



INCORPORATING HIGH-STRENGTH CONCRETE IN SEISMIC PROVISIONS OF THE CANADIAN CONCRETE STANDARD

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SUMMARY

In the 1994 Canadian Standards Association Standard CSA A23.3, a conservative upper limit on the concrete compressive strength of 55 MPa was chosen for the seismic design and detailing of ductile concrete structures. This paper presents the results of experimental programs carried out at the University of Sherbrooke and McGill University to provide experimental evidence and to develop design equations and detailing requirements for HSC structures. The experimental research programs included full-scale testing of (i) columns subjected to pure axial load, (ii) columns subjected to flexure and axial load, (iii) beam-column sub-assemblages, coupled-walls systems, and (iv) a full size frame building. Performance-based design equations are presented for HSC columns and are validated with the experimental results. In light of the research programs carried out, the upper limit on concrete compressive strength was increased from 55 to 80 MPa, with appropriate changes to the design and detailing provisions.

INTRODUCTION

This paper presents the results of experimental programs carried out at the University of Sherbrooke and McGill University to provide experimental evidence and to develop design equations and detailing requirements for HSC frame structures. The experimental research programs included full-scale testing of (i) columns subjected to pure axial load, (ii) columns subjected to flexure and axial load, (iii) beam-column sub-assemblages. Performance-based design equations are presented for HSC columns and are validated with the experimental results.

TREATMENT OF HIGH-STRENGTH CONCRETE IN SEISMIC DESIGN CODES

Seismic design provisions for nominally ductile and ductile structures in the 1994 CSA Standard A23.3 [1] limit the use of the design equations to concrete compressive strengths up to 55 MPa. This conservative limit was chosen because of concerns about the brittleness of high-strength concrete (HSC) in compression. The 2002 ACI Code [2] has no upper limit on concrete strength and the 1995 New Zealand Standard [3] limits the compressive strength to 70 MPa. Extensive experimental evidence, along with analytical studies, have improved the understanding of the behaviour of HSC elements subjected to

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seismic loading. This has enabled an increase in the upper limit of the concrete compressive strength from 55 MPa to 80 MPa for the seismic design of moderately ductile and ductile structural elements in the proposed 2004 CSA Standard [4].

EXPERIMENTS ON COLUMNS SUBJECTED TO PURE AXIAL LOAD

A series of tests was conducted to study the behaviour of HSC columns confined by rectangular ties. Over sixty columns (235×235×1400 mm) were tested under compressive concentric loading. Test variables studied in this research program are: the concrete compressive strength, the tie yield strength, the tie configuration, the lateral reinforcement ratio, the tie spacing, the longitudinal reinforcement ratio, the influence of concrete cover and the shape of the cross section.

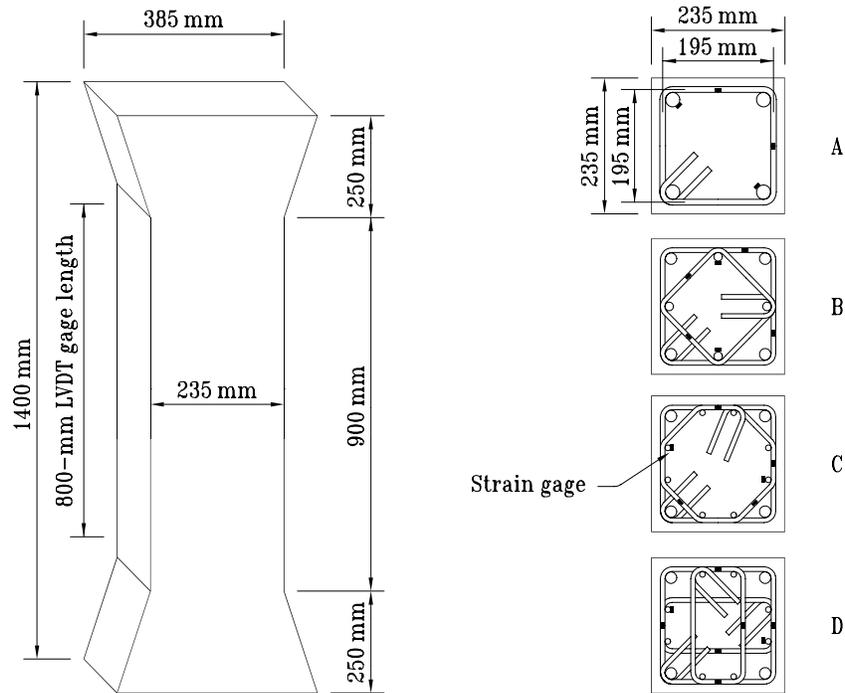


Fig. 1 Specimens tested under concentric compression [5].

Four different concrete mixes designed with specified 28-day strengths of 60, 80, 100, and 120 MPa were used with corresponding water-binder ratios of 0.50, 0.35, 0.25, and 0.24, respectively. Silica fume was used in all mixes to obtain high strength, workability, and reduction of fine particle segregation. The crushed stone had a maximum diameter of 10 mm. A concrete slump of 200 mm was used to ensure that the concrete could be placed through the dense reinforcement cages.

Deformed steel bars were used for the longitudinal reinforcement and plain steel bars for lateral reinforcement. Tension tests were performed on steel samples of each bar diameter and steel strength for each batch of steel bars. The average yield strength values were determined from at least three tension tests. The yield strength of the high-strength lateral reinforcement is defined at an offset strain of 0.2%. All ties were anchored with 135° bends extending 75 mm into the concrete core, which is longer than the minimal length of 6-bar diameters required by the ACI Code [2].

Electrical-resistance strain gauges were glued to the vertical bars and the transverse ties as shown in Fig. 1. A set of ties located at the mid-height of each specimen was instrumented with strain gauges placed on two adjacent sides of each tie of the chosen set, as shown in Fig. 1. The axial displacement of the specimens was recorded using four linear variable differential transformers (LVDTs) located at each corner of the specimens over a gauge length of 800 mm.

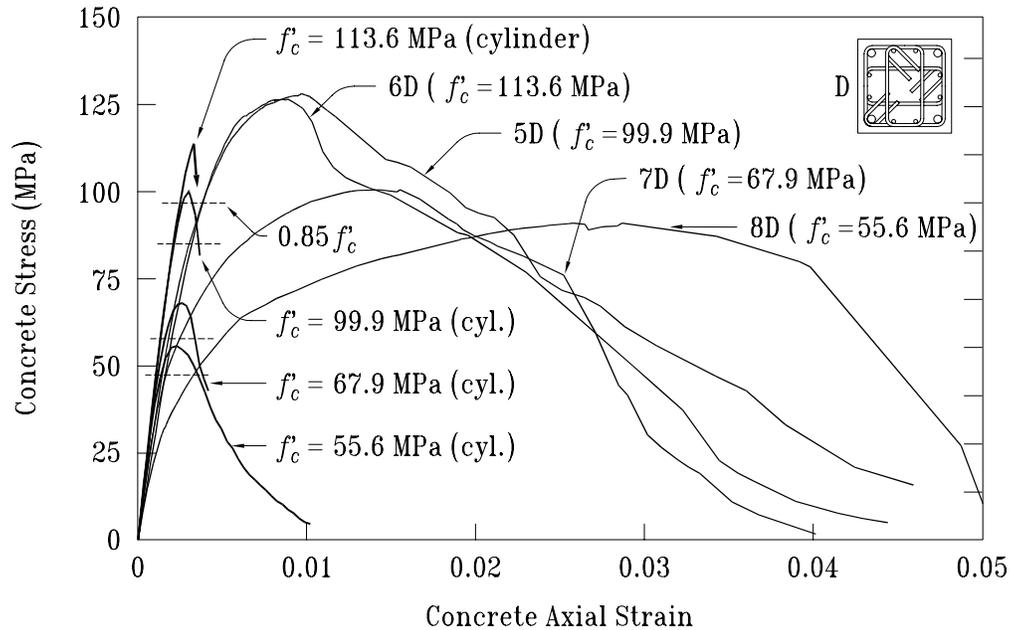


Fig. 2 Experimental stress-strain curves for different strength concretes [5].

High-strength concrete exhibits less lateral expansion under axial compression than normal-strength concrete due to its higher modulus of elasticity and its reduced microcracking. Consequently, the confining reinforcement comes into play later in the process and the efficiency of passive confinement of high-strength concrete is reduced. Figure 2 illustrates the stress-strain curves of the confined concrete for the equally-confined Specimens 6D, 5D, 7D, and 8D, along with the stress-strain curves of the corresponding plain concrete cylinders. Ratios of the confined concrete strength to the unconfined strength are 1.31, 1.51, 1.74, and 1.92 and the corresponding ratios for toughness (areas under stress-strain curves) are 8.54, 9.44, 9.91, and 15.67 for Specimens 6D, 5D, 7D, and 8D, respectively. The test results indicate that significant strength and toughness enhancements can be achieved when lateral reinforcement is provided. However, as evident from Fig. 2, greater strength and toughness gains are observed for specimens made with lower strength concrete.

EXPERIMENTS ON COLUMNS SUBJECTED TO FLEXURE AND AXIAL LOAD

Eighteen full-scale columns of dimensions 305×305×2150 mm were tested as shown in Fig. 3 in three different research programs. A varying transverse load is applied at the tip of the specimen, 2.00 m from the base of the column. Three parameters were investigated: concrete strength; lateral steel content and yield strength; and level of axial compression. Two research programs are described below [6, 7, 8].

In the first research program, the columns were designed to investigate the effects of two main parameters: (i) the level of axial load and (ii) the volumetric ratio of confinement steel. The level of axial load, defined as the applied axial compression divided by the column concrete axial load capacity, had values of 0.15,

0.25, and or 0.40. These ratios correspond to tension reinforcement yielding, balanced failure and crushing of the concrete, respectively. To investigate the influence of the volumetric ratio of confinement steel, the tie configuration was kept constant and the spacing of ties varied from 60 to 130 mm. A 130-mm tie spacing corresponds to a normal shear design controlled by the $d/2$ requirement. The 60-mm tie spacing represents full confinement required by the CSA Standard [1]. Test specimens are identified by concrete strength, tie configuration, tie spacing, and axial load level. Hence, C100B60N15 is a column with 100-MPa concrete with tie configuration B (see Fig. 1), spaced at 60 mm, and subjected to an axial load level ratio of 0.15.

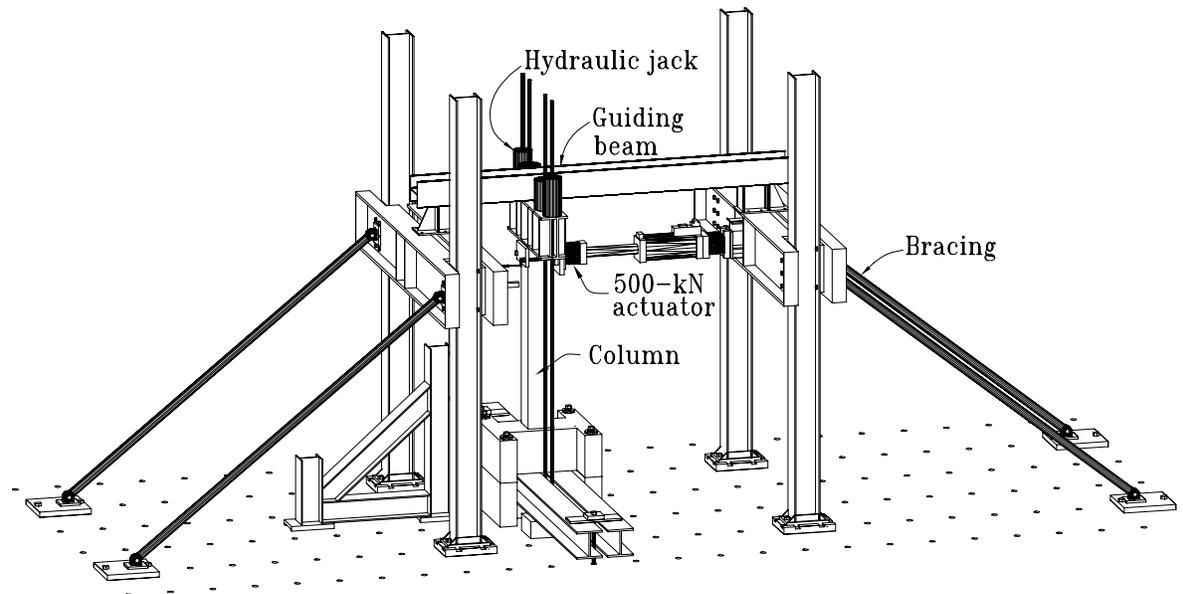


Fig. 3. Test setup for cyclic flexure and axial load tests [6].

The specified 100-MPa concrete resulted in actual average strengths ranging from 92.4 to 104.3 MPa. Three different sizes of Grade 400 reinforcing bars were used: No. 10, No. 15 and No. 20. Details of the transverse reinforcement are shown in Fig. 4. For each specimen, the longitudinal reinforcement consisted of four No. 15 and four No. 20 deformed bars, providing a reinforcement ratio of 2.15%. No. 10 hoops all had seismic hooks with 110 mm free end extensions.

Each specimen was heavily instrumented to determine the strains in the longitudinal and transverse reinforcement over the height of the column. The horizontal tip displacement was measured by a LVDT (see Fig. 3). Dywidag bars tensioned by hydraulic jacks were used to achieve the desired axial compression in the column. A 500-kN actuator with displacement and force control capabilities applied the horizontal load. During the test, the axial load was maintained constant by readjustment after each half cycle. The test ended when at least one of the three following events occurred: (i) the column was not able to sustain axial load, characterized by a 10% loss of axial load during a quarter of a cycle; (ii) flexural resistance dropped more than 50% of the maximum experienced capacity and (iii) a longitudinal bar ruptured, inducing a large drop of flexural capacity.

Figure 4 shows test results from a test series designed to study the influence of (i) the axial load level and (ii) the volumetric ratio of confinement reinforcement on the behaviour of HSC columns under axial compression and cyclic flexure.

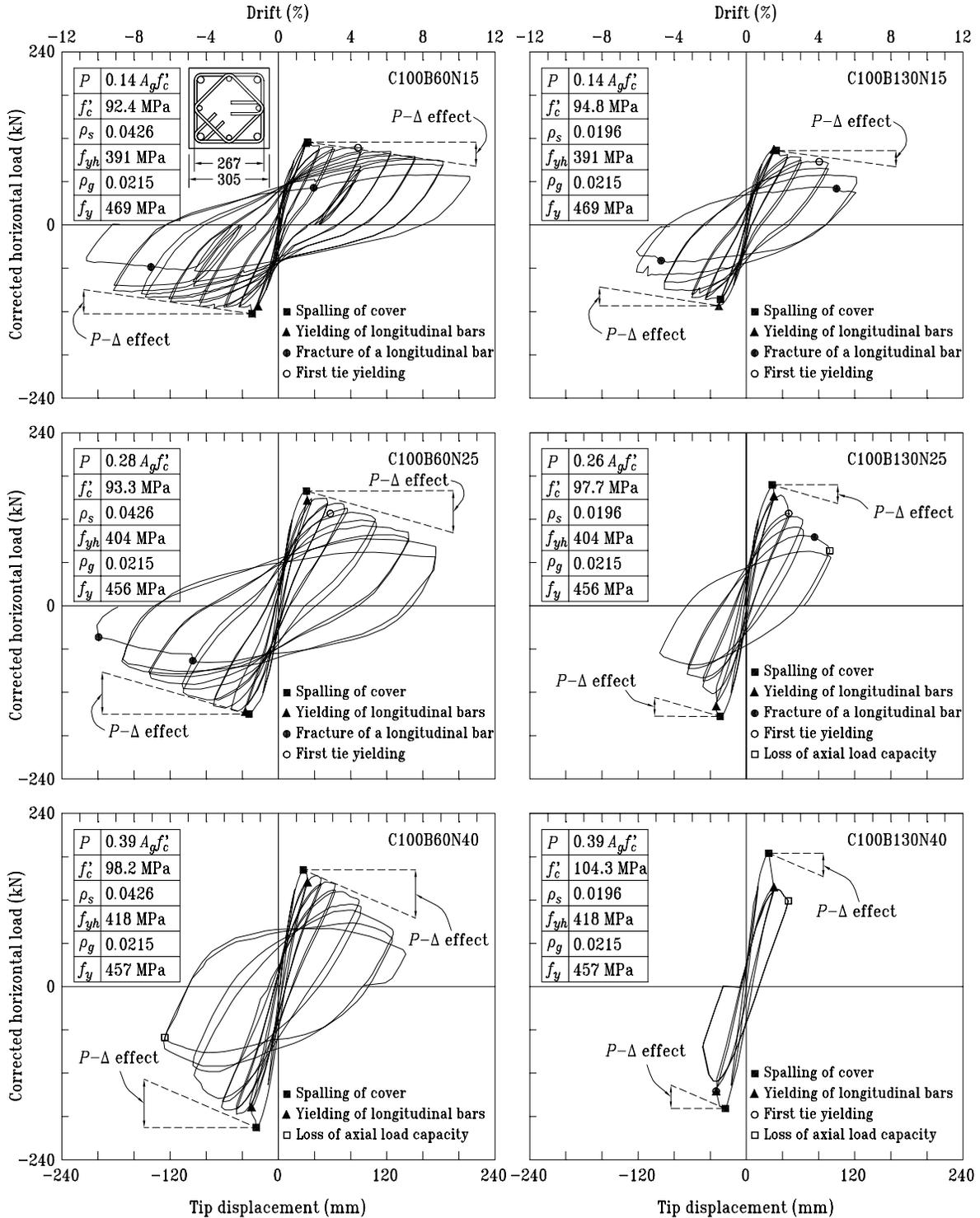


Fig. 4 Influence of axial load level and amount of transverse reinforcement in HSC columns subjected to flexure and axial load [6].

The influence of the axial load level is assessed with two sets of three columns having the same volumetric ratio of confinement steel. The first set of columns consists of specimens C100B60N15, C100B60N25, and C100B60N40. The three specimens had the same 4.26% volumetric ratio of transverse

steel and were made with concrete of comparable strength. They were subjected to axial load levels of 14%, 28%, and 39% respectively. In Fig. 4, all three specimens responses are given in the left column. The axial load level increases from the top of the figure to the bottom. The specimens with the lowest axial load level, C100B60N15, exhibited a very ductile behaviour as well as an excellent capacity to sustain large inelastic cyclic displacement. The displacement ductility drops from 8.8 to 8.2 to 4.7 when the axial load levels increase from 14 to 28 to 39%, respectively. As can be seen in Fig. 4, the axial load level has a beneficial effect on moment resisting capacity, but it has a negative influence on the inelastic behaviour of the column.

The second set is composed of specimens C100B130N15, C100B130N25, and C100B130N40. The three specimens have the same 1.96% volumetric ratio of transverse steel and were made with concrete of comparable strength. They have been subjected to axial load levels of 14%, 26% and 37%, respectively. The transverse ties were spaced at 130 mm. In Fig. 4, the responses of the three specimens are given in the right column. As for the previous three columns, this figure shows that the axial load level has a beneficial effect on moment resisting capacity, but it has a negative influence on the inelastic behaviour of the column.

This study together with the research carried out by Sheikh et al. [9] indicate that the ACI confinement reinforcement requirements are not conservative for high levels of axial load and are conservative and uneconomical in a great number of situations of practical interest in which columns support small axial compressions. For example, Specimen C100B130N15 exhibits acceptable ductile behaviour, even though it is made with HSC confined with less than 45% of the transverse reinforcement required by the ACI Code [2]. On the other hand, specimen C100B60N40, which complied with the ACI transverse reinforcement requirement, experienced similar behaviour. For HSC to be used economically, the confinement reinforcement must be related to the level of axial load. The influence of the volumetric ratio of confinement steel is assessed in Fig. 4 for three pairs of columns. The first set comprises C100B60N15 and C100B130N15. The three columns on the left hand side contain 4.26% of transverse reinforcement while those on the right contain 1.96%. It is evident that as the volumetric ratio increases the displacement ductility increases. For example, Specimen C100B60N25 has a displacement ductility and an ultimate drift ratio about four times higher than with C100B130N25. For this research program, a volumetric ratio of confinement steel of about 2% seems acceptable to reach ductile behaviour under an axial load less or equal to 15% of axial load capacity. For an axial load level of 40%, a volumetric ratio of confinement steel of about 4% seems adequate. It is noted that the 1995 New Zealand Standard [3] includes the influence of axial load on the required amount of confinement reinforcement.

PERFORMANCE BASED MODEL FOR CONFINEMENT

Cusson and Paultre have shown that confinement efficiency can be measured with the effective confinement index, I_e , defined as

$$I_e = \frac{f'_{\ell e}}{f'_c} = K_e \frac{A_{sh} f'_h}{c s f'_c} \quad (1)$$

where $f'_{\ell e}$ is the effective confinement stress at peak strength, f'_c is the compressive concrete strength, A_{sh} is the total cross-sectional area of the confinement reinforcement in the direction under consideration, f'_h is the stress in the confinement reinforcement at peak concrete strength, c is the confined core dimension and s is the tie spacing, K_e is the confinement efficiency defined as [10, 11]:

$$K_e = \frac{\left(1 - \frac{\sum w_i^2}{6c_x c_y}\right) \left(1 - \frac{s'}{2c_x}\right) \left(1 - \frac{s'}{2c_y}\right)}{(1 - \rho_{cc})} \quad (2)$$

where w_i is the clear horizontal spacing between two adjacent longitudinal bars, c_x and c_y are the dimensions of the confined core measured centre-to-centre of the outer hoop, s' is the clear spacing between ties and ρ_{cc} is the volumetric ratio of longitudinal steel.

Paultre et al. [12] have proposed an equation relating the effective confinement index to the ductility level, μ_ϕ , that a column is required to reach, as given by :

$$I_e = 0.01 \frac{P}{P_0} \mu_\phi \quad (3)$$

It is possible to calculate the required amount of transverse reinforcement for a given target ductility level by equating Equations. (1) and (3). This method was used for the design of the six columns tested by Robles [7]. All columns tested in this research program were circular and made of concrete with a target strength of 100 MPa. Test specimens in this series are identified by concrete strength (C100), spiral transverse reinforcement Grade 400 (S) or Grade 500 (SH), spiral pitch (37, 70 or 100 mm) and the axial load level (N15). Hence, C100S37N15 represents a column made of 100 MPa concrete with spiral transverse reinforcement having a pitch of 37 mm, and subjected to a target axial load level of 15% of the concrete axial load capacity. As can be seen from Fig. 5, all specimens reached about the same curvature ductility under increasing axial load levels. Thus it is possible to choose an appropriate amount of transverse reinforcement to attain the desired curvature ductility while accounting for level of axial load.

PROPOSED CONFINEMENT REQUIREMENTS FOR THE 2004 CSA STANDARD

In the proposed 2004 CSA Standard [4], ductile frame structures are subdivided into “ductile” and “moderately ductile” seismic force resisting systems with ductility-related force modification factors, R_d , of 4.0 and 2.5, respectively. The equations for the required amounts of confinement reinforcement were derived based on the effective confinement index of Equation 3. For the derivation of the equations, it was assumed that the required curvature ductility, μ_ϕ , is equal to 4 times R_d . Hence, the required curvature ductilities are 16 and 10 for columns in ductile moment-resisting frames and columns in moderately ductile frames, respectively. The resulting design equations for confinement for the proposed CSA Standard are given below:

Ductile Columns

The transverse reinforcement for columns in ductile moment-resisting frames shall be provided as follows:

- (a) the volumetric ratio of circular hoop reinforcement, ρ_s , shall not be less than the larger of the amounts required by Equations (4) and (5):

$$\rho_s = 0.4k_p \frac{f_c'}{f_{yh}} \quad (4)$$

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f_c'}{f_y} \quad (5)$$

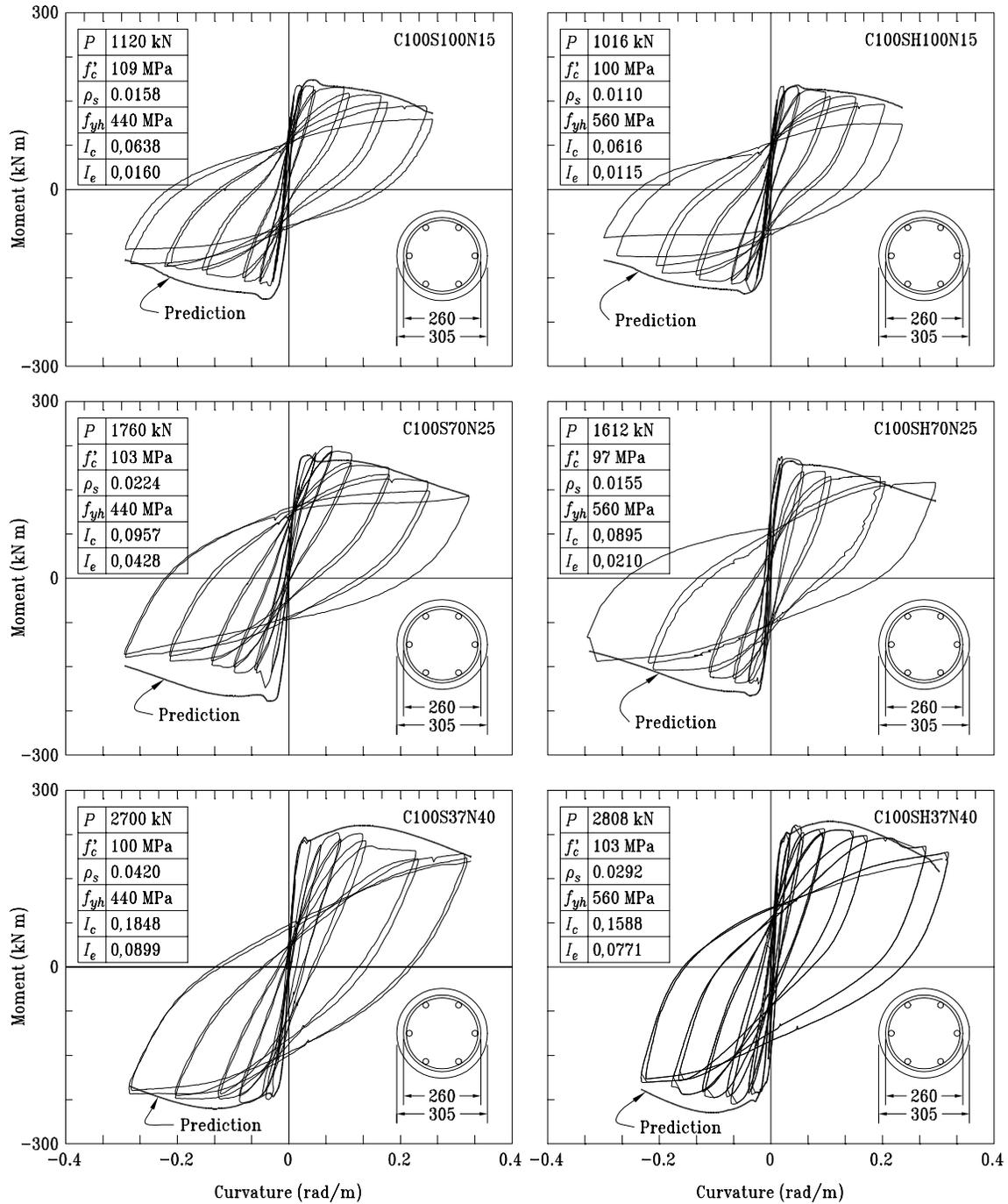


Fig. 5 Influence of amount of transverse reinforcement and axial load levels in HSC columns subjected to flexure and axial load [7].

where $k_p = P_f / P_0$, P_f is the maximum factored axial load for earthquake loading cases, P_0 is the nominal axial load resistance at zero eccentricity, f'_c is the specified compressive strength of concrete and f_{yh} is the specified yield strength of transverse reinforcement and shall not be taken as greater than 500 MPa.

- (b) the total effective area, A_{sh} , in each of the principal directions of the cross section within spacings of rectangular hoop reinforcement shall be not less than the larger of the amounts required by Equations (6) and (7)

$$A_{sh} = 0.2k_n k_p \frac{A_g}{A_{ch}} \frac{f_c'}{f_{yh}} sh_c \quad (6)$$

$$A_{sh} = 0.09 \frac{f_c'}{f_{yh}} sh_c \quad (7)$$

where $k_n = n_\ell / (n_\ell - 2)$, n_ℓ is the total number of longitudinal bars in the column cross section that are laterally supported by the corner of hoops or by hooks of seismic crossties, $k_p = P_f / P_0$ and f_{yh} shall not be taken as greater than 500 MPa.

In addition to the confinement requirements given above, the transverse reinforcement must be adequate to avoid shear failures and must satisfy the detailing requirements given below:

Transverse reinforcement shall be spaced at distances not exceeding:

- (a) one-quarter of the minimum member dimension;
- (b) 6 times the diameter of the smallest longitudinal bar; or
- (c) s_x as defined by Equation (8)

$$s_x = 100 + \left(\frac{350 - h_x}{3} \right) \quad (8)$$

where h_x is the maximum horizontal spacing of hoop or crosstie legs on all faces of the column.

In the direction perpendicular to the longitudinal axis of the member, the distance h_x shall not exceed the greater of 200 mm or one-third of the core dimension in that direction, but in any case not more than 350 mm.

The confinement reinforcement shall be provided over a length, ℓ_o , from the face of each joint and on both sides of any section where flexural yielding may occur as a result of inelastic lateral displacement of the frame. The length, ℓ_o , shall be determined as follows:

- (a) where $P_f \leq 0.5\phi_c f_c' A_g$, ℓ_o shall be not less than either 1.5 times the largest member cross section dimension or one-sixth of the clear span of the member (ϕ_c is the resistance factor for concrete, equal to 0.65).
- (b) where $P_f > 0.5\phi_c f_c' A_g$, ℓ_o shall be not less than either 2 times the largest member cross section dimension or one-sixth of the clear span of the member.

Columns, which due to their connection to rigid members such as foundations or discontinued walls or due to their position at the base of the structure may develop plastic hinges, shall be provided with transverse reinforcement as specified above over their clear height. This transverse reinforcement shall continue into the discontinued member for at least the development length of the largest longitudinal reinforcement in the column.

Moderately Ductile Columns

Transverse reinforcement for moderately ductile columns shall be detailed as hoops and cross ties or spirals. Columns, which due to their connection to rigid members such as foundations or discontinued walls or due to their position at the base of the structure may develop plastic hinges, shall be provided with transverse reinforcement over their clear height as specified below:

- (a) the volumetric ratio of circular hoop reinforcement, ρ_s , shall not be less than the larger of the amounts required by Equations (9) and (5):

$$\rho_s = 0.3k_p \frac{f'_c}{f_{yh}} \quad (9)$$

where $k_p = P_f / P_0$ and f_{yh} shall not be taken as greater than 500 MPa.

- (b) the total effective area, A_{sh} , in each of the principal directions of the cross section within spacing s of rectangular hoop reinforcement shall be not less than the larger of the amounts required by Equations (10) and (7)

$$A_{sh} = 0.15k_n k_p \frac{A_g}{A_{ch}} \frac{f'_c}{f_{yh}} s h_c \quad (10)$$

where $k_n = n_\ell / (n_\ell - 2)$, $k_p = P_f / P_0$ and f_{yh} shall not be taken as greater than 500 MPa.

In addition to the confinement requirements given above, the transverse reinforcement must be adequate to avoid shear failures and must satisfy the detailing requirements given below:

Transverse reinforcement shall be provided at both ends of the columns over a length equal to the largest of one-sixth of the clear height, the maximum cross-sectional dimension or 450 mm, with a spacing not exceeding :

- (a) 8 longitudinal bar diameters;
- (b) 24 tie diameters; or
- (c) one-half of the minimum column dimension.

In the direction perpendicular to the longitudinal axis of the column, cross tie or legs of overlapping hoops shall have centre-to-centre spacings not exceeding 350 mm.

EXPERIMENTS ON BEAM-SLAB-COLUMN SUB-ASSEMBLAGES

Figures 6 and 7 show the geometry and reinforcement details for two beam-slab-column sub-assemblages that were tested under reversed cyclic loading. These sub-assemblages were designed and detailed in conformance with the seismic provisions of CSA A23.3-94 [1] for ductile moment-resisting frames ($R = 4.0$). The columns contain adequate confinement reinforcement and are stronger than the beams. The beams have closely spaced hoop reinforcement to avoid shear failures, to provide sufficient confinement and to prevent buckling of the longitudinal bars. The joints were also adequately confined and designed to resist the shear resulting from beam hinging. Both specimens were designed for the same flexural capacity in the beams. Specimen R4 was designed and detailed for a concrete compressive strength of 30 MPa, resulting in a 400 mm wide by 600 mm deep beam section and a 450×450 mm column. The No. 10 ($f_y = 480$ MPa) column hoops, consisted of a square and diamond-shaped hoop and were spaced at 80 mm as shown in Fig. 6. Specimen R4H was designed and detailed for a concrete compressive strength

of 70 MPa, resulting in a 350 mm wide by 600 mm deep beam section and a 350×350 mm column. The No. 10 (higher strength $f_y = 648$ MPa) column hoops consisted of a peripheral square hoop, together with two rectangular hoops at 65 mm as shown in Fig. 7. The specimens had a 100 mm thick slab that was 1.9 m wide. Side beams protruding from the columns had dimensions of 400×600 mm, 350×600 mm for Specimens R4 and R4H, respectively.

The specimens were tested with a constant column axial compressive load of 1076 kN. Reversed cyclic, that is downward (positive) and upward (negative) loading was applied near the end of the beam at a distance of 2000 mm from the centre of the column. The column was pinned at the top and the bottom to represent the points of contraflexure in the prototype structure. The actual average concrete strengths, at the time of testing, were 38.3 and 83.75 MPa for Specimens R4 and R4H, respectively. It is noted that the higher strength concrete has resulted in slightly smaller beams, a significantly smaller column and a larger amount of confinement reinforcement in the columns.

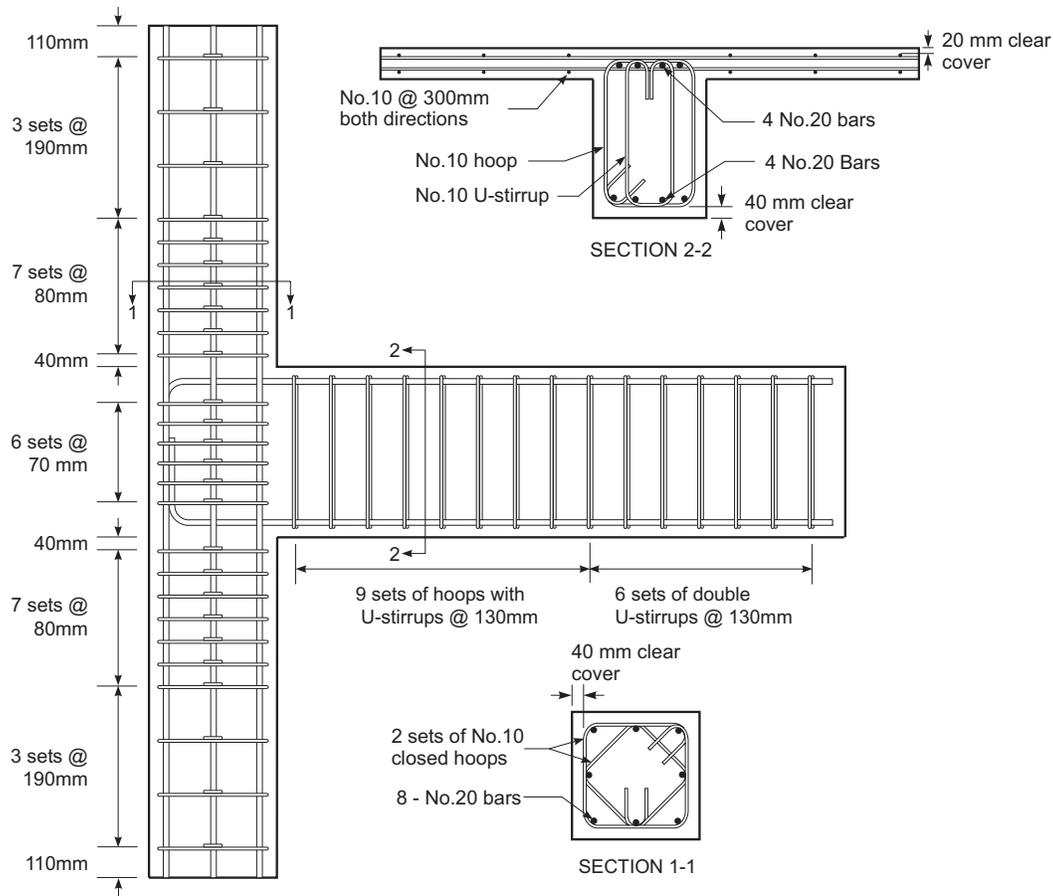


Fig. 6 Ductile beam-slab-column sub-assembly R4 designed with 70 MPa concrete [13].

Figure 8 compares the load applied near the end of the beam versus the beam tip deflection responses for the two sub-assemblages. Normal-strength concrete Specimen R4 attained displacement ductilities greater than 10 under positive loading and exceeding 8 under negative loading. These ductilities were limited during testing due to the displacement limitations of the loading apparatus. Specimen R4H reached a displacement ductility of 7 in the positive direction and 10 in the negative loading direction. Both specimens exhibited significant flexural hinging in the beams. The displacement ductility for Specimen

R4H was limited due to the rupture of the slab bars. These slab bars were cold-rolled No. 10 bars with limited ductility and strain hardening. All other reinforcement, except for the column ties, was weldable grade, hot-rolled reinforcement, as required by the CSA Standard [1]. This test demonstrates the fact that this more brittle reinforcement should not be used for the flexural reinforcement of ductile elements.

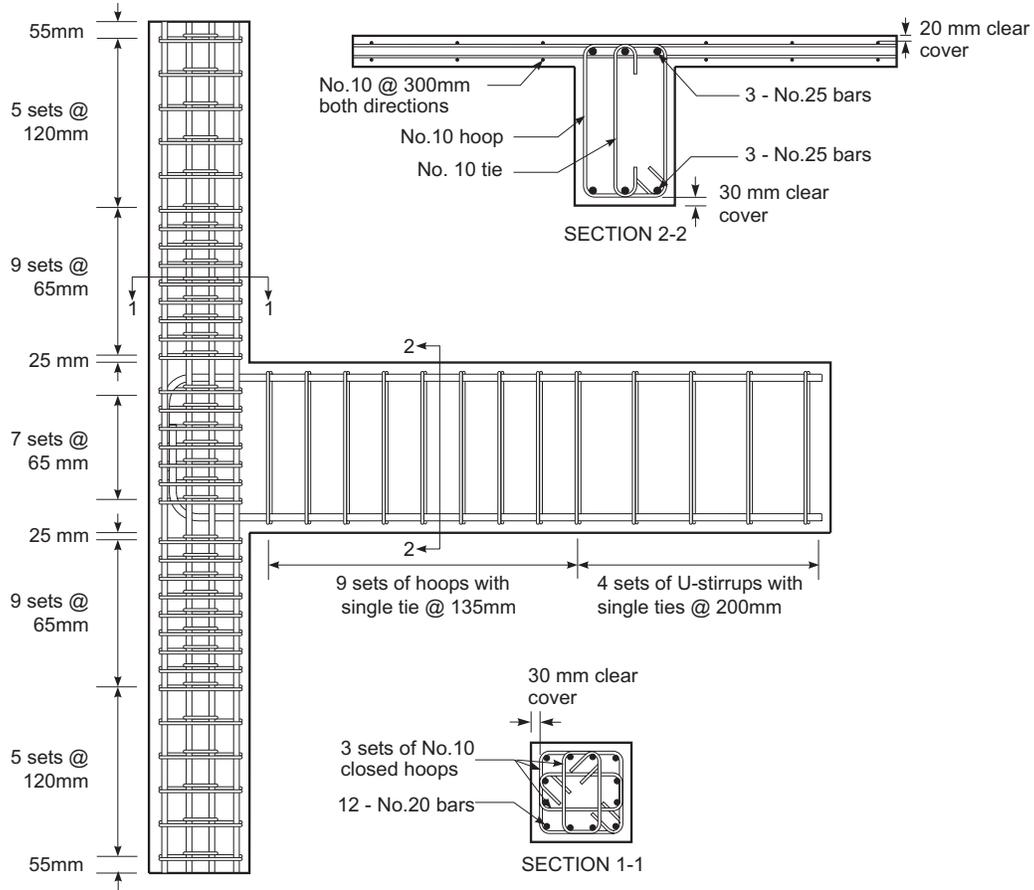


Fig. 7 Ductile beam-slab-column sub-assembly R4H designed with 30 MPa concrete [14].

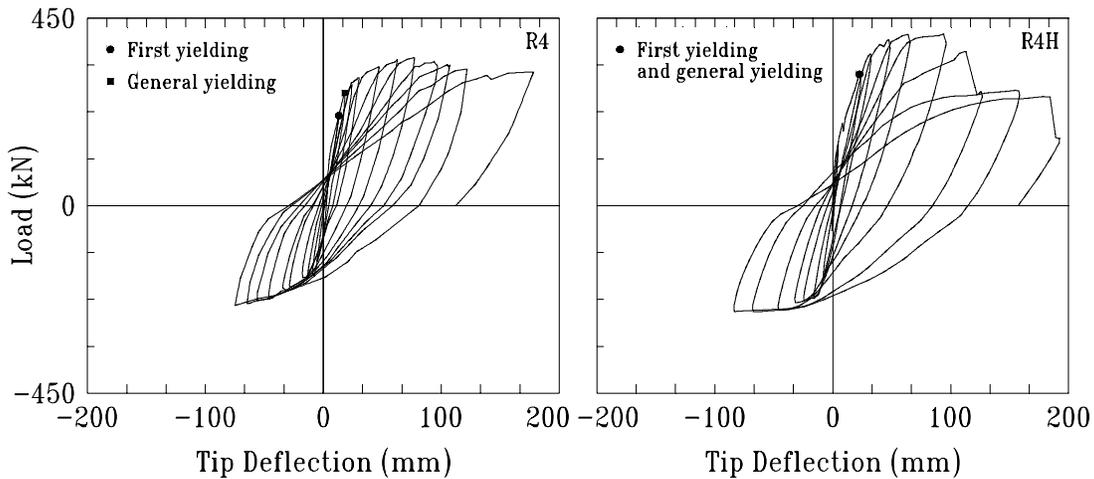


Fig. 8 Load-deflection responses of Specimens R4 and R4H [13, 14].

CONCLUSIONS

This paper highlights some of the important features concerning the seismic behaviour of HSC frame structures. The need to account for axial load level in determining the confinement reinforcement is emphasized. Proposed code equations for the confinement of ductile and moderately ductile columns were derived based on required ductility levels. The excellent behaviour of properly designed and detailed sub-assemblages demonstrates that HSC can exhibit the necessary ductile response. The experimental programs and analytical studies have enabled the upper limit on concrete compressive strength to be raised from 55 MPa to 80 MPa for ductile and moderately ductile frame structures.

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