Report on High-Strength Concrete Columns

Reported by joint ACI-ASCE Committee 441

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This report reviews the state of the knowledge of the behavior of high-strength concrete (HSC) columns. High-strength concrete, as used in this report, is defined as concrete with compressive strength exceeding 70 MPa (10,000 psi). The report provides highlights of research available on the performance of HSC columns under monotonically increasing concentric or eccentric compression, and with incrementally increasing lateral deformation reversals and constant axial compression.

Research results are used to discuss the effect of cover concrete and parameters related to transverse reinforcement on strength and ductility of HSC columns subjected to concentric load.

The behavior of HSC columns subjected to combined axial load and bending moment is discussed in terms of variables related to concrete and transverse reinforcement. In addition to discussion on flexural and axial capacity, this report also focuses on seismic performance of HSC columns.

Keywords: axial load; bending moment; columns; cover concrete; ductility; flexural strength; high-strength concrete; longitudinal reinforcement; seismic design; transverse reinforcement.

CONTENTS

CHAPTER 1—INTRODUCTION

One application of high-strength concrete (HSC) has been in the columns of buildings. In 1968 the lower columns of the Lake Point Tower building in Chicago, Illinois, were con-

ACI 441R-96 became effective November 25, 1996.

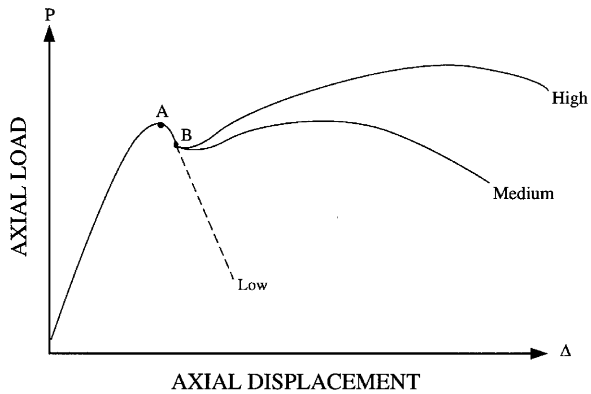

Fig. 1—Schematic behavior of HSC columns subjected to concentric axial loads, incorporating low, medium, and high amounts of transverse reinforcement

structed using 52 MPa concrete.[1] More recently, several high rise buildings[1-4] have utilized concrete with compressive strengths in excess of 100 MPa in construction of columns.

Many studies[4-9] have demonstrated the economy of using HSC in columns of high-rise buildings, as well as low to mid-rise buildings.[10] In addition to reducing column sizes and producing a more durable material, the use of HSC has been shown to be advantageous with regard to lateral stiffness and axial shortening.[11] Another advantage cited in the use of HSC columns is reduction in cost of forms. This is achieved by using HSC in the lower story columns and reducing concrete strength over the height of the building while keeping the same column size over the entire height.

The increasing use of HSC caused concern over the applicability of current building code requirements for design and detailing of HSC columns. As a result, a number of research studies have been conducted in several countries during the last few years. The purpose of this paper is to summarize major aspects of some of the reported data.

The major objectives of reported studies have been to investigate the validity of applying the current building code requirements to the case of HSC, to evaluate similarities or differences between HSC and normal-strength concrete (NSC) columns, and to identify important parameters affecting performance of HSC columns designed for seismic as well as non-seismic areas. These concerns arise from the fact that requirements for design and detailing of reinforced concrete columns in different model codes are primarily empirical and are developed based on experimental data obtained from testing column specimens having compressive strengths below 40 MPa.

The reported information can be divided into two general categories: performance of HSC columns under concentric axial load; and performance of HSC columns under combined axial load and bending moment. This report gives the highlights of the reported data in each of these categories. In this report, HSC is defined as concrete with compressive strength greater than 70 MPa.

CHAPTER 2—PERFORMANCE OF HSC COLUMNS UNDER CONCENTRIC LOADS

The majority of reported studies[12-27] in the field of HSC columns concern the behavior of columns subjected to concentric loads. Understanding the behavior of columns under concentric loads assists in quantifying the parameters affecting column performance. However, conclusions from this type of loading should not necessarily be extended to the case of combined loading, a situation most frequently encountered in columns used in buildings.

Reported data indicate that stress-strain characteristics of high-strength concrete, cover concrete, and parameters related to confining steel have the most influence on response of HSC columns subjected to concentric loads. The effect of the first parameter is discussed in Sec. 3.1. The remaining two parameters are discussed in the following sections.

2.1—Effect of cover concrete

Figure 1 shows a schematic load-axial deformation response under concentric loads of HSC columns with transverse reinforcement. As concrete strength increases, the ascending portion of the curve approaches a straight line. In general, spalling of the cover concrete is reported[12-27] to occur prior to achieving the axial load capacity of HSC columns, as calculated by the following equation:

$$P_o = 0.85 f'_c (A_g - A_{st}) + A_{st} f_y \qquad (1)$$

where:

P_O = Pure axial load capacity of columns calculated according to the nominal strength equations of ACI 318-89

f'_c =Concrete compressive strength

A_g =Gross cross-sectional area of column

A_{st} =Area of longitudinal steel

f_y =Yield strength of longitudinal steel

The 1994 edition of the Canadian Code for Design of Concrete Structures also uses this equation for computing P_o, except that the factor 0.85 is replaced by

$$\alpha_1 = (0.85 - 0.0015 f'_c) \geq 0.67$$

in which f'_c is in MPa. Hence, P_o calculated by the Canadian code will be somewhat less than that calculated by ACI 318-89.

Point A in Fig. 1 indicates the loading stage at which cover concrete spalls off. The behavior of HSC columns beyond this point depends on the relative areas of the column and the core and on the amount of transverse reinforcement provided. Following spalling of the cover concrete, the load-carrying capacity of columns generally drops to point B in Fig. 1. Beyond this point, Bjerkeli et al.,[19] Cusson et al.,[25] and Nishiyama et al.[28] report that it is possible to increase the maximum axial strength of columns up to 150 percent of that calculated by the ACI 318-89 provisions and obtain a ductile behavior by providing sufficient transverse reinforcement. The effect of the amount of transverse reinforcement is

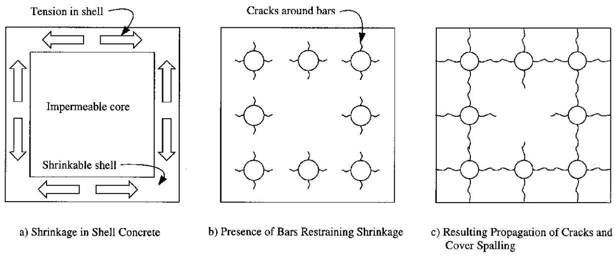

Fig. 2—Factors promoting cover spalling in high-strength concrete columns (adapted from Ref. 29)

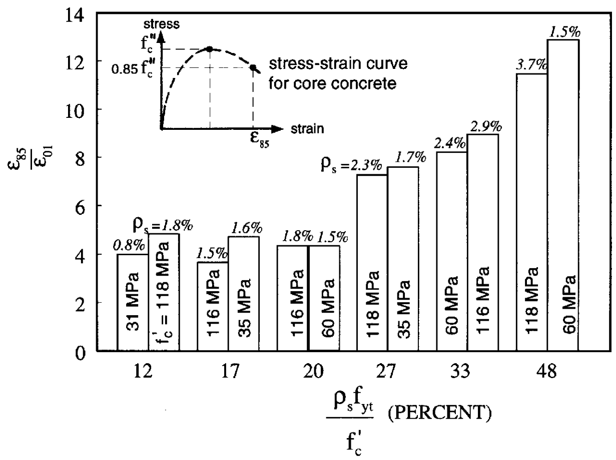

Fig. 3—Columns with different concrete strengths showing similar axial ductility ratios (f′$_c$ = concrete compressive strength based on standard cylinder test) (adapted from Ref. 30)

shown schematically in Fig. 1 and will be discussed further in later sections.

The loss of cover concrete in HSC columns before reaching the axial capacity calculated by ACI 318-89 is contrary to the observed behavior of concrete columns made of NSC. Collins et al.[29] provide the following explanation for the factors resulting in early spalling of cover concrete in HSC columns. According to those authors, the low permeability of HSC leads to drying shrinkage strain in cover concrete, while the core remains relatively moist. As a result, tensile stresses are developed in the cover concrete as shown in Fig. 2a. Moreover, longitudinal steel, as depicted in Fig. 2b, promotes additional cracking. The combination of these two mechanisms (see Fig. 2c) then results in the formation of a cracking pattern that, according to those authors, is responsible for early loss of cover concrete, thereby preventing HSC columns from reaching their axial load capacity predicted by Eq. (1) prior to spalling of cover concrete.

Early spalling of concrete cover may also be initiated by the presence of a closely spaced reinforcement cage that separates core and cover concrete. Cusson et al.[25] attributed the spalling of the cover to planes of weakness created by the dense steel cages. They state that spalling becomes more prevalent as the concrete strength increases.

Saatcioglu and Razvi[27,30] also observed early spalling of cover concrete in their tests. Those researchers indicated that the presence of closely spaced reinforcement cage between

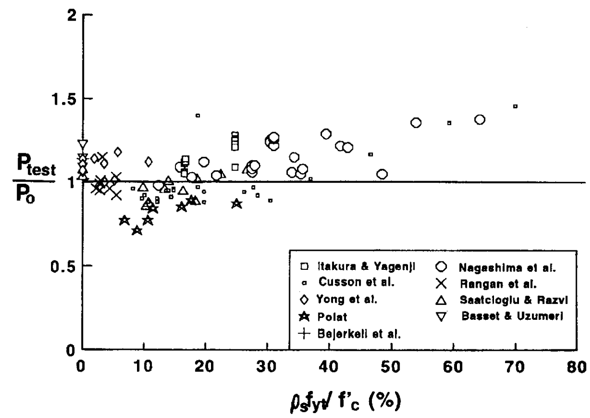

Fig. 4—Comparison of experimental and calculated concentric strengths of columns (adapted from Ref. 30)

the core and the cover concrete provided a natural plane of separation, which resulted in an instability failure of the cover concrete under high compressive stresses. The spalling in their tests occurred at a stress level below that corresponding to the crushing of plain concrete.

2.2—Effect of volumetric ratio of transverse reinforcement

In the case of NSC, an increase in the amount of transverse reinforcement has been shown to increase strength and ductility.[31] The same observation has been reported[19,25,27] for the case of HSC, though to a lesser degree. Some researchers have attributed this phenomenon to the relatively smaller increase in volume during microcracking of HSC, resulting in less lateral expansion of the core. The lower lateral expansion of core concrete delays the utilization of transverse reinforcement.

Reported data[12-27,30] indicate that in the case of HSC, little improvement in strength and ductility is obtained when the volumetric ratio of transverse reinforcement is small. For instance, Bjerkeli et al.[19] report that a volumetric ratio of 1.1 percent was not sufficient to generate any improvement in column behavior, while the use of 3.1 percent resulted in columns performing in a ductile manner.

Sugano et al.,[32] Hatanaka et al.,[23] and Saatcioglu et al.[27,30] report a correlation between the non-dimensional parameter, $\rho_s f_{yt}/f'_c$, and axial ductility of HSC columns subjected to concentric loads. Figure 3 shows the relationship between this parameter and axial ductility of columns with different compressive strengths. In this figure, the axial ductility of columns is represented by the ratio $\varepsilon_{85}/\varepsilon_{01}$, where ε_{85} is the axial strain in core concrete when column load on the descending branch is reduced to 85 percent of the peak value and ε_{01} is the axial strain corresponding to peak stress of plain concrete. For each pair of columns compared, similar reinforcement arrangements and tie spacings were maintained. As indicated in this figure, columns of different compressive strength having the same $\rho_s f_{yt}/f'_c$ value result in almost the same axial ductility, provided that certain minimum limitations are met for the volumetric ratio and spacing of transverse reinforcement.[30]

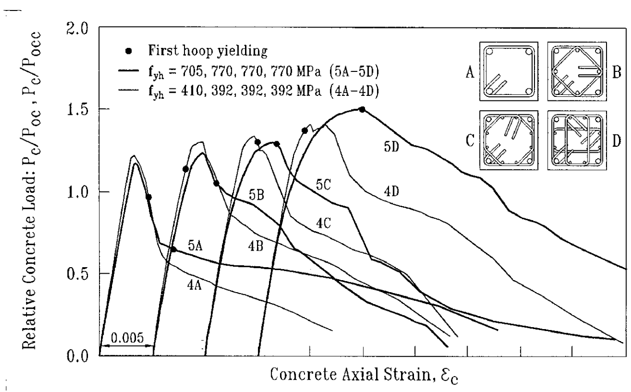

Fig. 5—Effect of transverse reinforcement yield strength (adapted from Ref. 25)

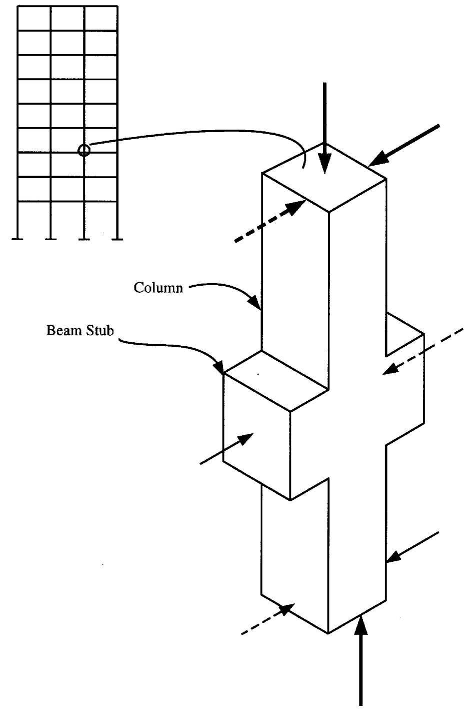

Fig. 6—Overall view of test specimens

Figure 4 shows the relationship between the parameter $\rho_s f_{yt}/f'_c$ and the ratio of experimentally obtained axial load capacity for 111 HSC columns to that predicted by Eq. 1. From this plot it could be observed that columns with a low volumetric ratio of transverse reinforcement may not achieve their strength as calculated by ACI 318-89; however, well-confined columns can result in strength in excess of that calculated by ACI 318-89. Excess strength of columns with relatively higher amounts of transverse reinforcement is generally obtained after spalling of cover concrete. This strength enhancement comes as a result of an increase in strength of the confined core concrete.

2.3—Effect of longitudinal and transverse reinforcement strength

The yield strength of the confinement steel determines the upper limit of the confining pressure. A higher confining pressure applied to the core concrete, in turn, results in higher strength and ductility. Figure 5 shows normalized axial load-axial strain response of core concrete for four pairs of HSC columns.[25] For each pair of columns, all parameters were kept constant except the yield strength of the transverse reinforcement. The yield strength of transverse reinforcement for columns 4A, 4B, 4C, and 4D and columns 5A, 5B, 5C, and 5D was approximately 400 MPa and 700 MPa, respectively. As indicated in this figure, for well confined columns (C and D), increasing the yield strength of transverse reinforcement results in an increase in strength and ductility. However, for type A columns, where only peripheral ties are provided, the gain in strength and ductility is negligible. Reported data of HSC columns[17,25,27] indicate that when high-strength concrete is used in well-confined columns, the full yield strength of transverse reinforcement is utilized. On the other hand, in a poorly confined HSC column, tensile stresses that develop in the transverse reinforcement remain below yield strength even at the time of column failure.

2.4—Effect of longitudinal and transverse reinforcement arrangement

Well-distributed longitudinal and transverse reinforcement results in a larger effectively confined concrete area and more uniform distribution of the confining pressure, thereby improving the effectiveness of the confining reinforcement. In the case of NSC,[30,33] the arrangement of the transverse reinforcement and laterally supported longitudinal reinforcement has been shown to have a significant influence on strength and ductility of columns. Similar observations have been reported in the case of HSC columns.[17,27,30] Transverse reinforcement in the form of single peripheral hoops has been shown to result in very low strength and ductility of HSC columns.[17,25,27] Similar observations have also been reported for NSC columns.[34]

More detailed discussions of the behavior of HSC columns subjected to concentric axial load are presented in Refs. 25 and 30.

Table 1— Comparison of calculated and experimental flexural strengths for specimens tested by Bing et al. (adapted from reference 40)

Specimen number	Axial load level P/f'_cA_g	f'_c MPa	f_y of ties MPa	$M_{EXP}/M_{NZS3101}$	M_{EXP}/M_{MOD}
1	0.3	98	1317	0.94	0.99
2	0.3	98	453	0.98	1.03
3	0.6	93	1317	0.87	1.08
4	0.6	93	453	0.86	1.07
5	0.6	93	1317	0.82	1.02

Table 2— Comparison of calculated and experimental flexural strengths (adapted from reference 42)

Specimen number	Axial load level P/P_o*	f'_c MPa	M_{EXP}/M_{ACI}	M_{EXP}/M_{MOD}
1	0.2	54	1.25	—
2	0.2	51	1.23	—
3	0.2	101	0.93	1.04
4	0.2	100	1.01	1.14
5	0.2	102	0.91	1.02
6	0.2	102	0.98	1.1
7	0.3	104	0.87	1.12

*$P_o = 0.85 f'_c (A_g - A_{st}) + A_{st} f_y$

CHAPTER 3—PERFORMANCE OF HSC COLUMNS UNDER COMBINED AXIAL LOAD AND BENDING MOMENT

Two major questions must be addressed when designing HSC columns. First, does the rectangular stress block described in Section 10.2.7 of ACI 318-89 apply to HSC? Second, are the confinement rules given in ACI 318-89 Sections 10.9.3 and 21.4.4 adequate for HSC? In regions of high seismicity, a major concern has been the ductility of HSC columns, resulting in a reluctance to use HSC in these areas compared with regions of low seismicity. As a result, the focus of most reported investigations[32,35-43] on performance of HSC columns under combined loading has been primarily to comprehend the seismic behavior of these columns. Some of these studies also have presented data that could be used to assess the flexural capacity of HSC columns subjected to combined loading. However, available data for HSC columns subjected to combined loading are relatively limited compared with HSC columns subjected to concentric loading.

To date, most experimental research has involved testing of scaled columns. Figure 6 shows a general configuration of a typical column specimen used in most reported studies. This type of specimen represents half of the upper and lower column, together with a small portion of the floor beam. These specimens are usually subjected to constant axial load and to a repeated lateral displacement sequence similar to the one shown in Fig. 7. This type of specimen is designed so that no damage is inflicted on the beam-column joint.

3.1—Flexural strength

There is no universal agreement on the applicability of ACI 318-89 code requirements for calculating flexural strength of HSC column sections subjected to combined axial load and bending moment.

Columns are usually designed for combined axial load and bending moment using the rectangular stress block defined in ACI 318-89 Section 10.2.7. This stress block was originally derived by Mattock et al.,[44] based on tests of unreinforced concrete columns loaded with axial load and moments so as to have the neutral axis on one face of the test specimen.[45] The concrete strengths ranged up to 52.5 MPa. The stress block was defined by two parameters: the intensity of stress in the stress block, which was designated as α_1; and the ratio of the depth of the stress block to the depth of the neutral axis, which was designated as β_1. Mattock et al.[44] proposed $\alpha_1 = 0.85$ and β_1 as follows:

$$\beta_1 = 1.05 - 0.05(f'_c/6.9) \text{ but not more than } 0.85 \quad (2)$$

for f'_c in MPa. That proposal was incorporated into Sec. 1504g of ACI 318-63.

Based on similar tests of concrete columns with concrete strengths ranging from 79 to 98 MPa, Nedderman[46] proposed a lower limit on β_1 of 0.65 for concrete strengths in excess of 55 MPa. This limit was incorporated in ACI 318-77. Similar tests were carried out by Kaar et al.[47] on concretes with compressive strength ranging from 24 to 102 MPa and by Swartz et al.[48] on concretes ranging from 58 to 77 MPa in compressive strength.

When the equation for β_1 was compared with the test data, a conservative lower bound was selected and the product $\alpha_1\beta_1$ was shown to lead to a conservative estimate for the total compression force in concrete in an eccentrically loaded column. For a rectangular stress block, the distance from the resultant compressive force in concrete to the centroid of the rectangular cross-section is ($h/2$ - $\beta_1 c/2$), where h is the total depth of the cross-section. A conservative lower bound estimate of β_1 leads to an overestimation of this distance and, hence, to an overestimation of the moment resisted by compression in the concrete. This is most serious for columns failing in compression, and with e/h ratios less than about 0.3, where e = eccentricity of axial load and h = overall thickness of the column cross-section.

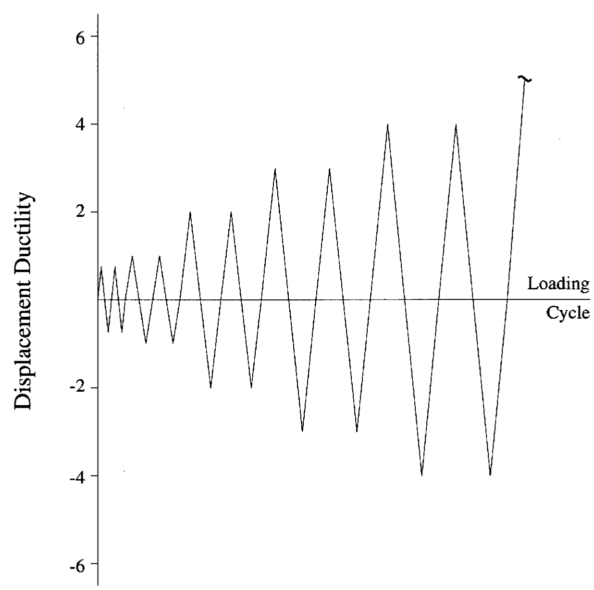

Fig. 7—Lateral displacement sequence

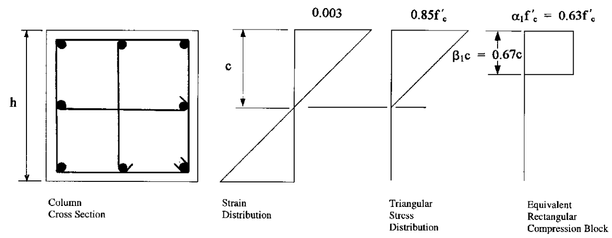

Fig. 8—Modified compression stress block

Table 1 gives a comparison of calculated and experimental flexural strengths of five column specimens tested by Bing et al.[40] As indicated in this table, the ratios of the experimentally obtained flexural strength to that calculated according to the New Zealand Standard (NZS) 3101 procedures (the same as ACI 318-89 requirements) are less than 1, especially for columns subjected to higher axial load levels. Based on these tests, Bing et al. have suggested that an equivalent rectangular compressive stress block with an average stress, $\alpha_1 f'_c$, and a depth, $a = \beta_1 c$, be used in design of HSC column cross-sections, where:

$$\alpha_1 = 0.85, \text{ for } f'_c \leq 55 \text{ MPa}$$

and

$$\alpha_1 = 0.85 - 0.004\,(f'_c - 55) \geq 0.75, \text{ for } f'_c > 55 \text{ MPa}$$

Table 1 also gives the ratio of the experimentally obtained flexural strength for test columns to that calculated by the modified procedure. As indicated in Table 1, the modified procedure gives a better estimation of test results. For the type of specimens tested by Bing et al., flexural strengths obtained from tests are usually 10 to 25 percent higher than the calculated values when NSC is used. This higher strength is attributed primarily to confinement provided by the beam stub in the critical region of the test column. See Fig. 6 for the type of test column used in that testing program.

Table 2 gives a comparison of experimentally obtained flexural strengths for some of the test columns reported in Ref. 42 to those calculated by ACI 318-89 requirements. As indicated in this table, the ACI 318-89 procedure results in reasonable calculation of flexural strengths for test columns with concrete compressive strengths equal to 54 and 51 MPa. The conservatism of the ACI 318-89 procedure in calculating flexural strength of these two test columns is similar to NSC columns. As stated earlier, NSC column tests usually give 10 to 25 percent higher flexural strength than that calculated by ACI 318-89 procedure for the type of test columns used. However, as concrete compressive strength or level of axial load increases, the ratio of experimentally obtained flexural strength to that calculated by ACI 318-89 procedures decreases and falls below 1, as indicated by Table 2. This is especially true for the test column with an axial load equivalent to 30 percent of the axial load strength of the column. Those authors[42] offer the following explanation for this observation.

Available test data indicate that typical stress-strain curves in compression for HSC are characterized by an ascending portion that is primarily linear, with maximum strength achieved at an axial strain between approximately 0.0024 and 0.003. Therefore, it may be more appropriate to use a triangular compression stress block having properties shown in Fig. 8 for calculating the flexural strength of HSC columns when f'_c exceeds approximately 70 MPa. In this approach, the maximum compressive stress is assumed to be 0.85 f'_c at an axial compressive strain of 0.003. Considering the equilibrium of horizontal forces and moment equilibrium, it can be shown that the equivalent rectangular compression block shown in Fig. 8 has the following properties: intensity of compression stress equals 0.63 f'_c rather than 0.85 f'_c, the value currently specified in ACI 318-89, and the depth of the rectangular compression block is equal to 0.67 times the depth of the neutral axis, corresponding approximately to current ACI 318-89 requirements for f'_c greater than 55 MPa. Those authors[42] recommend that, until further research is conducted, the following equivalent rectangular compression block be adopted for calculating the nominal moment strength of concrete columns with f'_c exceeding 70 MPa and designed according to seismic provisions of ACI 318-89: When f'_c exceeds 70 MPa, the stress intensity of an equivalent rectangular compression block must be decreased linearly from 0.85 to 0.6, using the expression

$$\alpha_1 = 0.85 - 0.0073(f'_c - 69) \geq 0.6$$

for f'_c in MPa. Table 2 also gives the ratios of experimentally obtained flexural strength to the strength using the modified procedure described above for the five test columns having $f'_c \geq 100$ MPa.

A comprehensive investigation assessing the applicability of the rectangular compression block specified in ACI 318-89 for computing flexural strength of HSC columns is reported by Ibrahim and MacGregor.[49] The objective of the research project was to investigate the applicability of the rectangular stress block to HSC. The experimental phase of the investigation consisted of testing a total of 21 C-shaped specimens, 15 of which had rectangular cross-sections and six of which had triangular cross-sections having the extreme compression fiber at the tip of the triangle. The rectangular specimens included three plain specimens, while the triangular specimens included two plain specimens. The test specimens were loaded so that the entire cross section was subjected to compressive force, with the strain at one face remaining zero. The main variables were concrete strength, shape of cross section, and amount of transverse steel. The study was limited to relatively low longitudinal and transverse reinforcement ratios. The volumetric ratios of ties ranged from those required for non-seismic design to the minimum required for seismic design according to ACI 318-89.

Ibrahim et al.[49] compared the concrete component of the measured load and moment strengths of 94 tests of eccentrically loaded columns with the strengths computed using the ACI 318-89 for columns with concrete strengths ranging up to 130 MPa. For fifty-five percent of the tests, the concrete component of the strength was less than that calculated by ACI 318-89. There was a definite downward trend in the strength ratios as f'_c increased. Those authors concluded that the ACI 318-89 stress block needed revision for HSC.

Those authors[49] also reported that, for all specimens, the maximum concrete compressive strains before spalling were greater than 0.003, and concluded the following:

1. The rectangular stress block can be used to design HSC cross-sections with some modification to the parameters used to define the stress block.

2. The constant value of 0.85 for the compressive stress intensity factor as currently used by ACI 318-89 is unconservative for HSC and the following modified value should be used:

$$\alpha_1 = (0.85\text{-}0.00125\, f'_c) \geq 0.725 \ (f'_c \text{ in MPa})$$

3. The distance from extreme compression face to centroid of the rectangular compression block (parameter $\beta_1\, c/2$) as specified by ACI 318-89 leads to an overestimation of the lever arm. They proposed the following equation:

$$\beta_1 = (0.95\text{-}0.0025\, f'_c) \geq 0.70 \quad (f'_c \text{ in MPa})$$

It has been reported[50,51] that ACI 318-89 provisions give a good estimation of flexural strengths of HSC beams. When a cross section is subjected to a bending moment only, the depth of the neutral axis at ultimate conditions is generally small and the shape of the compression block becomes less important. However, in the case of columns, the depth of the neutral axis is a significant portion of the member's overall depth, particularly if the level of axial load is relatively high, making the nominal moment capacity more sensitive to the assumed shape of the compression block.

The Canadian Code for Design of Concrete Structures[52] treats the flexural stress block for HSC in two ways. Design may be based on equations for the stress-strain curves of the concrete with peak stresses no greater than $0.9f'_c$. Alternatively, a modified rectangular stress block is defined by:

$$\alpha_1 = 0.85 - 0.0015f'_c \geq 0.67 \ \ (f'_c \text{ in MPa})$$
$$\beta_1 = 0.97 - 0.0025f'_c \geq 0.67 \ \ (f'_c \text{ in MPa})$$

These two equations were based in part on those proposed by Ibrahim et al.[49] with the further provision that they represent a stress-strain curve with peak stress not greater than 0.9 f'_c. Additionally, the Canadian code allows using 0.0035 as the maximum concrete strain.

The Canadian code (52) specifies that its strength equations and related detailing rules are applicable for concretes with f'_c ranging from 20 MPa to 80 MPa. Concrete strengths higher than 80 MPa are permitted if the designer can establish structural properties and detailing requirements for the concrete to be used. However, the Canadian code limits the range of f'_c in members resisting earthquake-induced forces to 20 MPa to 55 MPa for normal density concretes and 20 MPa to 30 MPa for structural low density concretes.

Test results on column specimens with compressive strength from about 50 to 55 MPa, and subjected to combined axial load and bending moment, have been reported by Sheikh.[53] Test columns were subjected to high axial loads ($0.6\, f'_c A_g$ to $0.7\, f'_c A_g$). Results indicate that, at this level of concrete compressive strength, ACI 318-89 procedures conservatively predict the flexural strength of the columns. Based on data reported by Sheikh and the two column tests (with f'_c = 54 and 51 MPa) given in Table 2, it could be concluded that the flexural strength of columns with a level of confinement prescribed by seismic provisions of ACI 318-89 and compressive strength below 55 MPa could be calculated conservatively by ACI 318-89 procedures.

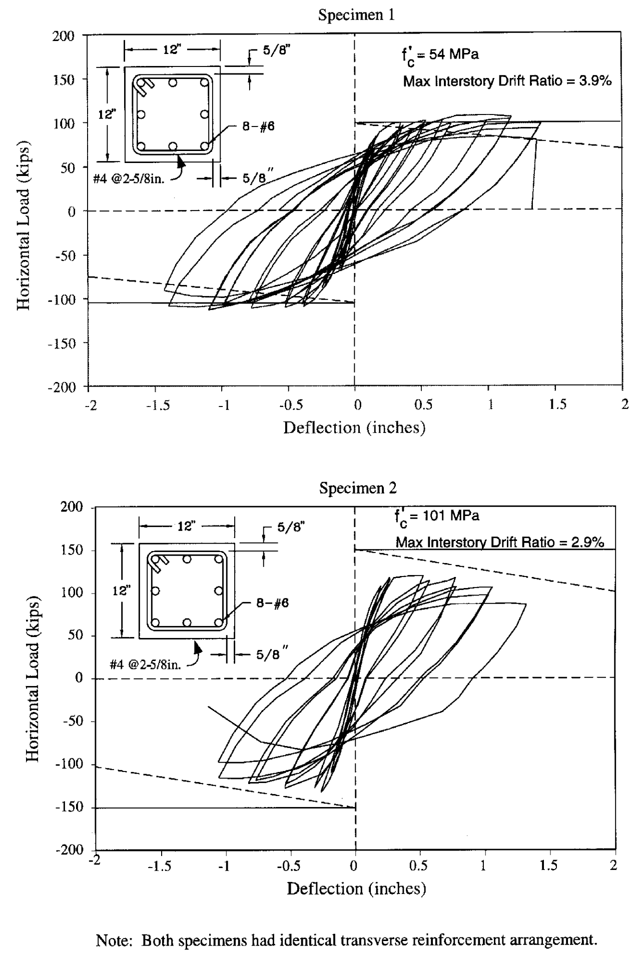

Fig. 9—Effect of concrete compressive strength on ductility (adapted from Ref. 42)

3.2—Ductility of HSC columns under combined axial load and bending moment

Reinforced concrete columns in moment-resisting frames constructed in areas of high seismicity should be proportioned to have adequate curvature and displacement properties. This requirement has arisen, in part, as a result of observations[54-57] of field performance of columns after major earthquakes, which indicate that, despite following the strong-column weak-beam concept in design,[58-62] damage could occur at ends of the columns. Therefore, it becomes necessary for reinforced concrete columns to be proportioned in such a way that they are capable of inelastic response without appreciably losing load-carrying capacity.

Table 3— Effect of yield strength of ties (adapted from reference 42)

Specimen number	f'_c MPa	f_y MPa	Maximum drift index, percent
1	100	414	4
2	102	828	3.9

Table 4— Effect of yield strength of ties (adapted from reference 42)

Specimen number	f'_c MPa	f_y MPa	Tie spacing (mm)	Maximum drift index, percent
1	100	414	41	4
2	102	828	67	2.8

One of the ways in which building codes, such as those in the U.S.,[63] ensure such ductility in columns is by specifying the amount of transverse reinforcement in critical regions of columns. However, these equations are empirical and based on strength criteria, though intended to provide ductility. Through experimental testing of NSC columns it has been shown that, although these equations are based on strength criteria, they also provide adequate ductility for reinforced NSC columns.[31,34] The extension of these equations to the case of HSC columns has been questioned.

Using the type of specimen shown in Fig. 6 and loading procedures depicted in Fig. 7, researchers have investigated effects of concrete compressive strength, type, spacing, amount and yield strength of transverse reinforcement, and level of axial loads on ductility of HSC columns. Following is a brief description of some parameters affecting the performance of HSC columns under combined and repeated loading.

3.2.1—*Effect of concrete compressive strength and axial load on ductility*

An increase in concrete compressive strength tends to result in lower ductility. Ductility also is affected adversely by an increase in the level of axial load applied to the column.

Lateral load versus lateral displacement diagrams are shown in Fig. 9 for two columns tested using the test setup shown in Fig. 6.[42] These results can be compared for effect of concrete compressive strength on ductility. The concrete compressive strength, the amount of transverse reinforcement in the critical regions of each column, and maximum interstory drift ratio for each column prior to failure are indicated in Fig. 9. Both columns were subjected to constant axial load level, equivalent to 20 percent of the axial load capacity of the columns. For both specimens, the spacing, amount, type, and yield strength of transverse reinforcement was the same. Both specimens had identical longitudinal steel arrangement. Both specimens used #4, Grade 60 (414 MPa yield strength) peripheral hoops at 64-mm spacing. Seismic provisions of ACI 318-89 require a larger amount of transverse reinforcement for Specimen 2 due to the higher concrete compressive strength. As indicated in Fig. 9, increasing concrete compressive strength from 54 MPa to 101 MPa resulted in almost 25 percent reduction in the maximum interstory drift ratio of Specimen 2.

This reduced interstory drift ratio, however, should not be interpreted as evidence that HSC should not be constructed in areas of high seismicity. Assuming that a 4 percent interstory drift ratio represents a very good level of ductility, Azizinamini et al.[42] report that when axial load levels are below 20 percent of column axial load capacity (which is the case for most columns encountered in seismic design), adequate ductility exists for columns with transverse reinforcement levels that are even slightly below the seismic requirement of ACI 318-89. The type of test column used in their investigation was similar to that shown in Fig. 6. Figure 10 shows the cyclic lateral load versus lateral deflection relationships for two of the specimens tested. Specimens 3 and 4, whose response is shown in, had concrete compressive strengths of approximately 50 and 100 MPa, respectively, at the time of testing. Both specimens used #3, Grade 60 peripheral hoops and cross ties spaced at 38-mm. It can be observed from this figure that, although increasing concrete

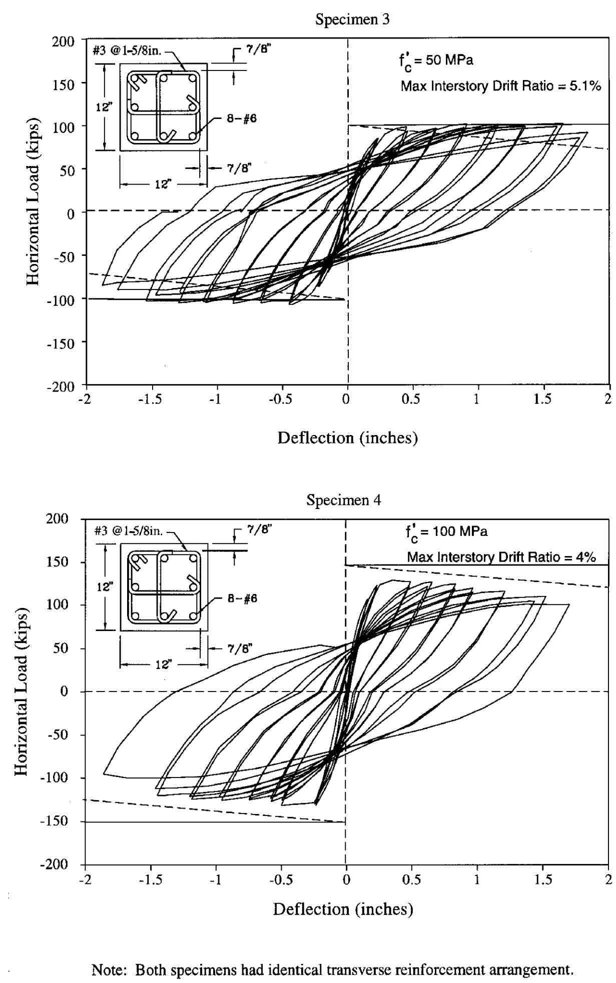

Fig. 10—Effect of concrete compressive strength on ductility (adapted from Ref. 42)

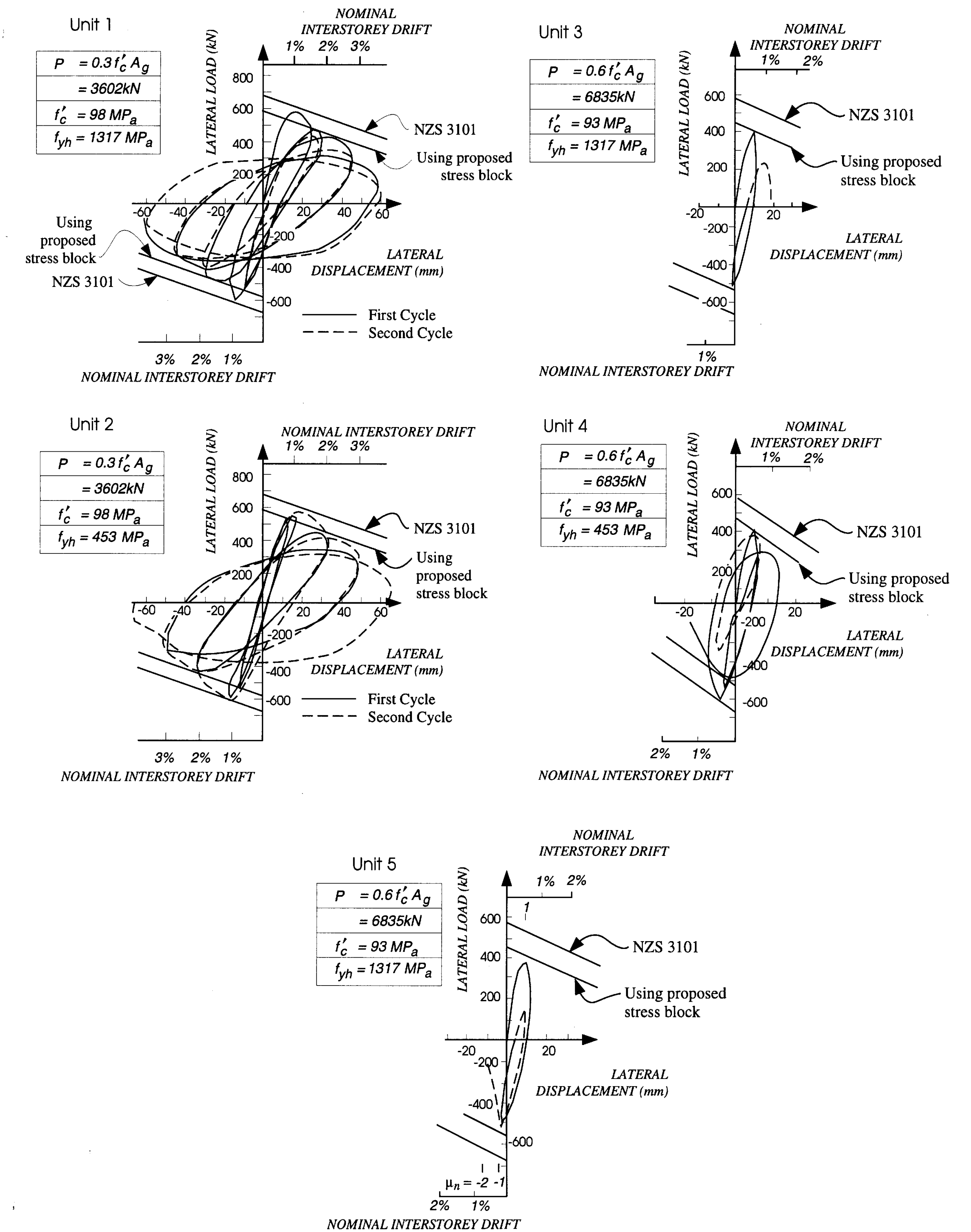

Fig. 11—Measured lateral load vs. lateral displacement hysteresis loops of columns (adapted from Ref. 40)

compressive strength resulted in a decrease in the maximum interstory drift ratio, Specimen 4, with 100 MPa concrete compressive strength, still exhibited a good level of ductility (4 percent interstory drift ratio). Both columns were subjected to constant axial load corresponding to 20 percent of their respective concentric axial load capacities.

Further evidence that HSC columns are able to behave in a ductile manner under relatively small axial load levels (below 20 percent of concentric axial load capacity) is provided by test results reported by Thomsen et al.[41] Those authors report results of tests on twelve relatively small columns (150-mm square cross section) with compressive strength of approximately 83 MPa. These specimens were subjected to constant axial load and repeated lateral loads. The level of axial load used by these investigators varied between 0 percent and 20 percent of the concentric axial load capacities of the columns. Those authors report that all columns were able to sustain 4 percent interstory drift ratio before failure, which was characterized by buckling of longitudinal bars.

Data are limited on ductility of HSC columns with axial load in the range of 20 percent to 30 percent of column axial load capacity.

In general, when the level of axial load is above 40 percent of column axial load capacity and concrete compressive strength is approximately 100 MPa, larger amounts of transverse reinforcement than specified in the seismic provisions of ACI 318-89 are needed. Test results indicate[36,37] that when the level of axial load is high, the use of transverse reinforcement having high yield strength could be necessary because of high confinement demands.

The behavior of HSC columns under combined bending moment and relatively high axial load was investigated by Bing et al.[40] Those authors report tests on five square HSC columns having an overall configuration similar to the specimen shown in Fig. 6. Each specimen had a 350 x 350-mm cross-section and was subjected to constant axial load and repeated lateral loads. Table 1 gives concrete compressive strength, level of applied constant axial load, and yield strength of transverse reinforcement for each test column. Figure 11 shows the lateral load vs. lateral displacement behavior for each column. The amounts of transverse reinforcement provided in the test regions of specimens 1, 2, 3, 4, and 5 (designated as unit 1, 2, 3, 4, and 5 in Fig. 11) were 133 percent, 103 percent, 131 percent, 108 percent, and 92 percent, respectively, of NZS 3101 requirements.[58] For Specimens 1 and 2, seismic provisions of ACI 318-89 would require 1.06 times as much transverse reinforcement as that specified by NZS 3101. For Specimens 3, 4, and 5, on the other hand, seismic provisions of ACI 318-89 would require 0.62 times as much transverse reinforcement as NZS 3101. This difference stems from the fact that NZS 3101 requirements include the effect of axial load level in calculating the required amount of transverse reinforcement for columns. As indicated in Table 1 and Fig. 11, the level of axial load on the test columns was relatively high (either 0.3 $f'_c A_g$ or 0.6 $f'_c A_g$). From these data, those authors concluded that the ductility of HSC columns designed on the basis of NZS 3101 is not adequate and that higher amounts of transverse reinforcement would be needed, especially when axial load levels are relatively high. Given the fact that at high axial load levels, ACI 318-89 seismic provisions require a lower amount of transverse reinforcement than that specified by NZS 3101 requirements, it could be concluded that for columns subjected to high axial load levels, the amount of transverse reinforcement specified by the seismic provisions of ACI 318-89 is not adequate.

Information is limited on columns with loads in excess of 0.6 times the axial load capacity and concrete compressive strength above 70 MPa. Muguruma et al.[39] report tests on twelve 200-mm square HSC columns, having geometry similar to that shown in Fig. 6, with concrete compressive strengths exceeding 120 MPa at the time of testing. Two of the variables investigated by these authors were concrete compressive strength (80 to 120 MPa) and level of axial load (25 percent to 63 percent of column's axial load capacities). One of the major conclusions drawn by those authors is that square HSC columns could be made to behave in a ductile manner, even at high axial load levels, by using high yield strength transverse reinforcement. However, two points deserve closer examination when interpreting their test results: (a) The amount of transverse reinforcement provided for test columns was as high as 230 percent of that required by seismic provisions of ACI 318-89; and (b) for a relatively small column cross-section (200 x 200 mm), Muguruma et al.[39] used an arrangement of 12 longitudinal bars with an extremely congested scheme of transverse reinforcement.

When concrete compressive strength is below 55 MPa, test data indicate that even at high axial load levels, ductility comparable to NSC columns could be achieved.[53] From a review of reported data, the following conclusions could be made with regard to the seismic behavior of HSC columns:

1. Columns with concrete compressive strength of approximately 55 MPa exhibited an acceptable level of ductility, even at high axial load levels.

2. Columns that had approximately 100-MPa concrete and axial loads below 20 percent of column axial load capacity, and that were designed based on seismic provisions of ACI 318-89, exhibited adequate ductility.

3. Data are limited to evaluate ductility of HSC columns with axial loads in the range of 20 percent to 30 percent of column axial load capacities.

4. Columns with concrete compressive strength of approximately 100 MPa and with axial loads above 30 percent of column axial load capacity require higher amounts of transverse reinforcement than that required by seismic provisions of ACI 318-89. Furthermore, in this range of axial load levels, higher yield strength transverse reinforcement might be necessary. Few data are available to provide design guidelines in this range of axial load levels.

3.2.2—*Effect of yield strength of transverse reinforcement*

High yield strength transverse reinforcement (yield strength exceeding 800 MPa) has been shown to be advantageous when the level of axial loads is high (above 40 percent of column axial load capacity). Figure 12 shows bending moment caused by lateral loads versus lateral displacement response of two test columns reported by Muguruma et al.,[39] with configuration similar to that shown in Fig. 6. All transverse reinforcement details, except for yield strength, were identical for both specimens. Both specimens were subjected to axial load levels of approximately 60 percent of the column axial load capacity. Compressive strength of concrete for both specimens was 85.7 MPa at the time of testing. Specimens AL-2 and AH-2 used 328- and 792-MPa yield-strength transverse reinforcement, respectively. Specimen AH-2, which used higher-yield-strength steel for transverse reinforcement, was able to sustain a 4 percent interstory drift ratio prior to failure. However, the amount of transverse reinforcement provided for specimen AH-2 was approximately 230 percent of that required by the seismic provisions of the ACI 318-89. Response of specimen AL-2, which had 328-MPa transverse reinforcement in an amount approximately equal to requirements of seismic provisions of ACI 318-89, was unsatisfactory, as indicated in Fig. 12.

When the level of axial load is relatively low (less than or equal to 20 percent of axial load capacity), the use of higher-yield-strength steel for transverse reinforcement may not result in any improvement in strength and ductility of HSC columns.[39,42] Table 3 shows results of two test columns that could be compared for effect of yield strength of transverse reinforcement on ductility of HSC columns. Both specimens were subjected to constant axial load levels equivalent to 20 percent of the column capacity, and to repeated lateral loads. Both specimens failed at an interstory drift ratio of approximately 4 percent. This result indicates that increasing the yield strength of the transverse reinforcement has no influence on the ductility capacity of HSC columns at relatively low axial load levels. The yielding of transverse reinforcement for specimen #1 (using conventional-yield-strength transverse reinforcement) in Table 3 was observed just prior to achievement of 4 percent interstory drift ratio, coinciding with failure of the specimen. Therefore, it would be anticipated that increasing the yield strength of transverse reinforcement would have no significant effect.

When the level of axial load is low, an additional consideration should also be taken into account. One reason for using higher strength steel for transverse reinforcement in HSC columns is to allow larger spacing of ties. However, one should be very careful in using this approach, particularly when the level of axial load is low. This point could be explained by comparing the behavior of the two test columns given in Table 4. The spacing of transverse reinforcement for Specimens 1 and 2 in Table 4 was 41 and 67 mm, respectively. Because higher-yield-strength transverse steel (828 MPa) was used, Specimen 2 had larger spacing and, in addition, a larger transverse reinforcement ratio than that required by the seismic provisions of ACI 318-89 (105 percent). On the other hand, Specimen 1, which had 414-MPa transverse reinforcement, used smaller spacing and only 76 percent of the transverse reinforcement required by the seismic provisions of ACI 318-89. Table 4 indicates that Specimen 2 failed at an interstory drift ratio of 2.8 percent, while Specimen 1, which had a smaller amount of transverse reinforcement, failed at an interstory drift ratio of 4 percent. The use of high strength steel for transverse reinforcement could result in satisfying the transverse reinforcement requirements at a larger tie spacing. However, the larger tie spacing may lead to early buckling of longitudinal bars, as was the case for Specimen 2.

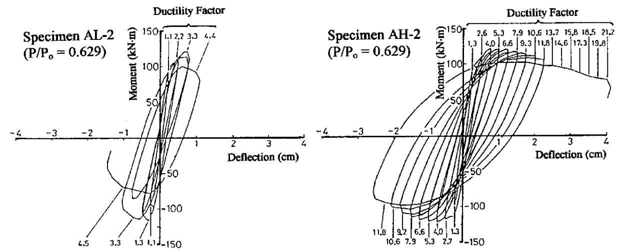

Fig. 12—Effect of yield strength of transverse reinforcement (adapted from Ref. 39)

In summary, the reported data support the following conclusions regarding the use of high-yield-strength transverse reinforcement in seismic areas:

1. Use of high-yield-strength steel as transverse reinforcement is beneficial when axial loads are relatively high (above 40 percent of the axial load capacity of the column). Further, at these axial load levels, more transverse reinforcement than that required by the seismic provisions of ACI 318-89 is required to achieve adequate ductilities. The reported information is not sufficient to establish this required transverse reinforcement level.

2. When axial load is below 20 percent of the axial load capacity of the column, use of high-yield-strength steel for transverse reinforcement is not recommended. Although such practice could result in satisfying the code requirements, the larger permissible tie spacing may lead to premature buckling of the longitudinal steel and to lower ductility than would be achieved with lower strength steel.

CHAPTER 4—RECOMMENDED RESEARCH

Effects of the parameters related to concrete and transverse reinforcement and their influence on the behavior of HSC columns were discussed. The available information was divided into two general categories (columns under concentric loads and columns subjected to the combined effects of axial load and bending moment), and the effects of different parameters on the behavior of HSC columns were discussed separately for each category. Based on these discussions, additional areas of research are identified for increasing the understanding of behavior of HSC columns. These areas of research include: a) effect of sustained loads; b) effect of loading rate; c) effect of concrete mix proportions such as ratio of coarse to fine aggregate and silica fume; d) effect of different arrangements of transverse reinforcement on ductility under combined axial and repeated lateral loads; e) fire resistance of HSC columns; and f) additional physical tests on the effects of axial load, transverse reinforcement yield strength, longitudinal reinforcement arrangement, and transverse reinforcement layout on ductility of HSC columns under combined axial and repeated lateral loads.

CHAPTER 5—REFERENCES

1. Russell, H. G., “High-Strength Concrete in North America,” *Proceedings* of Symposium on Utilization of High-Strength Concrete, Stavenger, Norway, June 15-18, 1987.

2. “Economical Composite System is Stiffened with Record 19,000 psi Mix,” *Engineering News-Record*, Feb. 16, 1989.

3. “High-Strength Concrete for Pacific First Center,” *Concrete International*, April 1989.

4. Shah, S. P.; Zia, P.; and Johnston, D., “Economic Considerations for Using High-Strength Concrete in High Rise Buildings,” a study prepared for Elborg Technology Co., December 1983.

5. Kunze, W. E., “High-Strength Concrete in the United States,” *Proceedings* of XIV Biennial Conference in Concrete, Concrete Institute of Australia, May 1989.

6. Luther, M. D., and Bauer, K. C., “Using High-Strength Concrete Simplifies Precast Column Design,” *Concrete Construction*, June 1987.

7. Schmidt, W., and Hoffman, E. D., “9000 PSI Concrete—Why? Why Not?,” *Civil Engineering*, ASCE, V. 45, No. 5, May 1985.

8. “Computer Cuts Column Costs,” *Concrete Construction*, V. 31, No. 5, May 1986.

9. Moreno, J., and Zils, J., “Optimization of High-Rise Concrete Buildings,” *Analysis and Design of High-Rise Buildings*, SP-97, ACI International, Detroit, 1985.

10. Smith, J. G., and Rad, F. N., “Economic Advantages of High-Strength Concrete in Columns,” *Concrete International*, April 1989.

11. Colaco, J. P., “75-Story Texas Commerce Plaza, Houston—The Use of High-Strength Concrete,” *High-Strength Concrete*, SP-87, ACI International, Detroit, 1985.

12. Muguruma, H.; Watanabe, F.; Iwashimizu, T.; and Mitsueda, R., “Ductility Improvement of High-Strength Concrete by Lateral Confinement,” *Transactions of the Japan Concrete Institute*, Vol. 5, 1983, pp. 403-410.

13. Martinez, S.; Nilson, A. H.; and Slate, F. O., “Spirally Reinforced High-Strength Concrete Columns,” *Research Report* No. 82-10, Department of Structural Engineering, Cornell University, Ithaca, August 1982, 255 pp.

14. Martinez, S.; Nilson, A. H.; and Slate, F. O., “Spirally Reinforced High-Strength Concrete Columns,” ACI JOURNAL, Sept.-Oct. 1984, pp. 431-442.

15. Basset, R., “Behavior of Reinforced High Strength Lightweight Concrete Columns with Rectangular Ties,” M.A.Sc. Thesis, University of Toronto, Toronto, 1983, 108 pp.

16. Basset, R., and Uzumeri, S. M., “Effect of Confinement on the Behavior of High Strength Lightweight Concrete Columns,” *Canadian Journal of Civil Engineering*, Vol. 13, No. 6, Dec. 1986, pp. 741-751.

17. Yong, Y. K.; Nour, M. G.; and Nawy, E. G., “Behavior of Laterally Confined High-Strength Concrete under Axial Loads,” ASCE *Journal of Structural Engineering*, Vol. 114, No. 2, 1988, pp. 332-351.

18. Abdel-Fattah, H., and Ahmed, S. H., “Behavior of Hoop Confined High-Strength Concrete under Axial and Shear Loads,” ACI *Structural Journal*, V. 86, Nov.-Dec. 1989, pp. 652-659.

19. Bjerkeli, L.; Tomaszewica, A.; and Jensen, J. J., “Deformation Properties and Ductility of High-Strength Concrete,” *High-Strength Concrete*, Second International Symposium, SP-121-12, ACI International, 1990, pp. 215-238.

20. Rangan, B. V.; Saunders, P.; and Seng, E. J., “Design of High-Strength Concrete Columns,” SP 128-52, ACI International, 1991, pp. 851-862.

21. Nagashima, T.; Sugano, S.; Kimura, H.; Ichikawa, A., “Monotonic Axial Compression Test on Ultra-High-Strength Concrete Tied Columns,” *Proceedings* of 10th World Conference on Earthquake Engineering, Madrid, Spain, 1992, pp. 2983-2988.

22. Itakura, Y., and Yagenji, A., “Compressive Test on High-Strength R/C Columns and Their Analysis Based on Energy Concept,” *Proceedings* of 10th World Conference on Earthquake Engineering, Madrid, Spain, 1992, pp. 2599-2602.

23. Hatanaka, S., and Tanigawa, Y., “Lateral Pressure Requirements for Compressive Concrete,” *Proceedings* of 10th World Conference on Earthquake Engineering, Madrid, Spain, 1992, pp. 2603-2608.

24. Galeota, D.; Giammatteo, M. M.; and Marino, R., “Strength and Ductility of Confined High-Strength Concrete,” *Proceedings* of 10th World Conference on Earthquake Engineering, 1992, Madrid, Spain, pp. 2609-2613.

25. Cusson, D., and Paultre, P., “High-Strength Concrete Columns Confined by Rectangular Ties,” ASCE *Journal of Structural Engineering*, Vol. 120, No. 3, March 1994, pp. 783-804.

26. Polat, M. B., Behavior of Normal and High-Strength Concrete under Axial Compression,” M.A. Sc. Thesis, Department of Civil Engineering, University of Toronto, 1992, 175 pp.

27. Saatcioglu, M., and Razvi, S., “Behavior of Confined High-Strength Concrete Columns,” *Proceedings* of CPCA/CSCE Structural Concrete Conference, Toronto, May 19-21, 1993, pp. 37-50.

28. Nishiyama, M.; Fukushima, I.; Watanabe, F.; and Muguruma, H., “Axial Loading Tests on High-Strength Concrete Prisms Confined by Ordinary and High-Strength Steel,” *Symposium on Utilization of High-Strength Concrete*, Lillehammer, Norway, 1993, 1, pp. 322-329.

29. Collins, M. P.; Mitchell, D.; and MacGregor, J. G., “Structural Design Considerations for High-Strength Concrete,” *Concrete International: Design and Construction*, 15(5), 27-34.

30. Razvi, S. R., and Saatcioglu, M. “Strength and Deformability of Confined High-Strength Concrete Columns.” ACI *Structural Journal*, Vol. 91, 1994, pp. 678-687.

31. Sheikh, S. A., and Uzumeri, S. M., “Strength and Ductility of Tied Concrete Columns,” *Journal of the Structural Division*, ASCE, Vol. 106, No. ST5, May 1980, pp. 1079-1102.

32. Sugano, S.; Nagashima, T.; Kimura, H.; Tamura, A.; and Ichikawa, A., “Experimental Studies on Seismic Behavior of Reinforced Concrete Members of High-Strength Concrete,” *High-Strength Concrete*, Second International Symposium, SP-121-5, ACI International, 1990, pp. 61-87.

33. Saatcioglu, M., and Razvi, S., “Strength and Ductility of Confined Concrete,” ASCE *Journal of Structural Engineering*, Vol. 118, No. 6, June, 1992.

34. Sheikh, S. A., and Uzumeri, S. M., “Analytical Model for Concrete Confinement in Tied Columns,” *Journal of the Structural Division*, ASCE, V. 108, ST12, December 1982, pp. 2703-2722.

35. Watanabe, F.; Muguruma, H.; Matsutani, T.; and Sanda, D., “Utilization of High-Strength Concrete for Reinforced Concrete High-Rise Buildings in Seismic Areas,” *Proceedings* of Symposium on Utilization of High-Strength Concrete, Stavanger, Norway, June 1987.

36. Muguruma, H., and Watanabe, F., “Ductility Improvement of High-Strength Concrete Columns with Lateral Confinement,” *Proceedings* of Second International Symposium on Utilization of High-Strength Concrete, Berkeley, May 20-23, 1990.

37. Kabeyasawa, T.; Li, K. N.; and Huang, K., “Experimental Study on Strength and Deformability of Ultra High-Strength Concrete Columns,” *Transactions* of the Japan Concrete Institute, Vol. 12, 1990, pp. 315-322.

38. Sakai, Y.; Hibi, J.; Otani, S.; and Aoyama, H., “Experimental Study of Flexural Behavior of Reinforced Concrete Columns Using High-Strength Concrete,” *Transactions* of the Japan Concrete Institute, Vol. 12, 1990, pp. 323-330.

39. Muguruma, H., and Watanabe, F., “Ductile Behavior of High-Strength Concrete Columns Confined by High-Strength Transverse Reinforcement,” SP 128-54, ACI International, 1991.

40. Bing, Li; Park, R.; and Tanaka, H., “Effect of Confinement on the Behavior of High-Strength Concrete Columns under Seismic Loading,” Pacific Conference on Earthquake Engineering, New Zealand, Nov. 1991.

41. Thomsen, J. H., and Wallace, J. W., “A Study of High-Strength Reinforced Concrete Columns Subjected to Lateral and Axial Loads,” Open Paper Session, ACI Spring Convention, 1992.

42. Azizinamini, A.; Kuska, S.; Brungardt, P.; and Hatfield, E., “Seismic Behavior of Square High-Strength Concrete Columns,” ACI *Structural Journal*, Vol. 91, No. 3, May-June 1994, pp. 336-345.

43. Lloyd, N. A., and Rangan, B. V., “High-Strength Concrete Columns under Eccentric Compression,” *Research Report* No. 1/95, School of Civil Engineering, Curtin University of Technology, Perth, Western Australia, January 1995.

44. Mattock, A. H.; Kriz, L. B.; and Hognestad, E., “Rectangular Concrete Stress Distribution in Ultimate Strength Design,” ACI JOURNAL, *Proceedings*, V. 57, No. 8, Feb. 1961, pp. 875-928.

45. Hognestad, E.; Hanson, N. W.; and McHenry, D., “Concrete Stress Distribution in Ultimate Strength Design,” ACI JOURNAL, *Proceedings*, V. 52, No. 6, Dec. 1955, pp. 455-479.

46. Nedderman, H., “Flexural Stress Distribution in Very-High-Strength Concrete,” MSc Thesis, University of Texas at Arlington, December 1973, 182 pp.

47. Kaar, P. H.; Hanson, N. W.; and Capell, H. T., “Stress-Strain Characteristics of High-Strength Concrete,” Douglas McHenry International Symposium on Concrete and Concrete Structures, SP-55, American Concrete Institute, Detroit, 1978, pp. 161-185.

48. Swartz, S. E.; Nikaeen, A.; Narayan Babu, H. D.; Periyakaruppan, N.; and Refai, T. M. E., “Structural Bending Properties of High-Strength Concrete,” SP-87, American Concrete Institute, Detroit, 1985, pp. 147-178.

49. Ibrahim, Hisham, and MacGregor, James G., “Flexural Behavior of High-Strength Concrete Columns,” *Structural Engineering Report* No. 196, University of Alberta, Edmonton, Alberta, March, 1994.

50. Shin, S. W.; Ghosh, S. K.; and Moreno, J., “Flexural Ductility of Ultra-High-Strength Concrete Members,” ACI *Structural Journal*, July-Aug. 1989.

51. Shin, S. W., “Flexural Behavior Including Ductility of Ultra-High-Strength Concrete Members,” Ph.D. Dissertation, Department of Civil Engineering, University of Illinois at Chicago, 1986.

52. Canadian Standards Association, “A23.3-94 Design of Concrete Structures,” Rexdale, Ontario, December 1994, 199 pp.

53. Sheikh, S. A., “Deformability of High-Strength Concrete Columns,” *Proceedings* of Third International Symposium on Utilization of High-Strength Concrete, Lillehammer, Norway, June 1993, pp. 346-353.

54. “Reducing Earthquake Hazards: Lessons Learned from Earthquakes,” *Publication* No. 68-02, Earthquake Engineering Research Institute, El Cerrito, CA, Nov. 1986.

55. Rosenblueth, E., “The Mexico Earthquake: A First Hand Report,” *Civil Engineering*, ASCE, 56(1), Jan. 1986.

56. Kreger, M. E., and Sozen, M. A., “Seismic Response of Imperial County Services Building in 1979,” *Journal of Structural Engineering*, ASCE, 115(12), Dec. 1989.

57. Ghosh, S. K., and Corley, W. G., “Behavior of Reinforced Concrete Framing Systems,” *Proceedings* of International Conference on the Mexico Earthquake—1985, ASCE, Sept. 1986.

58. “Code of Practice for the Design of Concrete Structures, NZS 3101" Standards Association of New Zealand, Wellington, 1982 (Parts 1 and 2).

59. Uniform Building Code, International Conference of Building Officials, Whittier, CA, 1988.

60. “Tentative Provision for the Development of Seismic Regulations for Buildings,” *Publication* No. ACT-3-06, Applied Technology Council, Palo Alto, CA, June 1978.

61. “Recommended Lateral Force Requirements and Commentary,” Seismology Committee, Structural Engineers Association of California, San Francisco, CA, 1980.

62. “CAN3-A23.3-M84 Design of Concrete Structures for Buildings,” Canadian Standards Association, Rexdale, Ontario, 1984.

63. “Building Code Requirements for Reinforced Concrete (ACI 318-89),” ACI International, Detroit, 1989.

CHAPTER 6—NOTATION

A_g = gross area of column cross-section

A_{st} = cross-sectional area of longitudinal reinforcing steel

f'_c = concrete compressive strength

f_y = yield strength of non-prestressed steel

f_{yh} = yield strength of hoops

f_{yt} = yield strength of transverse reinforcing steel

P = axial load

P_o = pure axial load capacity of columns calculated according to nominal strength equations of ACI 318-89

α_1 = compression block stress intensity factor

β_1 = ratio of depth of compression stress block to depth of neutral axis

Δ = axial displacement

ε_{01} = axial strain corresponding to peak stress of plain concrete

ρ_s = ratio of volume of hoops to total volume of core concrete (out-to-out of hoops)

This report was submitted to letter ballot of the committee and was approved in accordance with Institute procedures.