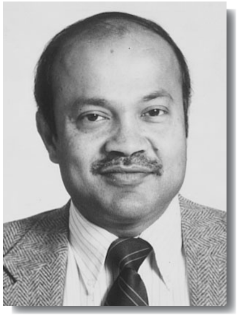
Limit on Strength of Normal Weight Concrete

This article points out and emphasizes the fact that no United States code or national standard imposes an upper limit on the strength of normal weight concrete that can be used in construction, including structures built in regions of high seismicity. The 2002 Edition of the ACI Building Code Requirements for Structural Concrete (ACI 318-02) has chosen to explicitly state this fact.



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The purpose of this article is to provide background information on the desirability of not placing an upper limit on the strength of normal weight concrete. Prior to delving into the main thrust of this discussion, however, it is necessary to define the meaning of high strength and high performance concrete.

High strength concrete (HSC) in the United States is generally considered to be concrete with a compressive strength of 6000 psi (41 MPa) or greater. In a 1984 ACI Committee 363 report, revised and reissued in 1992,[1] 6000 psi (41 MPa) was selected as a lower strength limit for high strength concrete. According to the report, the selection was not intended to imply that there is a drastic change in material properties or in production techniques that occur at that particular compressive strength. In reality, all changes that take place above 6000 psi (41 MPa) represent a process that starts with the lower strength concretes and continues with high strength concretes.

The ACI TAC (Technical Activities Committee) Subcommittee on High Performance Concrete (HPC) has offered the following definition of HPC:

"Concrete which meets special performance and uniformity requirements that cannot always be achieved routinely by using only conventional materials and normal mixing, placing, and curing practices. The requirements may involve enhancements of characteristics such as placement and compaction without segregation, long-term mechanical properties, early-age strength, toughness, volume stability, or service life in severe environments."

Since compressive strength can be easily quantified and higher concrete strength is often (but not always) associated with other desirable performance characteristics of concrete, Zia[2] found it convenient to define HPC in terms of compressive strength. In Reference 2, Zia indicated a minimum strength level of 6000 psi (41 MPa) for concrete to qualify as HPC.

ACI 318[3] does not define either HSC or HPC. Furthermore, these terms are not defined in any other U.S. code or national standard.

UPPER LIMIT ON CONCRETE STRENGTH

ACI 318-99 requires that the specified compressive strength of lightweight aggregate concrete used in the design of members of special moment frames in any seismic zone[4] or seismic performance[5,6] or design[7] category, and of members of seismic-force-resisting systems in Seismic Zones 3, 4,[4] Seismic Performance Categories D and E,[5,6] or Seismic Design Categories D, E, and F,[7] must not exceed 4000 psi (28 MPa). This limit exists primarily because of the paucity of experimental and field data on the behavior of members made with lightweight aggregate concrete subjected to displacement reversals in the nonlinear range. The limit has been revised to 5000 psi (35 MPa) in ACI 318-02.[3]

Lightweight aggregate concrete with a higher design compressive strength may be used if demonstrated by experimental evidence that structural members made with that lightweight aggregate concrete have the strength and toughness equal to or exceeding those of comparable members made with normal weight aggregate concrete of the same strength.

ACI 318 has never imposed an upper limit on the strength of normal weight concrete that can be used in construction, even for structures located in high seismic zones or assigned to high seismic performance or design categories. No such limit has ever existed in any other U.S. standard, in any of the model building codes,[4-7] or in the building code of any legal jurisdiction within the United States.

It appears now that within the jurisdiction of the City of Los Angeles, California, and in neighboring jurisdictions such as Long Beach, a limit of 6000 psi (42 MPa) has at least from time to time been informally imposed on the strength of normal weight concrete used in special moment frames. Special approval has apparently been required for concrete of higher strength. Research has so far been unsuccessful in locating any written regulation. In fact, a piece of written communication from the City of Los Angeles, dated August 6, 2001, states:

"Currently, the 1999 Edition of the Los Angeles Building Code does not have any 'special' prescriptive provisions or local amendments to Chapter 19 regarding high strength concrete. The City of Los Angeles, Department of Building and Safety enforces the 1999 edition of the Los Angeles City Building Code and the ACI 318-95 as adopted by the 1997 Uniform Building Code."

More recently, however, the City of Los Angeles Department of Building and Safety issued an intra-departmental correspondence, dated October 2, 2001, which states in part:

"The use of high strength concrete in structural elements designed to resist seismic loading shall not exceed 8000 psi specified concrete compressive strength without special approval."

This is a disturbing local modification to ACI 318 requirements that runs contrary to strongly held opinion within ACI Committee 318. In fact, ACI 318 feels so strongly about the issue that for the 2002 Edition of the standard, it has been decided that an explicit statement should be added, pointing out the absence of a strength limit. Accordingly, the following sentence has been added to Section 1.1.1:

"No maximum specified compressive strength shall apply unless restricted by a specific code provision."

ACI feels so strongly about not placing a strength limit for normal weight concrete that it has been making adjustments in the standard on an ongoing basis to account for sometimes differing properties of high strength concrete. The most important of these adjustments is described in the remainder of this paper.

CONCRETE MIX DESIGN

A significant change affecting high strength concrete mix design has been processed by ACI Committee 318 for inclusion in ACI 318-02.

It has been concluded that the current ACI 318-99 requirements concerning target overstrengths for mix design purposes are inappropriate for concrete compressive strengths in excess of 5000 psi (34 MPa). The required average compressive strength f'_{cr} used as the basis for selection of concrete proportions in ACI 318-99 is too high for concrete proportioned using field data which allows the calculation of the standard deviation.

The required average compressive strength is too low when field data are not available to establish the standard deviation. In addition, acceptance criteria for the strength of concrete are biased against high strength concrete due to the current limitation on an individual strength test below f'_c.

Required Average Strength When Test Records Are Available for Determination of Standard Deviation

Section 5.3.2.1 of ACI 318-99 has been changed as shown below, to include Table 5.3.2.1 on this page. Strike-through lines indicate deletion of existing text; underlining indicates addition of new text:

"**5.3.2.1** – Required average compressive strength f'_{cr} used as basis for selection of concrete proportions shall be ~~the larger of Eq. (5.1) or (5.2)~~ <u>determined from Table 5.3.2.1</u> using ~~a~~ <u>the</u> standard deviation calculated in accordance with 5.3.1.1 or 5.3.1.2.

$$\cancel{f'_{cr} = f'_c + 1.34s \qquad (5\text{-}1)}$$

~~Or~~

$$\cancel{f'_{cr} = f'_c + 2.33s - 500 \qquad (5\text{-}2)}\text{"}$$

Eq. (5-1) provides a probability of 1-in-100 that the average of three consecutive tests will be below the specified compressive strength f'_c. Eq. (5-2) provides a similar probability that an individual test will be more than 500 psi below the specified compressive strength f'_c. Eq. (5-3) provides the same 1-in-100 probability that an individual test strength will be more than 10 percent below f'_c.

Table 5.3.2.1. Required average compressive strength when data are available to establish a standard deviation.

Specified compressive strength, f'_c, psi	Required average compressive strength, f'_c, psi
5000 or less	Use the larger of Eqs. (5-1) and (5-2) $f'_{cr} = f'_c + 1.34s$ (5-1) $f'_{cr} = f'_c + 2.33s - 500$ (5-2)
Over 5000	Use the larger of Eqs. (5-1) and (5-3) $f'_{cr} = 0.90f'_c + 2.33s$ (5-3)

Required Average Strength When Test Records Are Not Available for Determination of Standard Deviation

ACI 318-99 requires that when a concrete production facility does not have field strength test records for calculation of standard deviation, the required average strength f'_{cr} is to be determined from Table 5.3.2.2.

In ACI 318-02, Table 5.3.2.2 will be modified as shown on this page.

The following section has been added to Commentary Section R5.3.3:

"For strengths over 5000 psi where the average strength documentation is based on laboratory trial mixtures, it may be appropriate to increase f'_{cr} calculated in Table 5.3.2.2 to allow for a reduction in strength from the laboratory to the field."

In addition to the above, Sections 5.4.1 and 5.6.3.3 have been modified as follows:

"**5.4.1 –** …(third sentence) This alternative shall not be used for specified compressive strength greater than ~~4000~~ 5000 psi.

5.6.3.3 – Strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met:

(a) Every arithmetic average of any three consecutive strength tests equals or exceeds f'_c.

(b) No individual strength test (average of two cylinders) falls below f'_c by more than 500 psi when f'_c is 5000 psi or less; or by more than 0.10 f'_c when f'_c is more than 5000 psi."

According to the above, the concrete strength is considered to be satisfactory as long as the averages of any three consecutive strength tests remain above the specified f'_c and no individual strength test falls below the specified f'_c by more than 500 psi when f'_c is 5000 psi (34 MPa) or less or falls below f'_c by more than 10 percent when f'_c is over 5000 psi (34 MPa).

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.

ACI 318-99 requires that at every section of a flexural member where tensile reinforcement is required by analysis, except as provided in Sections 10.5.2, 10.5.3 and 10.5.4, the area A_s provided shall not be less than that given by:

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

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

[ACI 318-99 and -02 Eq. (10-3)]

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

The $200/f_y$ ($1.4/f_y$, SI) 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 5000 psi (34 MPa) is used, the $200/f_y$ ($1.4/f_y$, SI) value may not be sufficient. The minimum amount of flexural reinforcement was, therefore, changed in ACI 318-95 to the current requirements.

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 smaller value given either by:

$$A_{s,min} = \frac{6\sqrt{f'_c}}{f_y} b_w d$$

$$A_{s,min} = \frac{\sqrt{f'_c}}{2 f_y} b_w d \quad \text{(SI units)}$$

[ACI 318 Eq. (10-4)]

or by Eq. (10-3) with b_w set equal to the width of the flange.

In ACI 318-02, Section 10.5.2 has been rewritten (largely editorially) to read as follows:

"**10.5.2 –** For statically determinate members with a flange in tension, the area $A_{s,min}$ shall be equal to or greater than the value given by Eq. (10-3) 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 Sections 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 must be the minimum shrinkage and temperature reinforcement required by Section 7.12. Maximum spacing of this reinforcement must not exceed three times the thickness or 18 in. (457 mm).

SHEAR STRENGTH

ACI 318-99 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 or torsional 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 new restriction, however.

Values of $\sqrt{f'_c}$ greater than 100 psi

Table 5.3.2.2. Required average compressive strength when data are not available to establish a standard deviation.

Specified compressive strength, f'_c, psi	Required average compressive strength, f'_{cr}, psi
Less than 3000	$f'_c + 1000$
3000 to 5000	$f'_c + 1200$
Over 5000	~~$f'_c + 1400$~~ $1.10f'_c + 700$

(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 a minimum web reinforcement equal to $f_c'/5000$ ($f_c'/35$, SI) times, but not more than three times, the amounts required by Sections 11.5.5.3, 11.5.5.4 or 11.6.5.2.

Section 11.5.5.3 requires a minimum amount of shear reinforcement in reinforced concrete beams, capable of carrying at least 50 psi (0.35 MPa) of nominal shear stress. The amount of shear reinforcement has to be twice as large as soon as the specified compressive strength exceeds 10,000 psi (69 MPa), if the designer is to benefit from the shear strength contributed by the excess compressive strength of concrete. The multiplier of two goes up linearly to a value of three, corresponding to a specified compressive strength of 15,000 psi (103 MPa), and remains level for higher strengths.

A limited number of tests of reinforced concrete beams made with high strength concrete[8,9] ($f_c' > 8000$ psi or 55 MPa) suggested that the inclined cracking load increases less rapidly than Eq. (11-3) or (11-6) of ACI 318 would indicate. This was offset by an increased effectiveness of the stirrups compared to the strength predicted by Eqs. (11-17), (11-18) and (11-19). Other tests of high strength concrete girders with minimum web reinforcement indicated that this amount of web reinforcement may be inadequate to prevent brittle shear failures when inclined cracking occurs.[10]

In ACI 318-02, Section 11.1.2.1 has been modified as follows (strikethrough lines indicate deletion of existing text; underlining indicates addition of new text):

"**11.1.2.1 –** Values of $\sqrt{f_c'}$ greater than 100 psi shall be permitted in computing V_c, V_{ci}, and V_{cw} for reinforced or prestressed concrete beams and concrete joist construction having minimum web reinforcement ~~equal to $f_c'/5000$ times, but not more than three times, the amount required by~~ <u>in accordance with</u> 11.5.5.3, 11.5.5.4, or 11.6.5.2."

Section 11.5.5.3 in its turn has been modified as follows:

"…the minimum area of shear reinforcement for prestressed (except as provided in 11.5.5.4) and nonprestressed members shall be computed by

$$A_v = \cancel{50}\ \underline{75\sqrt{f_c'}}\ \frac{b_w s}{f_y} \qquad (11\text{-}13)$$

<u>but shall not be less than $(50b_w s)/f_y$</u>, where b_w and s are in inches."

The revised Eq. (11-13) provides for a gradual increase in the minimum area of transverse reinforcement, replacing the sudden increase in the minimum amount of transverse reinforcement at a compressive strength of 10,000 psi, which resulted from the 1989 provision.

There are no test data on the two-way shear strength of high strength concrete slabs or torsional strength. Until more practical experience is obtained with beams and slabs built with concretes with strengths greater than 10,000 psi (69 MPa), it is required by code to limit $\sqrt{f_c'}$ to 100 psi (25/3 MPa) in calculations of shear strength and torsional strength.

DEVELOPMENT LENGTH

ACI 318-99 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 (60 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 restriction in Chapter 12.

The results of recent research[11,12] 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 ensure an adequate level of inelastic deformability before member failure. As a minimum, No. 3 (10 mm diameter) 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 results also show 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 (25/3 MPa). On the other hand, there is no reason to restrict $\sqrt{f_c'}$ to 100 psi (25/3 MPa) in computing the tension splice length or development length when stirrups as required are provided along such length.

Subcommittee B of ACI 318 has been working with Committee 408 on code provisions. Consensus was not achieved on code language. As an interim step for the 2002 Code, Subcommittee B decided to include text in the Commentary to encourage designers to provide transverse reinforcement over developing and spliced bars in high strength concrete to promote ductile behavior, as recommended in References 11 and 12.

COLUMNS FORMING PART OF THE LATERAL-FORCE-RESISTING SYSTEM OF A STRUCTURE IN A REGION OF HIGH SEISMICITY

Column (factored axial compressive force on member greater than $A_g f_c'/10$) flexural strength is determined such that the sum of the design 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 design flexural strengths of the girders framing into the same joint by a factor of at least 1.2.

Table 1. Spacing of transverse reinforcements in reinforced concrete columns.

			f_{yh} (ksi) =	**60**				f'_c (ksi) =		**6**					
Transverse reinforcement	No. of legs	A_{sh} (sq in.)	h (in.) =	**20**		**24**		**28**		**32**		**36**		**40**	
#4			h_c (in.) =	16.5		20.5		24.5		28.5		32.5		36.5	
	2	0.4	s (in.) =	**2.69**	*4.04*	**2.17**	*3.25*	**1.81**	*2.72*	**1.56**	*2.34*	**1.37**	*2.05*	**1.22**	*1.83*
	3	0.6	s (in.) =	**4.04**	*6.06*	**3.25**	*4.88*	**2.72**	*4.08*	**2.34**	*3.51*	**2.05**	*3.08*	**1.83**	*2.74*
	4	0.8	s (in.) =	**5.39**	*8.08*	**4.34**	*6.50*	**3.63**	*5.44*	**3.12**	*4.68*	**2.74**	*4.10*	**2.44**	*3.65*
#5			h_c (in.) =	16.375		20.375		24.375		28.375		32.375		36.375	
	2	0.62	s (in.) =	**4.21**	*6.31*	**3.38**	*5.07*	**2.83**	*4.24*	**2.43**	*3.64*	**2.13**	*3.19*	**1.89**	*2.84*
	3	0.93	s (in.) =	**6.31**	*9.47*	**5.07**	*7.61*	**4.24**	*6.36*	**3.64**	*5.46*	**3.19**	*4.79*	**2.84**	*4.26*
	4	1.24	s (in.) =	**8.41**	*12.62*	**6.76**	*10.14*	**5.65**	*8.48*	**4.86**	*7.28*	**4.26**	*6.38*	**3.79**	*5.68*
#6			h_c (in.) =	16.25		20.25		24.25		28.25		32.25		36.25	
	2	0.88	s (in.) =	**6.02**	*9.03*	**4.83**	*7.24*	**4.03**	*6.05*	**3.46**	*5.19*	**3.03**	*4.55*	**2.70**	*4.05*
	3	1.32	s (in.) =	**9.03**	*13.54*	**7.24**	*10.86*	**6.05**	*9.07*	**5.19**	*7.79*	**4.55**	*6.82*	**4.05**	*6.07*
	4	1.76	s (in.) =	**12.03**	*18.05*	**9.66**	*14.49*	**8.06**	*12.10*	**6.92**	*10.38*	**6.06**	*9.10*	**5.39**	*8.09*

			f_{yh} (ksi) =	**60**				f'_c (ksi) =		**9**					
Transverse reinforcement	No. of legs	A_{sh} (sq in.)	h (in.) =	**20**		**24**		**28**		**32**		**36**		**40**	
#4			h_c (in.) =	16.5		20.5		24.5		28.5		32.5		36.5	
	2	0.4	s (in.) =	**1.80**	*2.69*	**1.45**	*2.17*	**1.21**	*1.81*	**1.04**	*1.56*	**0.91**	*1.37*	**0.81**	*1.22*
	3	0.6	s (in.) =	**2.69**	*4.04*	**2.17**	*3.25*	**1.81**	*2.72*	**1.56**	*2.34*	**1.37**	*2.05*	**1.22**	*1.83*
	4	0.8	s (in.) =	**3.59**	*5.39*	**2.89**	*4.34*	**2.42**	*3.63*	**2.08**	*3.12*	**1.82**	*2.74*	**1.62**	*2.44*
#5			h_c (in.) =	16.375		20.375		24.375		28.375		32.375		36.375	
	2	0.62	s (in.) =	**2.80**	*4.21*	**2.25**	*3.38*	**1.88**	*2.83*	**1.62**	*2.43*	**1.42**	*2.13*	**1.26**	*1.89*
	3	0.93	s (in.) =	**4.21**	*6.31*	**3.38**	*5.07*	**2.83**	*4.24*	**2.43**	*3.64*	**2.13**	*3.19*	**1.89**	*2.84*
	4	1.24	s (in.) =	**5.61**	*8.41*	**4.51**	*6.76*	**3.77**	*5.65*	**3.24**	*4.86*	**2.84**	*4.26*	**2.53**	*3.79*
#6			h_c (in.) =	16.25		20.25		24.25		28.25		32.25		36.25	
	2	0.88	s (in.) =	**4.01**	*6.02*	**3.22**	*4.83*	**2.69**	*4.03*	**2.31**	*3.46*	**2.02**	*3.03*	**1.80**	*2.70*
	3	1.32	s (in.) =	**6.02**	*9.03*	**4.83**	*7.24*	**4.03**	*6.05*	**3.46**	*5.19*	**3.03**	*4.55*	**2.70**	*4.05*
	4	1.76	s (in.) =	**8.02**	*12.03*	**6.44**	*9.66*	**5.38**	*8.06*	**4.61**	*6.92*	**4.04**	*6.06*	**3.60**	*5.39*

			f_{yh} (ksi) =	**60**				f'_c (ksi) =		**12**					
Transverse reinforcement	No. of legs	A_{sh} (sq in.)	h (in.) =	**20**		**24**		**28**		**32**		**36**		**40**	
#4			h_c (in.) =	16.5		20.5		24.5		28.5		32.5		36.5	
	2	0.4	s (in.) =	**1.35**	*2.02*	**1.08**	*1.63*	**0.91**	*1.36*	**0.78**	*1.17*	**0.68**	*1.03*	**0.61**	*0.91*
	3	0.6	s (in.) =	**2.02**	*3.03*	**1.63**	*2.44*	**1.36**	*2.04*	**1.17**	*1.75*	**1.03**	*1.54*	**0.91**	*1.37*
	4	0.8	s (in.) =	**2.69**	*4.04*	**2.17**	*3.25*	**1.81**	*2.72*	**1.56**	*2.34*	**1.37**	*2.05*	**1.22**	*1.83*
#5			h_c (in.) =	16.375		20.375		24.375		28.375		32.375		36.375	
	2	0.62	s (in.) =	**2.10**	*3.16*	**1.69**	*2.54*	**1.41**	*2.12*	**1.21**	*1.82*	**1.06**	*1.60*	**0.95**	*1.42*
	3	0.93	s (in.) =	**3.16**	*4.73*	**2.54**	*3.80*	**2.12**	*3.18*	**1.82**	*2.73*	**1.60**	*2.39*	**1.42**	*2.13*
	4	1.24	s (in.) =	**4.21**	*6.31*	**3.38**	*5.07*	**2.83**	*4.24*	**2.43**	*3.64*	**2.13**	*3.19*	**1.89**	*2.84*
#6			h_c (in.) =	16.25		20.25		24.25		28.25		32.25		36.25	
	2	0.88	s (in.) =	**3.01**	*4.51*	**2.41**	*3.62*	**2.02**	*3.02*	**1.73**	*2.60*	**1.52**	*2.27*	**1.35**	*2.02*
	3	1.32	s (in.) =	**4.51**	*6.77*	**3.62**	*5.43*	**3.02**	*4.54*	**2.60**	*3.89*	**2.27**	*3.41*	**2.02**	*3.03*
	4	1.76	s (in.) =	**6.02**	*9.03*	**4.83**	*7.24*	**4.03**	*6.05*	**3.46**	*5.19*	**3.03**	*4.55*	**2.70**	*4.05*

Note: 1 in. = 25.4 mm; 1 sq in. = 645 mm^2; 1 ksi = 6.895 MPa.
Bold faced numbers indicate maximum reinforcement spacing within length l_o at column ends.
Italicized numbers indicate maximum transverse reinforcement spacing outside of those regions.

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 that is 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 regions of potential plastic hinging at the two ends are established to confine the concrete core and to restrain the longitudinal compression bars from buckling. Portions of the column outside of the regions of potential plastic hinging are also governed by specific transverse reinforcement requirements.

Table 1 shows the spacing of transverse reinforcement in reinforced concrete columns, as required by Section 21.4.4 of ACI 318-99. Three specified compressive strengths of concrete equal to 6, 9 and 12 ksi (41, 62 and 83 MPa) and one specified yield strength of the transverse reinforcement equal to 60 ksi (410 MPa) is considered.

It should be clear from Table 1 that the extremely high confinement reinforcement requirement imposes a practical limit on the strength of normal weight concrete that can be used in a reinforced concrete column which forms part of the lateral-force-resisting system of a structure in a region of high seismicity. An arbitrary limit on such strength is, therefore, rendered unnecessary.

CONCLUDING REMARKS

ACI 318-02 explicitly states that there is no upper limit on the strength of normal weight concrete that can be used in structures, including those exposed to high seismic risk.

As pointed out in this paper, adjustments have been made on an ongoing basis in the ACI 318 code provisions to account for sometimes differing properties of high strength concrete. The consensus is that local imposition of arbitrary limits on the strength of normal weight concrete is unwarranted and unjustified.

Application of concrete having specified compressive strengths in the range of 8 to 10 ksi (56 to 70 MPa) is now considered routine and commonplace in cities like Seattle (UBC Seismic Zone 3) and San Francisco (UBC Seismic Zone 4).

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DISCUSSION NOTE

The Editors welcome discussion of reports, articles, and problems and solutions published in the PCI JOURNAL. The comments must be confined to the scope of the article being discussed. Please note that discussion of papers appearing in this issue must be received at PCI Headquarters by April 1, 2002.