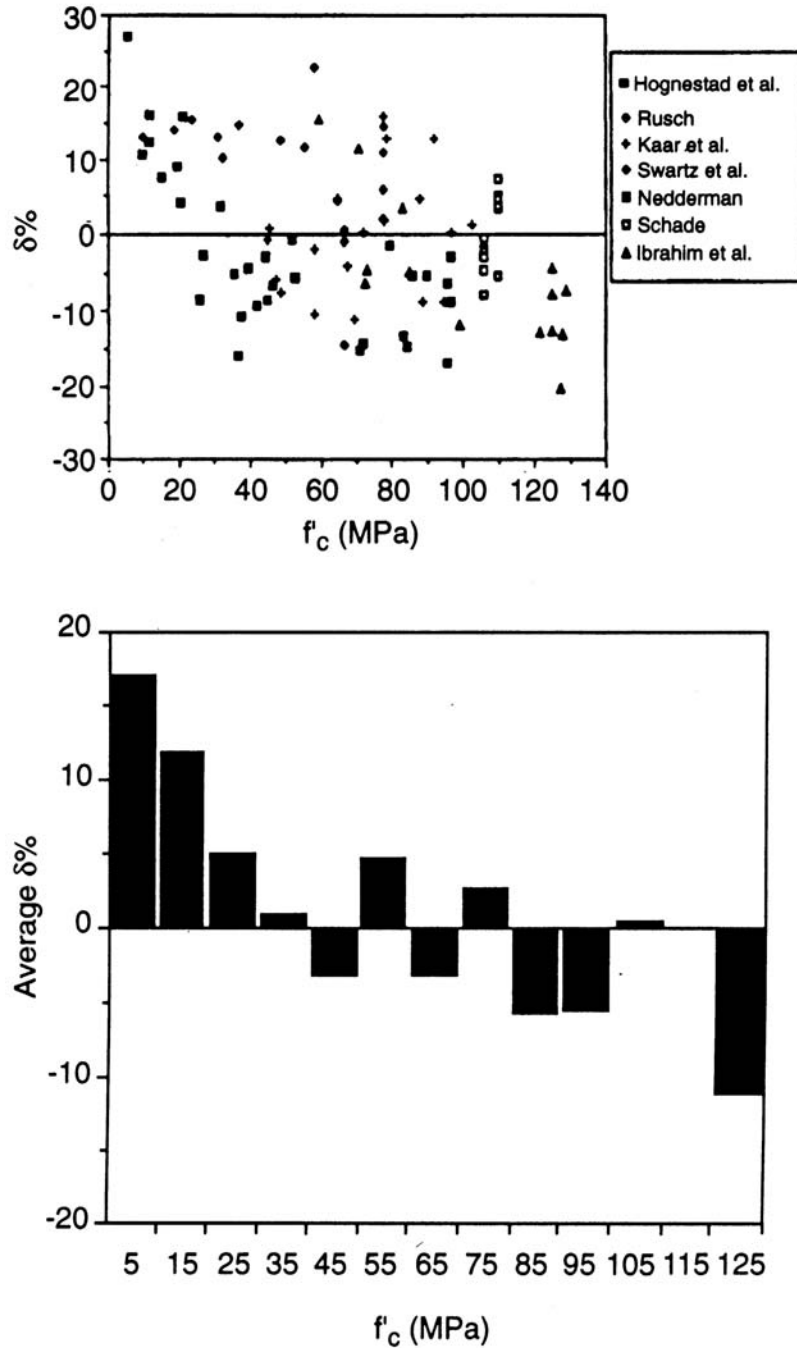


Figure 2 (a) Comparison of strength from tests of eccentrically loaded columns with that given by ACI 318, (b) Mean values of δ versus concrete strength

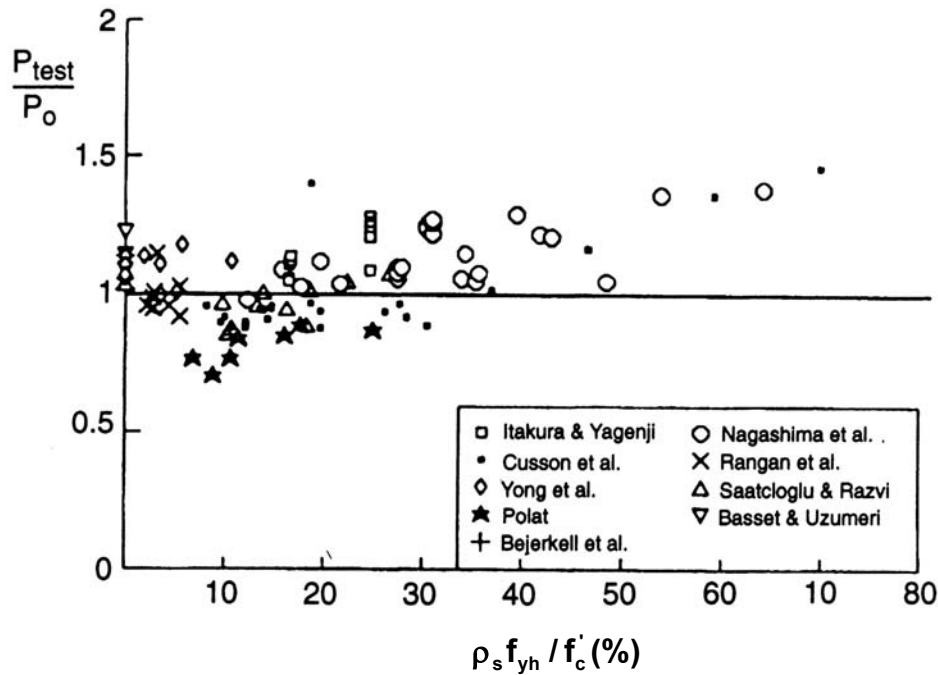


Figure 3 Comparisons of experimental and analytical concentric tests of columns

The latest New Zealand Concrete Design Standard¹³ has adopted a specific adjustment of the ACI rectangular stress block for high-strength concrete. The latest Canadian Standards Association Standard¹⁴ has adopted a different adjustment. Other modifications of the ACI rectangular stress block have also been proposed.¹⁵⁻¹⁷

An Innovation Task Group (ITG-4) has been formed within ACI with the purpose of developing a non-mandatory standard on the use of high-strength concrete in moderate to high seismic applications. Among the conclusions drawn by the ITG, the following is relevant to the current discussion.

According to the ITG, it is apparent from a review of existing literature that if the equivalent rectangular stress block of ACI 318-02 is used, the ratio of nominal to experimental column strength decreases as the axial load increases. Experimental results indicate that the nominal moment and axial load strengths of columns calculated with the ACI 318-02 rectangular stress block may be unconservative for compressive strengths greater than 12,000 psi (83 MPa).

From the point of view of seismic design, two consequences of overestimating the flexural strengths of columns are that the shear demand on the column calculated on the basis of the probable flexural strength is overestimated and that the ratio of column-to-beam moment strengths is overestimated. Overestimating the shear demand is conservative because it will lead to a higher amount of transverse reinforcement. Conversely, overestimating the ratio of column-to-beam moment strengths has a negative effect, because it will increase the probability of hinging in the columns. ACI 318-02 requires a minimum ratio of column-to-beam moment strengths of 1.2. Overestimating column flexural strength will decrease that ratio and may even result in a strong beam-weak column mechanism.

Because experimental results showed that the ACI 318-02 rectangular stress block is safe for normal-strength concrete, a recommendation was developed focusing on columns with compressive strengths greater than 8,000 psi (55 MPa). This was done by suggesting a stress block with a variable stress intensity

factor α_1 for concrete compressive strengths greater than 8,000 psi (55 MPa). Accordingly, it has been recommended that: “factor α_1 shall be taken as 0.85 for concrete strengths f'_c up to and including 8,000 psi (55 MPa). For strengths above 8,000 psi (55 MPa), α_1 shall be reduced continuously at a rate of 0.015 for each 1,000 psi (7 MPa) of strength in excess of 8,000 psi (55 MPa), but α_1 shall not be taken as less than 0.70.” The parameter β_1 , which defines the depth of the stress block, was not changed. Figure 4 compares the proposed stress block parameter α_1 with the ACI 318-02 value, as well as with the results of tests on columns with concrete strengths of up to 18,000 psi (124 MPa).

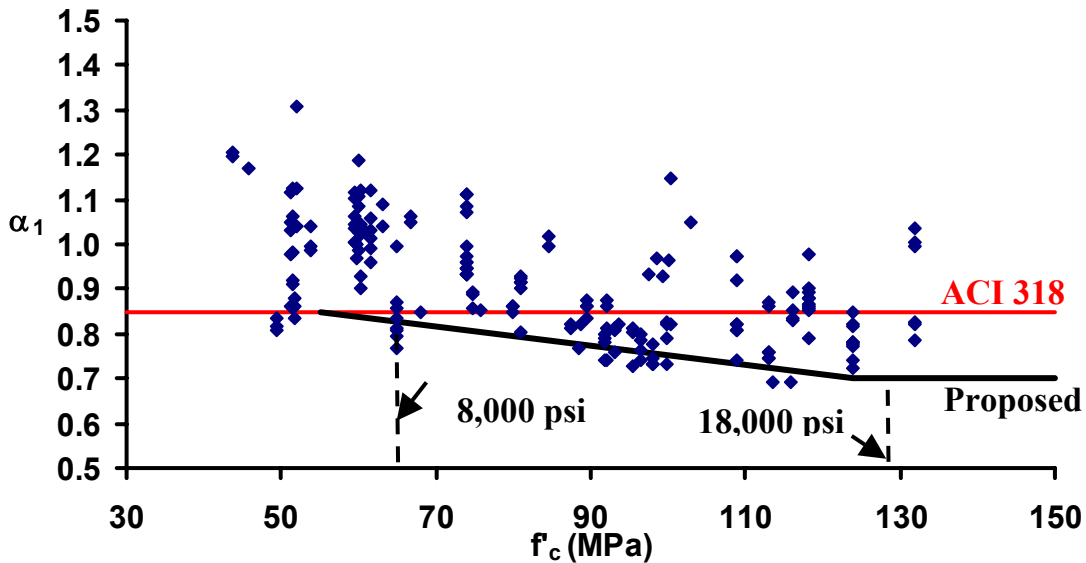


Figure 4 Stress intensity factors for concentrically tested columns having different levels of confinement

MINIMUM REINFORCEMENT FOR FLEXURAL MEMBERS

The provisions for a minimum amount of reinforcement are meant to apply to flexural members that, for architectural or other reasons, are much larger in cross section than required for strength. With a very small amount of tensile reinforcement, the computed moment strength as a reinforced concrete section using cracked section analysis becomes less than that of the corresponding unreinforced concrete section computed from its modulus of rupture. Failure in such a case can be sudden. To prevent such a failure, a minimum amount of tensile flexural reinforcement is required, and should be provided in both positive and negative moment regions.

So in ACI 318-95 the minimum reinforcement was made a function of the specified compressive strength of the concrete. ACI 318-02 Section 10.5.1 requires that at every section of a flexural member where tensile reinforcement is required by analysis, except as provided in 10.5.2, 10.5.3 and 10.5.4, the area A_s provided shall not be less than that given by

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d \tag{5}$$



$$(A_{s,\min} = \frac{\sqrt{f'_c}}{4f_y} b_w d)$$

and not less than $200 b_w d / f_y$ ($1.4 b_w d / f_y$).

The $200/f_y$ ($1.4/f_y$) value was originally derived to provide the same 0.5 percent minimum (for mild grade steel) required in older editions of the ACI Code. Indications were that, when concrete strength higher than about 5,000 psi (34 MPa) is used, the $200/f_y$ ($1.4/f_y$) value may not be sufficient.

Section 10.5.2 requires that for a statically determinate T-section with the flange in tension, the area $A_{s,\min}$ shall be equal to or greater than the value given by Eq. (5) with b_w replaced by either $2b_w$ or the width of the flange, whichever is smaller.

Section 10.5.3 allows that the requirements of 10.5.1 and 10.5.2 need not apply if at every section the area of tensile reinforcement provided is at least one-third greater than that required by analysis.

Section 10.5.4 provides that for structural slabs and footings of uniform thickness the minimum area of tensile reinforcement in the direction of the span shall be the minimum shrinkage and temperature reinforcement required by 7.12. Maximum spacing of this reinforcement shall not exceed three times the thickness nor 18 in. (487 mm).

SHEAR STRENGTH

ACI 318-95 Chapter 11 on Shear and Torsion restricts the values of $\sqrt{f'_c}$ to no more than 100 psi (25/3 MPa), meaning that the contribution of concrete to the shear strength of a structural member will not increase any further, once the specified compressive strength of concrete goes above 10,000 psi (69 MPa). There is an important exception to this restriction, however.

Values of $\sqrt{f'_c}$ greater than 100 psi (25/3 MPa) are permitted in computing V_c (nominal shear strength provided by concrete), V_{ci} (nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment), and V_{cw} (nominal shear strength provided by concrete when diagonal cracking results from excessive principal stress in web) for reinforced or prestressed concrete beams and concrete joist construction having minimum web reinforcement as required by Section 11.5.5.3, 11.5.5.4 or 11.6.5.2.

The 1989 edition of ACI 318 was the first edition to impose a limit of 100 psi (25/3MPa) on $\sqrt{f'_c}$ but waived this limit if transverse reinforcement sufficient to carry a nominal shear stress of 50 psi (0.35 MPa), multiplied by the factor $f'_c/5000 \leq 3$, were provided to prevent sudden shear failures at the onset of inclined cracking. The use of the factor $f'_c/5000 \leq 3$ resulted in a step-wise increase in the amount of transverse reinforcement with compressive strength, requiring that the product of the transverse reinforcement ratio and the yield strength of the transverse reinforcement ($A_v f_y / b_w s$) be at least 50 psi (0.35 Mpa) for concrete compressive strengths below 10,000 psi (69 Mpa) and double that amount ($A_v f_y / b_w s = 100$ psi or 0.69 MPa) for concrete compressive strengths slightly higher than 10,000 psi (69 Mpa). The amount of transverse reinforcement increased linearly with compressive strength up to a maximum $A_v f_y / b_w s$ of 150 psi (1.04 Mpa) for a concrete compressive strength of 15,000 psi (104 MPa). Experimental results by Roller and Russell¹⁸ showed that the amount of transverse reinforcement that resulted in a nominal shear stress of 150 psi (1.04 MPa) was barely enough to assure a safe estimate of strength using the ACI shear strength equation. Based on experimental results by several authors, a new form of Eq. (11-13) was introduced in the 2002 edition of the ACI 318 (Fig. 5) to estimate the minimum amount of transverse reinforcement in beams, with the goals of increasing the safety of the estimates and eliminating the sudden increase that occurred at a concrete compressive strength of 10,000 psi (69 Mpa). The minimum amount of transverse reinforcement is given by:

$$A_v = 0.75\sqrt{f'_c} \frac{b_w s}{f_y} \quad (6)$$

but shall not be less than $50b_w s/f_y$ ($0.33b_w s/f_y$), where A_v = area of shear reinforcement within a distance s , b_w = web width, f_y = specified yield strength of shear reinforcement, and s = spacing of shear reinforcement measured in a direction parallel to longitudinal reinforcement.

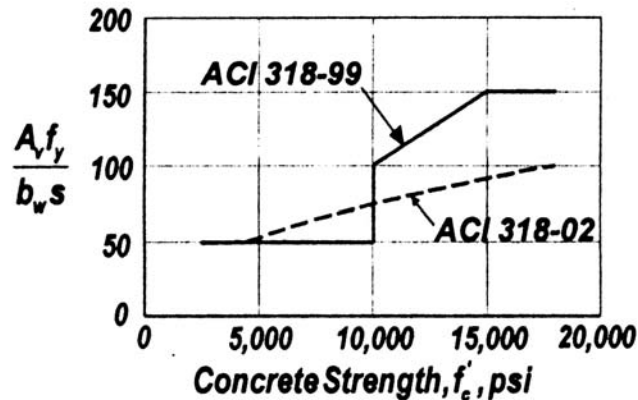


Figure 5 Minimum shear reinforcement

DEVELOPMENT LENGTH

ACI 318-95 Chapter 12 on Development and Splices of Reinforcement also restricts the value of $\sqrt{f'_c}$ to no more than 100 psi (25/3 MPa), meaning that the required development length of reinforcement embedded in concrete does not decrease any further, once the specified compressive strength of the concrete goes above 10,000 psi (69 MPa). This limit was also imposed in view of limited test results on the development of reinforcement embedded in concretes with very high compressive strengths. Unlike in Chapter 11, there is no exception to this important new restriction in Chapter 12.

The results of recent research^{19,20} have shown that when $\sqrt{f'_c}$ exceeds 100 psi (25/3 MPa), stirrups with a maximum spacing not to exceed a certain value need to be provided over the tension development or lap splice length, as applicable, to assure an adequate level of inelastic deformability before member failure. As a minimum, #3 (10 mm dia.) Grade 60 (414 MPa yield strength) reinforcing bars must be used for these stirrups. Test results indicate that the use of smaller bar sizes may result in fracturing stirrups before achieving an adequate level of inelastic deformability in the member. Research also shows that when $\sqrt{f'_c}$ exceeds 100 psi (25/3 MPa), stirrups with the maximum spacing mentioned above must be provided even when $\sqrt{f'_c}$ used in tension development or lap splice length calculation is limited to 100 psi (0.7 MPa). On the other hand, there is no reason to restrict $\sqrt{f'_c}$ to 100 psi (25/3 MPa) in computing tension splice length or development length when stirrups as required are provided along such length.

The Commentary to ACI 318-02 has added the following paragraph in Section R12.2.4:

“Although there is no requirement for transverse reinforcement along the tension development or splice length, recent research^{12.10, 12.11} indicates that in concrete with very high compressive strength, brittle anchorage failure occurred in bars with inadequate transverse reinforcement. In splice tests of No. 8 and No. 11 bars in concrete with an f'_c of approximately 1500 psi, transverse reinforcement improved ductile anchorage behavior.”

References 12.10 and 12.11 of ACI 318 are Refs. 19 and 20, respectively. 15,000 psi is 104 MPa.



Innovation Task Group 4 (ITG-4) of ACI has decided to go much further than ACI 318-02, as indicated below.

Research in bond and development of reinforcement by McCabe²¹ has indicated that design expressions based on the square root of the compressive strength of the concrete may be unconservative for compressive strengths greater than 10,000 psi (69 Mpa). Research by Azizinamini et al.²⁰ and Zuo and Darwin²² showed that the two main alternatives to correcting this problem were to increase the development length or to add transverse reinforcement. The main advantage of the latter approach is that it improves the behavior of the spliced or developed bars, because failure is significantly more ductile. This is particularly advantageous in seismic design.

Zuo and Darwin²² proposed a relationship between bond force and compressive strength to the $\frac{1}{4}$ power, based on a statistical study of monotonic tests of beams without transverse reinforcement and with concrete compressive strengths up to 16,000 psi. Their study concluded that the best fit between bond force and compressive strength for members with transverse reinforcement was obtained for compressive strength raised to the power of $\frac{3}{4}$, compared with compressive strength raised to the power of $\frac{1}{4}$ for members without transverse reinforcement.

An alternative design procedure was proposed by Azizinamini et al. (1999a). Rather than introducing new design equations, the procedure relies on a minimum amount of transverse reinforcement over the splice region. ITG-4 has chosen to adopt this approach for high-seismic applications. The situation already is that Section 21.3.2.3 of ACI 318-02 permits lap splices of flexural reinforcement only if hoop or spiral reinforcement is provided over the lap length. ITG-4 requires that when the value of $\sqrt{f'_c}$ exceeds 100 psi (0.69 Mpa), transverse reinforcement with a minimum cross-sectional area A_{sp} as given by Eq. (6) crossing the potential plane of splitting must be provided over the tension development length or the tension lap length:

$$A_{sp} = 0.5nA_b(f'_c / 15,000) \quad (6)$$

Additional requirements are that the maximum spacing of stirrups in the longitudinal direction not exceed 12 in. (305 mm), a minimum of three stirrups be used through the length of the splice, and that the bar size for the stirrups be at least No. 3 (10 mm dia.). It is additionally required that the development length be calculated using the equations in Sections 12.2.2 or 12.2.3 of ACI 318-02 assuming a value of $K_{tr} = 0$.

Conclusions from Zuo and Darwin²² for splices are consistent with those by Fujii et al.²³ for hooked bars. Fujii et al. summarized research on hooked bars in exterior joints carried out in Japan as part of their research program on high-strength materials. Concrete compressive strength of the specimens tested as part of the study ranged from 5,800 psi to 17,400 psi (40 to 120 MPa). All specimens in the testing program failed due to splitting of the side cover (cover to the side of the bar). Fujii et al. concluded that bond force was proportional to the cubic root of the compressive strength, rather than the square root of f'_c .

In view of the above, it is a concern that the current equation for the development length of hooked bars in tension of the ACI Code may result in unconservative estimates for compressive strengths above 10,000 psi (69 MPa). Given that no literature was found evaluating the use of the current ACI provisions for the development length of hooked bars in members with high-strength concrete, a modification to Eq. (21-6) is proposed in this report to reduce the likelihood of unconservative estimates in high-seismic applications. The proposed modification results in the same development lengths as given by ACI 318-02 for concrete compressive strengths up to 10,000 psi or 69 MPa (Fig. 6). For strengths greater than 10,000 psi (69 Mpa), the development length of a hooked bar, l_{dh} , increases in proportion to the fourth root of the compressive strength, resulting in an increase in development length that varies from 0 at 10,000 psi (69 Mpa) to approximately 20% at 20,000 psi (138 MPa).

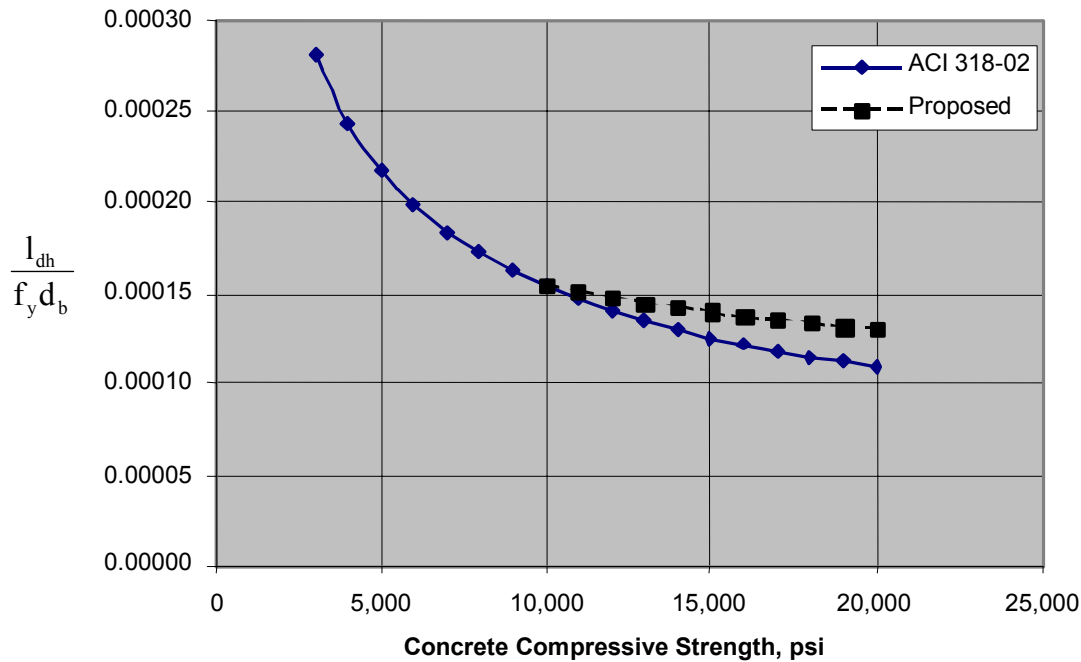
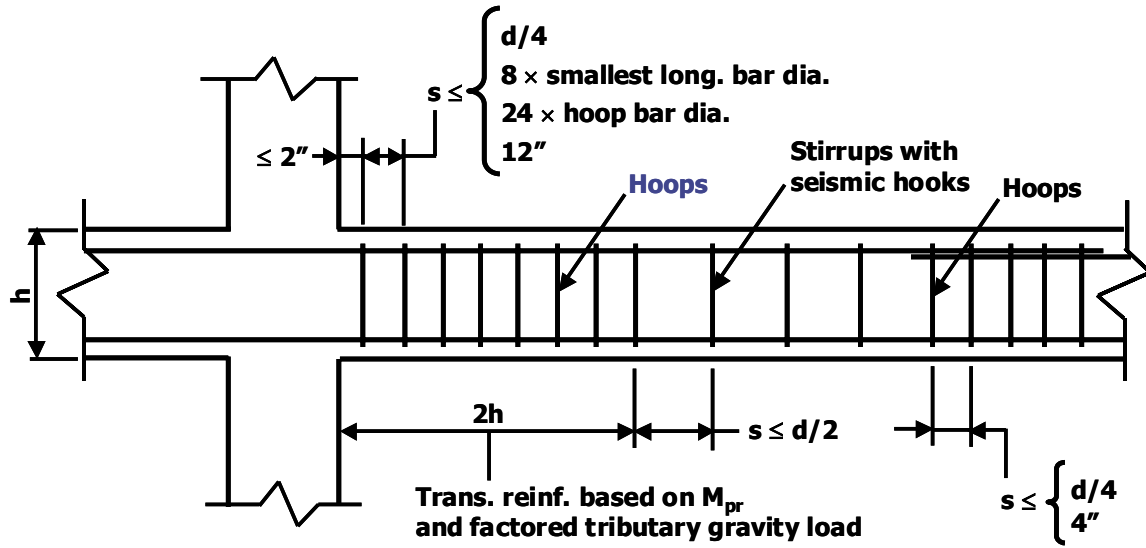


Figure 6 Proposed modification for development length of hooks

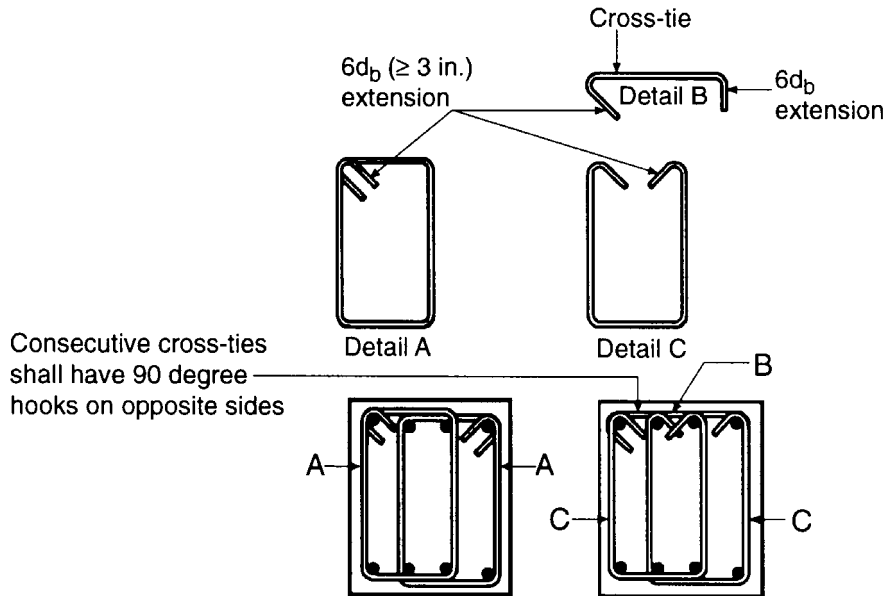
SEISMIC CONFINEMENT OF BEAMS

In the shear design of a beam (factored axial compressive force on member $\leq A_g f'_c / 10$, where A_g = gross area of section), the required shear strength is computed assuming that the beam has developed probable flexural strengths at the two ends (in opposite directions) and that it is loaded with factored (ACI) or unfactored (UBC) gravity loads along the span. Also, the quantity of shear reinforcement is established assuming the concrete contribution to shear strength is zero, if the earthquake-induced shear (that caused by moments equal to probable flexural strengths acting at the two ends) represents one-half or more of the required shear strength. In addition, the configuration and spacing of transverse reinforcement in regions of potential plastic hinging are established to confine the concrete core and to restrain the longitudinal bars in compression from buckling (Fig. 7).

There are no indications, based on available research,²⁴ that the above requirements are deficient in any way for concrete strengths ranging up to 15,600 psi (108 Mpa). Although ITG-4 had some concern over that the upper limit on hoop spacing established by ACI 318 is 12 in. (305 mm), a closer review showed that spacing limits in terms of the diameter of the longitudinal and the transverse reinforcement should be adequate to prevent buckling of the longitudinal reinforcement, while the amount of confinement afforded by the current spacing limits should be sufficient to achieve a limiting drift ratio of at least 2%.



(a) Spacing of transverse reinforcement



(b) One and two-piece hoops

Figure 7 Limitations on transverse reinforcement in beams

SEISMIC CONFINEMENT OF COLUMNS

Column (factored axial compressive force on member $> A_g f'_c / 10$) flexural strength is determined such that the sum of the nominal flexural strengths of the columns framing into a joint (calculated for the factored axial forces, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strengths) exceeds the sum of the nominal flexural strengths of the girders framing into the same joint by a factor of at least 1.2. This requirement is intended to result in frames where the flexural yielding of columns is restricted. Shear design is based on required shear strengths that correspond to the development of a moment at each column end equal to either (a) the maximum probable flexural strength of the section associated with the range of factored axial loads on the column, or (b) the column end moment corresponding to the development of probable flexural strengths at the ends of beams framing in. The required shear strength may never be less than the factored shear force determined from analysis of the structure. The configuration and spacing of the transverse reinforcement within the region of potential plastic hinging at the two ends are established to confine the concrete core and to restrain the longitudinal compression bars from buckling (Figs. 8, 9). Portions of the column outside of the regions of potential plastic hinging are also governed by specific transverse reinforcement requirements.

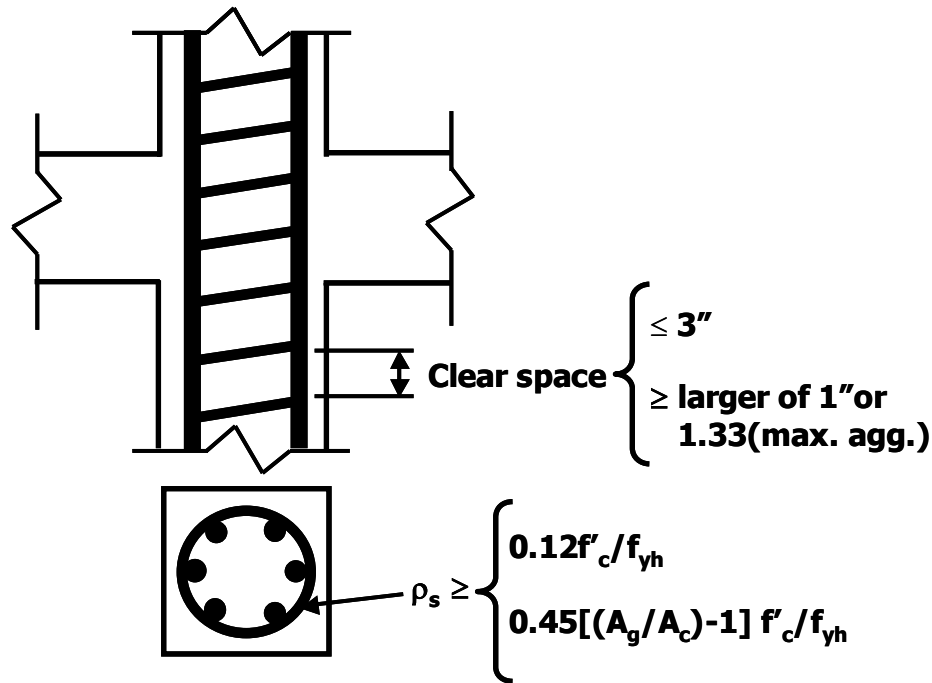
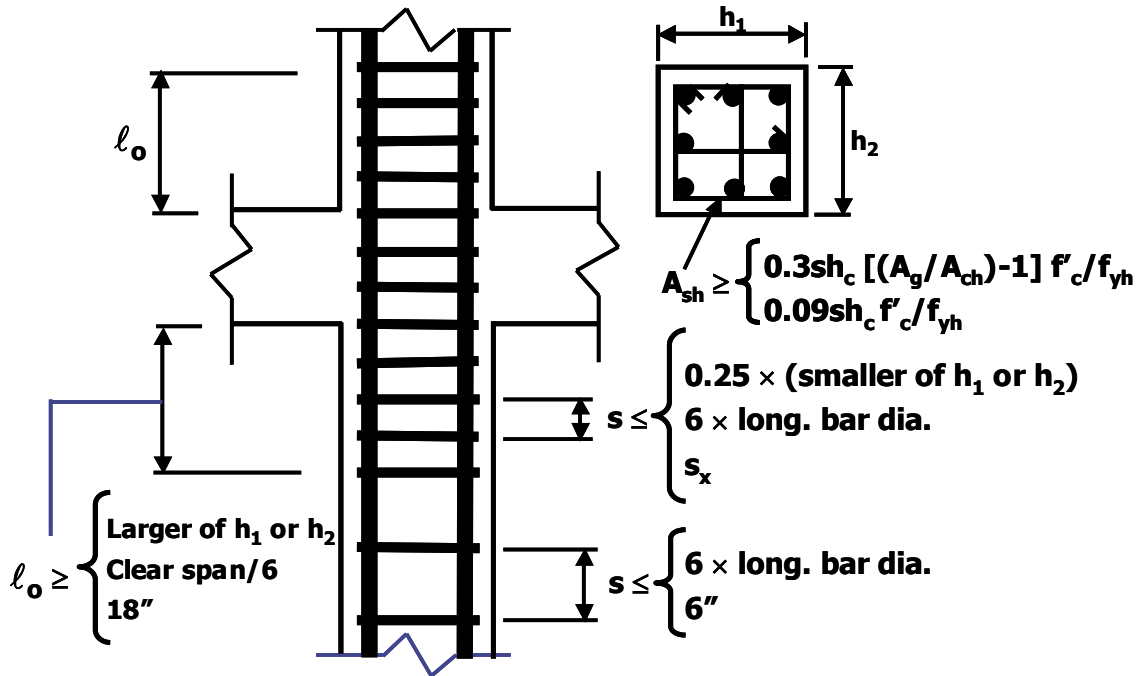
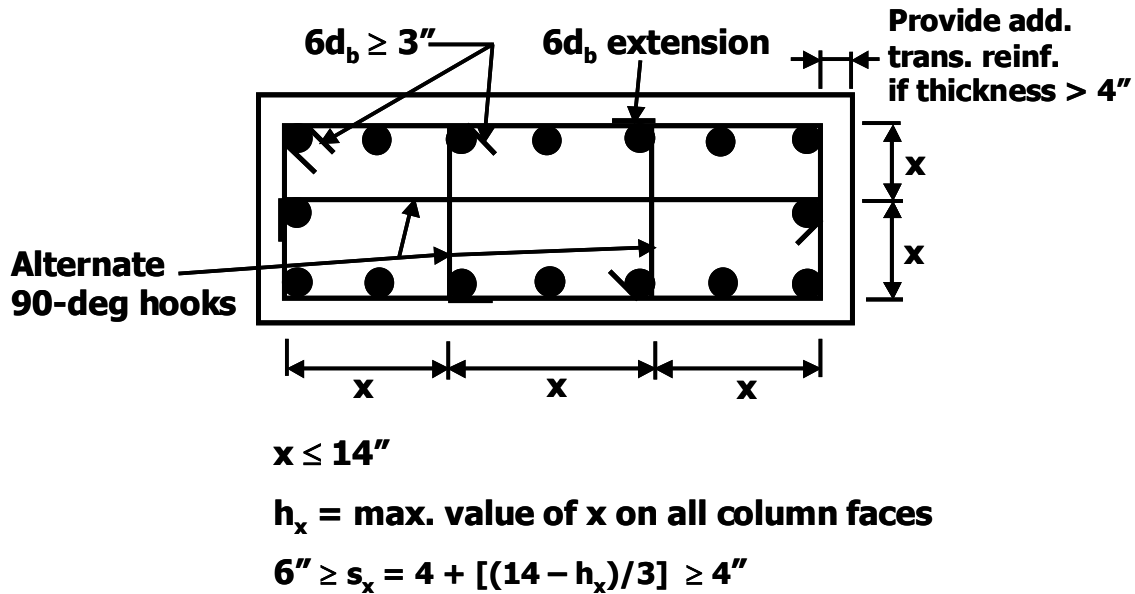


Figure 8 Seismic confinement requirements for columns: spiral confinement reinforcement



(a) Spacing and cross-sectional area requirements



(b) Transverse reinforcement details

Figure 9 Seismic confinement requirements for columns: ties for confinement reinforcement

The main disadvantage of the current ACI 318 provisions is that the safety afforded is not uniform for all columns, and the amount of transverse reinforcement required in members with lower levels of axial load is overly conservative. Although excessive conservatism does not pose a safety concern, it creates significant congestion problems, which hinder the use of high-strength concrete. Experimental research shows that a viable alternative to reducing congestion in plastic hinge regions is the use of high-yield-strength transverse

reinforcement. There is consensus among researchers that there should be an upper limit to the nominal yield strength of the transverse reinforcement used for confinement purposes, of approximately 120 ksi.

The experimental data reviewed by ITG-4 did not substantiate the need to reduce the maximum hoop spacing in columns. The following recommendations for changes to ACI 318 requirements have been agreed to by ITG-4 concerning the confinement of high-strength concrete columns located in seismically active regions:

The use of transverse reinforcement with a specified yield strength of 120,000 psi should be allowed to meet the confinement requirements for high-strength concrete columns. The yield strength of the reinforcement can be measured by the offset method of ASTM A 370²⁵ using 0.2 percent permanent offset.

The area ratio of transverse reinforcement shall not be less than that required by the following equation:

$$\rho_c = 0.35 \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_{ch}} - 1 \right] \frac{1}{\sqrt{k_c}} \frac{P_u}{A_g f'_c} \quad (7)$$

where

$$\frac{A_g}{A_{ch}} - 1 \geq 0.3 \quad (8)$$

and

$$\frac{P_u}{A_g f'_c} \geq 0.2 \quad (9)$$

A_{ch} = cross-sectional area of structural member measured out-to-out of transverse reinforcement, k_c = transverse reinforcement configuration parameter defined below, P_u = factored axial load at given eccentricity, $\rho_c = A_{sh}/h_c s$, and h_c = core dimension perpendicular to transverse reinforcement measured to the center of the hoops. Other symbols have been defined previously.

Transverse reinforcement shall have either circular or rectangular geometry. Reinforcement with circular geometry shall be in the form of spirals or hoops, and shall have parameter $k_c=1.0$. Reinforcement with rectangular geometry shall be provided in the form of single or overlapping hoops. Crossties of the same bar size and spacing as the hoops shall be permitted. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement. The parameter k_c for rectangular reinforcement shall be determined by the following equation:

$$k_c = \frac{0.15 h_c}{\sqrt{s} h_{x1}} \leq 1.0 \quad (10)$$

where h_{x1} = maximum horizontal spacing of hoop or crosstie legs perpendicular to h_c .

If the thickness of the concrete outside the confining transverse reinforcement exceeds 4 in. (102 mm), additional transverse reinforcement shall be provided at a spacing not exceeding 12 in. (305 mm). Concrete cover on the additional reinforcement shall not exceed 4 in. (102 mm).

CONCLUDING REMARKS

The legal codes of virtually all jurisdictions within the United States are based on one of five model codes: the *Uniform Building Code* (UBC), the *BOCA National Building Code* (BOCA/NBC), the *Standard Building Code* (SBC), the *International Building Code* (IBC), or the *NFPA 5000 Building Construction and Safety Code* (NFPA 5000). The concrete design and construction provisions of all five model codes



are based on ACI 318 *Building Code Requirements for Structural Concrete*. There is no upper limit on the strength of concrete that can be used in structural applications in ACI 318 or in any of the five model codes. This paper details and discusses the latest ACI 318 provisions that relate specifically to high-strength concrete. It points out a number of areas in which changes in ACI 318 provisions are needed to ensure continued satisfactory usage of high-strength concrete in seismic as well as non-seismic applications.

It should be noted that the report to be eventually issued by ITG-4 of ACI exists in draft form only. Thus, the ITG-4 recommendation discussed above should not be taken as final. In any case, ACI 318 almost invariably modifies suggested changes in the process of adopting them, if they adopt them at all.

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