Strength and Ductility of High-Strength Concrete Columns Confined with High-Strength Steel Ties

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Abstract: High-strength concrete (HSC) offers advantages in terms of performance and economy of construction. The use of HSC has become indispensable in high-rise building construction, particularly in columns. However, HSC columns are more brittle and have much less post-peak deformability than normal-strength concrete columns. The ductility of reinforced concrete columns can be increased by increasing volumetric ratio of confining steel; nonetheless, higher volumetric ratios of tie reinforcing bars are required for HSC columns to achieve a ductility level similar to that of normal-strength concrete columns. The Australian Standard for Concrete Structures AS3600 deals with the ductility issues of HSC columns by increasing the confining pressure applied to the core that is normally achieved by increasing the volumetric ratio and/or arrangement of tie reinforcement. It is expected that the use of high-strength steel (HSS) ties in columns will provide the high level of confinement required to enhance the ductility and strength of the concrete within the core and will offer a reduction in steel congestion, improved concrete placement, savings in the cost of labour and reduction in construction time. This paper reports the results of an experimental study on the strength and ductility performance of HSC columns confined with steel ties with a yield strength of 750 MPa, or more. The adequacy of AS3600 provisions to determine the confinement provided to core with HSS ties is investigated and design recommendations are made.

Keywords: Column, high-strength concrete, high-strength steel, confinement, ductility.

1. Introduction

Over the past few decades there has been a significant enhancement in the strength of concrete. The use of high-strength concrete (HSC) is paramount in high-rise building construction, especially in columns and core walls, to ensure sufficient strength in the structure. HSC outperforms conventional normal strength concrete in terms of strength, durability, and economy of construction; however, the brittle behaviour of this material is a significant weakness. As strength and ductility of concrete are inversely proportional, HSC is significantly more brittle than normal strength concrete (Saatcioglu, M. and Razvi, S. R., "High-strength concrete columns with square sections under concentric compression," Journal of Structural Engineering, 124(12), 1998, pp. 1438-1447.).

Research has identified two main problems in the use of HSC in columns: early spalling of the concrete cover and ductility. Spalling of the cover concrete is a concern for HSC columns with high axial loads applied at small eccentricities and this spalling occurs at loads lower than the theoretical squash load (Foster, S.J., "On behaviour of high-strength concrete columns: cover spalling, steel fibers, and ductility", ACI Structural Journal, 98(4), 2001, pp. 583-589.). The strength of HSC columns is affected by spalling of the cover due to the inability of the concrete core to carry increased loads after the cover is shed (Foster, S.J., "On behaviour of high-strength concrete columns: cover spalling, steel fibers, and ductility", ACI Structural Journal, 98(4), 2001, pp. 583-589.,Foster, S.J., Liu J. et al., "Cover spalling in HSC columns loaded in concentric compression", Journal of Structural Engineering, 124(12), 1998, pp. 1431-1437.). However, a somewhat less brittle behaviour in columns can be achieved through good confinement detailing of the core concrete; a higher level of confinement is needed in HSC columns to maintain a similar level of ductility to that of normal strength concrete (1,3-6). This requirement may be met by either increasing the volumetric ratio of confinement steel or using higher grades of reinforcement for the ties.

As construction and material costs escalate, demand has increased for stronger materials that occupy less space, provided the increased initial cost of materials is offset by the more significant economic benefit of increased rental space (7). With the advancement of high-performance (HP) construction materials, the demand for HP and high-strength steel (HSS) reinforcing bars has been increasing over the past few years, to combat unnecessary repair costs. By modifying the microstructure of the steel, bars may be produced that are less susceptible to corrosion compared with conventional steel and have a yield strength that can be twice that of conventional steel manufactured in the USA (8). The stress-strain characteristics of this reinforcement are quite different from conventional steel reinforcement and lack a well-defined yield point. One such steel, known as ChromX, conforming to ASTM A1035 (9), has been developed in the USA (10). Several demonstration projects including bridge decks, airport control towers, bridge piers, and high-rise condominiums have been constructed successfully using HP steel (8,11-13). Recently, many state transportation departments in the USA have begun to use HP steel as a direct replacement for conventional Grade 60 steel in concrete bridge decks. A summary of recent research outcomes on HP steel can be found in Shahrooz (14). With the increasing interest globally in HP and HSS reinforcement, an Australian steel manufacturer has been developing technology to manufacture a 750 MPa grade reinforcing steel.

HSS can provide various benefits to the construction industry by reducing member cross-sectional sizes and reinforcement quantities. In addition to that it potentially offers several other practical advantages; such as, reduction of congestion in heavily reinforced sections, improved concrete placement, savings in the cost of labour, reduction in construction time, and in some cases, enhanced resistance to corrosion (8,15-16).

Researchers have identified a potential use of HSS as tie reinforcement. There is experimental evidence that columns with the same reinforcement arrangement exhibit similar deformability, irrespective of concrete strength, provided that the ratio of $k_e \rho_s f_{yt}/f_c^{\prime}$, known as confinement parameter, is maintained and certain minimum limits are met for the volumetric ratio and spacing of ties (5-6,17-18), where k_e is an effectiveness factor, ρ_s is the volumetric ratio of the tie steel, f_{yt} is the yield strength of tie steel and f_c' is the characteristics compressive strength of concrete. The HSS, maintaining the same ratio of $k_e \rho_s f_{yt}/f_c^{\prime}$ to that of conventional steel, shall result larger tie spacing. A finite element analysis conducted by Foster et al. (3) investigating the effect of tie yield strength on the axial capacity and ductility of HSC columns revealed that while the increasing strength of the tie reinforcement from 400 MPa to 700, and 1000 MPa did not improve the column's strength, it enhanced the I_{10} ductility measure from 5.8 to 7.7, and 8.4, respectively. Bing et al. (19) found analytically that an increase in yield strength increases confining strength in column. Hence, it is expected that use of HSS ties in columns will provide the high level of confinement required to enhance the ductility and strength of the concrete within the core.

The requirements of confinement steel are outlined in current building standards, such as AS3600-2009 (20), Concrete Structures. In most experimental studies on the behaviour of HSC column, the experimental set-up for tie reinforcement detailing has been designed independently of the requirements of design standards. Therefore, there is no experimental confirmation of the rules which necessitates confirmation using analytical models (21). To design HSC column with HSS tie reinforcement as per the provisions of AS3600-2009 (20), it is important to check the strength and ductility of such columns experimentally. In the absence of experimental data, the use of new materials by designers may raise lack of confidence. Hence, it is imperative to conduct experimental investigation on the standard-designed HSC columns confined with HSS ties and develop experimental data base.

This paper presents the experimental results of six 200x200 mm square and six 250x150 mm rectangular section columns using a 100 MPa concrete, with 2% longitudinal steel of 500 MPa grade, confined with nominal 750 MPa grade steel ties at the maximum spacing allowed by AS3600-2009 (20) and loaded at different eccentricities. The ultimate strength and ductility measures of the test results are presented.

2. Theoretical Background

2.1 Ultimate strength

The ultimate strength of a column section may be defined in terms of its axial force-bending moment interaction diagram tracing the locus of all combinations of the ultimate axial force N_u and bending moment M_u. AS3600-2009 (20) estimates the squash load (N_{uo}) of a reinforced concrete column assuming a uniform compressive stress of $\alpha_1 f_c'$ on concrete, where α_1 is given by:

$$0.72 \leq \alpha_1 = 1.0 - 0.003 f_c' \leq 0.85 \tag{1}$$

The coefficient α_1 takes in to account the reduced strength of in situ concrete ($f_{cm} = 0.9 f_c'$) and a factor for the spalling of the cover concrete. The squash load N_{uo} is defined as:

$$N_{uo} = \alpha_1 f_c'(A_g - A_s) + A_s f_{sy} \tag{2}$$

where A_g is the gross concrete area, f_{sy} is the yield stress of the longitudinal steel, A_s is the total area of longitudinal steel. The pure bending capacity M_{uo} can be calculated with reasonable accuracy using the rectangular stress block parameters defined by AS3600 (20). However, studies by Foster (2), Foster et al. (3), and Liu et al. (22) showed that the capacity of HSC columns is significantly affected by cover spalling. In some instances, columns failed at loads as low as 70 percent of unconfined compressive strength (23,24).

2.2 Confinement of the core for HSC columns

Robustness and ductility are important considerations when it comes to the detailing of all concrete members. However, due to its more brittle nature, extra lateral reinforcements are required for HSC columns. Ductility in columns is derived from confinement provided by the tie reinforcement to the core and is a function of the yield strength of the ties, the concrete strength, the volumetric ratio of the tie reinforcement, and the arrangement of the ties. At high overload the cover concrete spalls from the section, and as such only the core concrete is considered in the calculation for ductility, where the core is defined as the section bounded by the centre-line of the outermost tie.

To ensure sufficient ductility in HSC columns, AS3600-2009 (20) requires the lateral reinforcing bars to provide at least a minimum confinement level within identified special confinement regions. In these regions, the minimum effective confining pressure provided at the time of yielding of the ties is $0.01f_c'$ (i.e. 1% of the concrete compressive strength). Other detailing measures are also enforced; specifically, spacing of lateral reinforcing bars is not to exceed the lesser of $0.6D_c$ and 300 mm, where D_c is the smaller dimension of a rectangular column cross section. The effective confining pressure, $f_{r.eff}$, applied to the core of a HSC column is calculated as:

$$f_{r.eff} = k_e f_r \tag{3}$$

where k_e is an effectiveness factor accounting for the arrangement of the confining bars and f_r is the average confining pressure on the cross section taken at the level of the ties. For non-circular sections, f_r is the smallest of the confining pressure calculated for each of the major directions calculated as:

$$f_r = \frac{\sum_{i=1}^{m} A_{b.fit} f_{sy.f} sin\theta}{d_s s} \tag{4}$$

in which, $A_{b.fit}$ is the area of one leg of the ties, $f_{sy.f}$ is the yield stress of the lateral bars, limited to 800 MPa, θ is the angle between the tie leg and the confinement plane, m is the number of tie legs crossing the confinement plane, d_s is the overall dimension measured between the centre-lines of the outermost confining bars, and s is centre to centre spacing of the ties along the column. For rectangular shaped sections, the confinement effectiveness factor is defined by:

$$k_e = \left(1 - \frac{nw^2}{6A_c}\right)\left(1 - \frac{s}{2b_c}\right)\left(1 - \frac{s}{2d_c}\right) \tag{5}$$

where, A_c is the cross-sectional area of the core bounded by the centre-line of the outermost confining bars, n is the number of laterally restrained longitudinal bars, w is the average clear spacing between adjacent tied longitudinal bars, b_c is the core dimension measured between the centre-lines of the outermost confining bars and taken across the width of the section, and d_c is the core dimension measured between the centre-lines of the outermost tie and taken through the depth of the section.

As a simplification, the requirement to provide an effective confining pressure of $0.01f_c'$ is deemed to be satisfied if the spacing of the ties meets the following:

$$s \le \frac{15nA_{b.fit}f_{sy.f}}{f_c'\sqrt{A_c}} \tag{6}$$

where n is the number of laterally restrained longitudinal bars.

2.3 Ductility

Ductility is an important feature of any structural member as it guards against unforeseen overload and sudden collapse. The ductile behaviour of any structural member depends on how well the member maintains the load for a considerable deformation in the post-peak regime. While many definitions/measures for ductility exist, Australian Standards - AS3600 adopted I_{10} index for ductility, where I_{10} is calculated in a manner similar to that set out in ASTM C1018 (25) for the measurement of toughness (26).

The I_{10} index is based on the area under the load versus combined nominal strain (ξ) curve similar to Figure 1. The strain parameter $\xi = (\varepsilon_{av} + \kappa e)$ is the sum of the average axial strain and the product of the eccentricity (*e*) of the internal axial force and the curvature (*k*) of the section. The ductility index I_{10} is the ratio of the area OAEF to the area OAB in Figure 1, which can be expressed as:

$$I_{10} = \frac{Area\ OAEF}{Area\ OAB} \tag{7}$$

where *A* corresponds to the point ξ_y and *E* corresponds to point $5.5\xi_y$, where ξ_y is the yield strain defined by the 3/4 rule (Figure 1). By this definition a perfectly elastoplastic material gives I_{10}=10 and for a perfectly elastic-brittle material I_{10}=1.

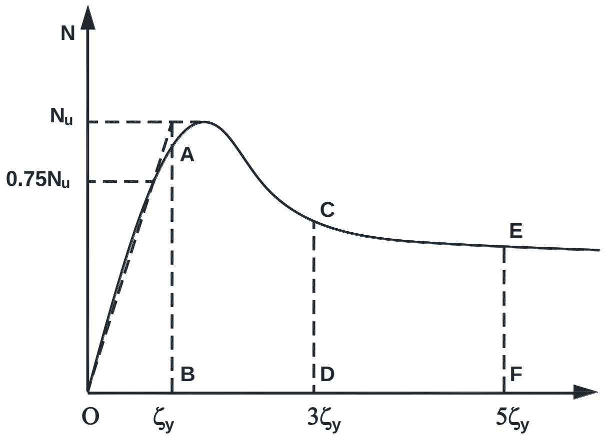


Figure 1. Ductility measurement in an eccentrically loaded column (17).

3. Experimental Program

3.1 Selection of variables and specimen details

The aims of the experimental program were: 1) to determine the strength and ductility of HSC columns confined with HSS ties; and 2) to study the adequacy of AS3600-2009 (20) requirements for confinement to the core in this regard. Total twelve concrete columns were prepared and grouped into two: Group A and B.

The main test variables for this study were initial load eccentricity, tie arrangements and column cross-section. Group A consisted of 200 x 200 mm square sections and Group B had 250 x 150 mm rectangular sections. The overall column lengths were 1800 mm for Group A and 2100 mm for Group B. All column ends were haunched to allow the eccentric loading and to control failure within the test zone. The middle 600 mm of the columns were prepared as test zone where ties were placed at required spacing.

The tie spacing was calculated following the provisions of AS3600-2009 (20) as discussed in Section 2.2. However, to compare the effect of confinement with the Standards maximum spacing to that of higher confinement, 3 of the specimens in Group B were provided with ties at half the spacing of that required by the Standard. To ensure failure in the test zone, the tie spacing was halved outside of the gauged zone.

The longitudinal reinforcement consisted of 8N12 bars and ties were provided by 5.5 mm bars. A clear cover of 15 mm was applied to the reinforcement design by placing plastic spacers on the outer ties. The test program is outlined in Table 1 and the details of column dimensions and reinforcement arrangements are given in Figure 2.

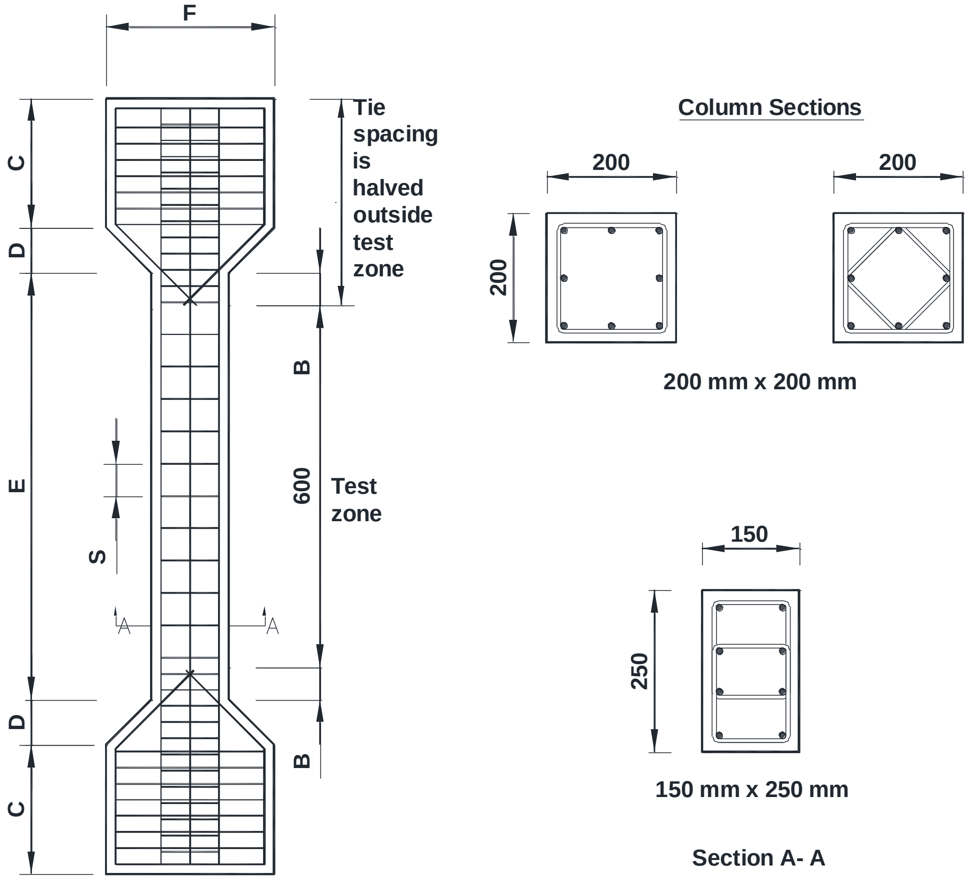
The columns were identified with a series of numbers and letters that represent the longitudinal steel ratio, the concrete strength, initial load eccentricity, tie spacing and tie type. For example, for column 2H20-70S, the longitudinal steel to concrete volumetric ratio is two percent, *H* identifies high strength concrete, the initial loading eccentricity is 20 mm, the centre-to-centre spacing of ties is 70 mm, *S* is the type of ties, indicating square ties for this instance.

3.2 Materials

The concrete used in this study was high-strength concrete with a target mean strength of 100 MPa. It was ready-mix concrete with maximum 10 mm aggregate and was supplied by a local ready-mix supplier. Cylinders (100 x 200 mm) and prisms (150 x 150 x 500 mm) were cast and tested to determine material properties. Tests for compressive strength (f_{cm}), indirect tensile strength (f_{sp}) by

Table 1. Details of test specimens.

Group	Specimen ID	Section (mm x mm)	Target Initial Load Eccentricity (mm)	Types of Tie	Tie Spacing (mm)
Group A	2H0-70S	200 x 200	0	Square	70
	2H10-70S	200 x 200	10	Square	70
	2H20-70S	200 x 200	20	Square	70
	2H50-70S	200 x 200	50	Square	70
	2H0-120D	200 x 200	0	Diamond	120
	2H50-120D	200 x 200	50	Diamond	120
Group B	2H0-50R	250 x 150	0	Rectangular	50
	2H12.5-50R	250 x 150	12.5	Rectangular	50
	2H25-50R	250 x 150	25	Rectangular	50
	2H0-100R	250 x 150	0	Rectangular	100
	2H12.5-100R	250 x 150	12.5	Rectangular	100
	2H25-100R	250 x 150	25	Rectangular	100



Column Size	C (mm)	D (mm)	E (mm)	F (mm)	Total Length (mm)
200 x 200 mm	350	100	900	400	1800
250 x 150 mm	400	150	1000	480	2100

Figure 2. Column details.

the Brazil test and modulus of rupture (f_{cf}) were performed in accordance with Australian Standards AS1012.9 (27), AS1012.10 (28), and AS1012.11 (29), respectively. The mean compressive strength at 28 days was f_{cm} = 96 MPa. Other properties were measured during testing period at 65 days and found as f_{cm}= 105 MPa, f_{sp}=7.4 MPa, f_{cf} = 9.0 MPa.

The longitudinal reinforcement consisted of eight Class N 12 mm diameter bars deformed bars with a nominal yield strength of 500 MPa. For the ties Class N 5.5 mm diameter wire with nominal yield strength 750 MPa was used. Tie spacing was selected based on the provisions given in AS3600-2009 (20). The true typical properties of reinforcing steel are given in Table 2, where f_{sy}, f_u, and E_s are yield strength, ultimate strength and modulus of elasticity, respectively, and the stress-strain curves are given in Figure 3.

Table 2. Properties of reinforcing bar.

Bar diameter (mm)	Area (mm^2)	f_{sy} (MPa)	f_u (MPa)	Ratio (f_u/f_{sy})	A_{gt} (%)	E_s (GPa)
12	115	520	640	1.23	7.5	200
5.5	31.2	810	934	1.15	6.8	200

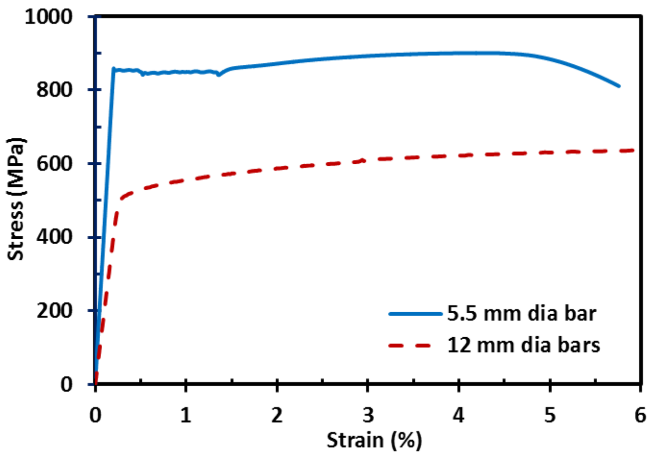


Figure 3. Stress-strain curves for reinforcing steel.

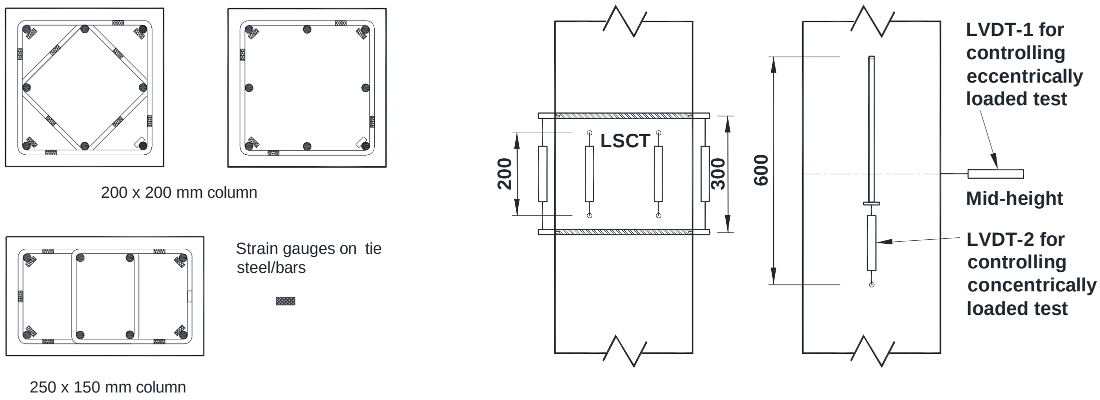
3.2 Instrumentation

The specimens were instrumented to measure the strains in the longitudinal reinforcing bars, tie steels and axial strains in the concrete. Strains in longitudinal bars were measured using 5 mm electrical resistance strain gauges attached to four corner longitudinal bars at their mid-level. For measuring the tie strain, 2 mm strain gauges were attached on the tie located at the mid-level. To measure the axial strain of concrete six linear strain conversion transducers (LSCTs) were installed with gauge lengths of 200/300 mm. For this purpose 10 mm diameter steel rods were placed through the specimen to mount the LSCTs. Lateral deflections were measured by linear variable displacement transducers (LVDT) and Laser Transducers. LVDT-1 and LVDT-2 controlled the loading rate for eccentrically and concentrically loaded tests, respectively. The instrumentations are shown in Figure 4.

3.2 Testing Procedure

Each column was tested vertically in a 5 MN stiff testing frame (Figure 5). High strength steel pins and bearing plates were placed at the desired nominal eccentricity at each column end to allow free rotation to the ends and to distribute the load. For the concentrically loaded columns, the rotational pins were removed from the test setup. A thin 4 mm Masonite board was placed between the specimen and the steel end plates to reduce any effect of unevenness at the ends.

Testing was undertaken in a closed loop servo control system. For the concentrically loaded columns, axial strain was used as the control through the test peak, with the strain measured over the 600 mm test zone (monitored by a LVDT placed vertically, Figure 5a). For the eccentrically loaded specimens,



(a) Strain gauge arrangement (b) LSCT and LVDT arrangement

Figure 4. Instrumentations.

(a) Concentric test set up (b) Eccentric test set up

Figure 5. 5 MN Instron testing frame.

For the eccentrically loaded tests, the test began under ram displacement control at a rate of 0.3 mm/min up to about 40% of the predicted column capacity. When the load reached this level, the load was paused and the electronically collected data was checked to ensure the control measures were behaving in a predictable way (i.e. moving forward in the direction of the control). When all indicators showed that the system was operating within normal bounds, control was then transferred to external lateral deflection control at a rate of 4.5 mm/hr. This loading continued at this rate as the load passed the peak and started dropping. When the load became stabilized, the displacement rate was increased to 10 mm/hr until failure or until 50 mm of lateral deflection had been reached.

For the concentrically loaded tests, the test began under ram displacement control at a rate of 0.3 mm/min up to 1500 kN. Control was then transferred to axial strain control at a strain rate of 0.0021 per hour (1.25 mm/hr on a 600 mm gauge length). When the load had dropped significantly from its peak value, the mode was changed back to ram displacement control at 0.05 mm/min.

4. Testing Results and Observations

4.1 General

Two adjustments were required in the experimental data. First a zero adjustment was made to all of the recorded data to filter out the effects of the redundancies, friction, etc. which prevent the curves showing an early linear behaviour. The second adjustment was the calculation of the initial eccentricity of the columns. Even with great care in the positioning of the columns for the planned initial eccentricity, the actual eccentricity needed to be determined precisely as this has a major influence on strength and ductility. A linear elastic back calculation was carried out for the behaviour of the columns in the early stage of the loading to determine the actual initial eccentricity of the load. The specimen 2H10-70S failed prematurely due to malfunction of servo-hydraulic system. The result of 2H10-70S will not be reported here.

4.2 Ultimate strength

One of the objectives of this study was to investigate the ultimate strength capacity of HSC columns confined by HSS ties placed at maximum spacing according to AS3600-2009 (20). The high yield-strength of tie steel resulted larger spacing than required by conventional steel.

For all columns actual measured eccentricities are used for moment calculations. Table 3 gives the peak loads, total eccentricities at peak load and the corresponding bending moments of all columns. In Figure 6 the ultimate loads and corresponding bending moments of the columns are plotted in their respective strength interaction diagram, defined by AS3600-2009 (20). Figure 7 compares the normalized interaction diagrams of the tested specimens (Group A) with that of the results from Foster and Attard (17) and Foster et al. (26).

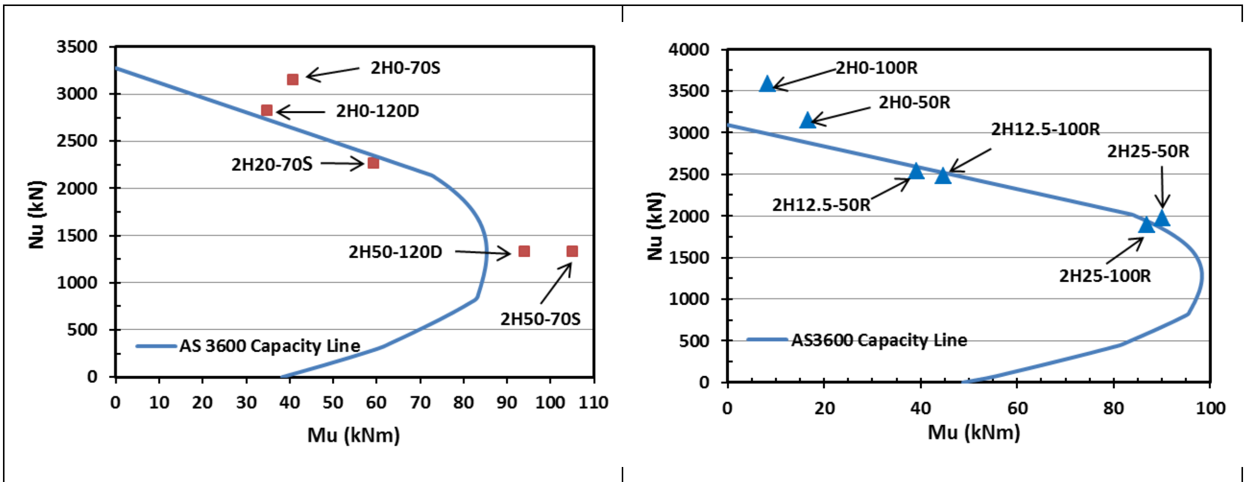
In Figure 6(a) four columns in Group A exceeded standard predicted ultimate strength with the calculated tie spacing. The capacity of two concentrically loaded columns 2H0-70S and 2H0-120D were affected due to actual eccentricity of loading plate. Specimen 2H20-70S achieved a slightly lower strength.

While bi-axial bending did not affect the capacity of columns in Group A, it influenced the capacity of the columns in Group B. With concrete axial strain measured in 6 LSCTs and steel strain measured in 4-corner longitudinal bars the bending of the column was monitored. In Figure 6(b) the two concentrically loaded columns demonstrated ultimate strength more than the predicted capacity. Due to bi-axial bending the column 2H12.5-50R and 2H12.5-100R were little short of reaching their standard calculated uniaxial strength line but showed consistent performance. From Table 3 and Figure 6(b) it can also be observed that specimen 2H12.5-50R and 2H25-50R reached a similar peak load ($\pm$80 kN) to that of 2H12.5-100R and 2H25-100R despite having double the volumetric ratio of ties. Hence, specified maximum spacing in AS3600-2009 (20) could be enough to gain the required strength in the HSC columns.

Table 3. Ultimate load, eccentricity, and ductility results.

Specimen ID	N_u (kN)	Actual Initial Load Eccentricity[a] (mm)	Total Eccentricity, *e,* at Peak Load (mm)	Moment, M_u (kNm)	I_{10}
2H0-70S	3144	11	13.0	40.9	-
2H20-70S	2263	20	26.2	59.4	7.38
2H50-70S	1332	64	78.9	105.1	-
2H0-120D	2826	9	12.3	34.8	-
2H50-120D	1330	61	70.7	94.1	-
2H0-50R	3144	1	5.33	16.8	-
2H12.5-50R	2532	11	15.4	39.1	9.82
2H25-50R	1977	35	45.6	90.1	8.93
2H0-100R	3584	1	2.3	8.4	-
2H12.5-100R	2484	11.5	18	44.7	6.89
2H25-100R	1896	32	45.8	86.9	9.27

[a]Actual eccentricities were measured using strains either from strain gauges fixed on longitudinal bars or LSCTs.



(a) Group A columns	(b) Group B columns

Figure 6. Column capacities compared to AS3600-2009 (20) interaction diagram.

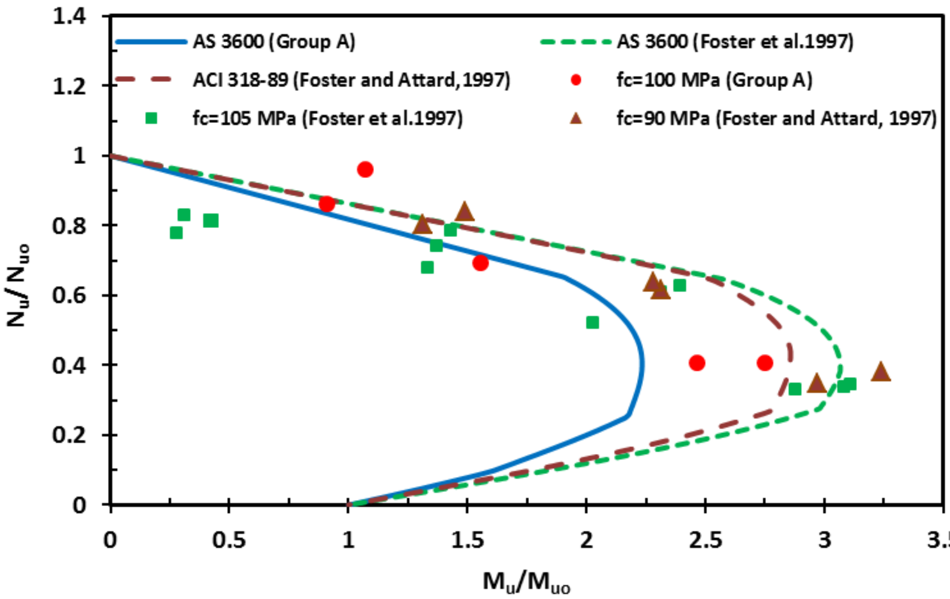


Figure 7. Comparison of Group A columns' capacity in normalised interaction diagram with Foster and Attard (17) and Foster et al. (26).

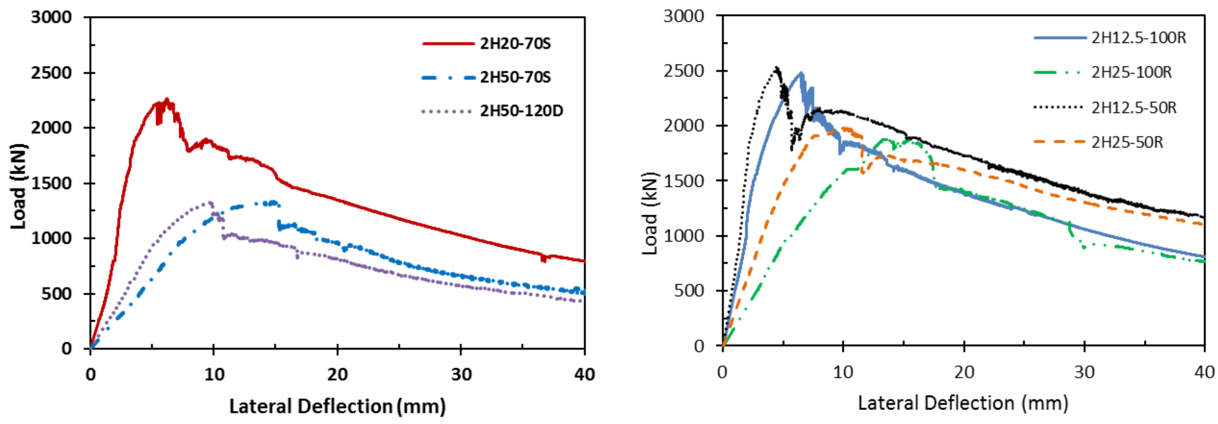
4.2 Column ductility

The second objective of this study was to investigate the ductility of HSC columns confined by HSS ties, while the ties were placed at the maximum spacing according to AS3600-2009 (20). There are different methods in the literature to indicate the ductility level of reinforced concrete columns. In this study ductility is measured following the method of Foster and Attard (17) as outlined in Section 2.3.

For proportioning column confinement reinforcement, AS3600-2009 (20) requires that a minimum ductility index of I_{10} = 5.6 be achieved. The ductility index I_{10} for the columns tested, for which I_{10} could be calculated, is given in Table 3 (noting that I_{10} cannot be calculated where the specimen failed outside of the central gauging zone). It is observed that the I_{10} values are well above the specified baseline.

From individual specimen results we can take note that 2H50-70S and 2H50-120D failed outside of gauged zone. Column 2H0-70S and 2H0-120D both were concentrically loaded and loads were controlled by the axial deformation rate. As the load reached the peak followed by cover spalling, there was a sudden load drop. Since the axial deformation control was running at very small rate, a sudden change in axial deformation caused the Instron actuator to apply a high load in a short period of time.

It did not allow running the test longer in post-peak region. It is recommended to change the load controlling procedure for concentrically loaded test. The load versus deflection curves for all columns tested eccentrically are shown in Figure 8.



(a) Group A columns (b) Group B columns

Figure 8. Load versus deflection curves.

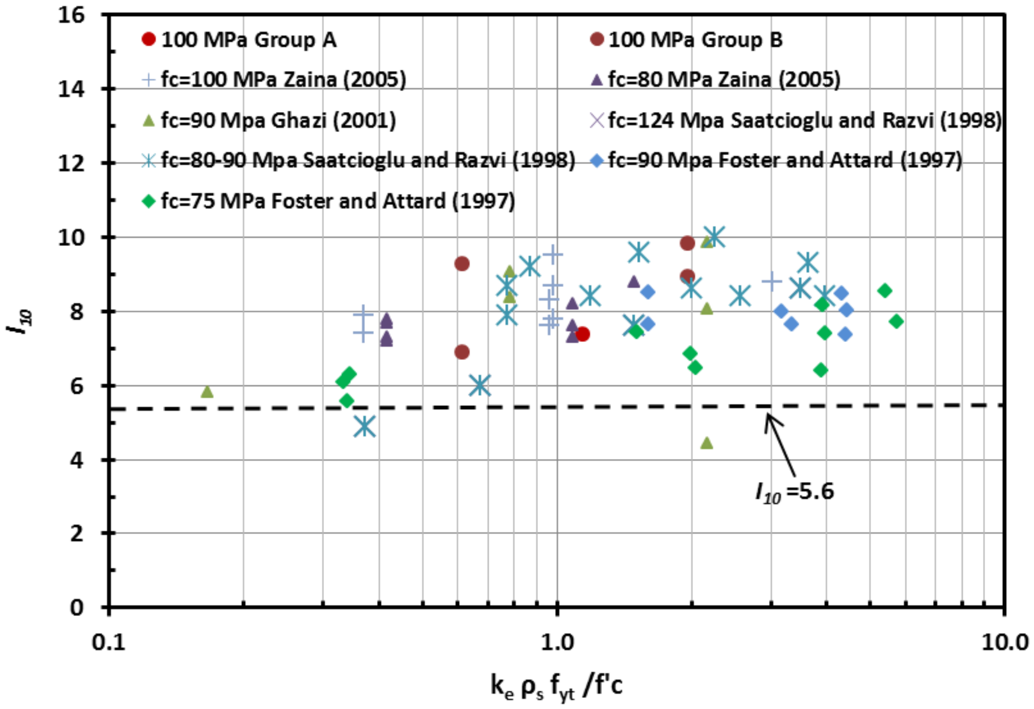


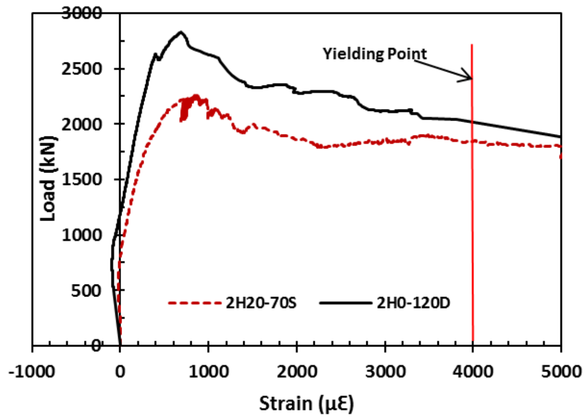
Figure 9. Ductility index of *I₁₀* versus the confinement parameter $k_e\rho_s f_{yt}/f_c^{'}$

Several researchers (5,6,17,18,30) have proposed that ductility in a column is a function of the confinement parameter, $k_e\rho_s f_{yt}/f_c^{'}$. The confinement parameter $k_e\rho_s f_{yt}/f_c^{'}$ can be used to calculate the required spacing for the tie reinforcement in columns so that the same level of ductility can be achieved for different grades of concrete strength. Figure 9 compares the ductility index of I_{10} calculated using the test results of Foster and Attard (17), Ghazi (31), Saatcioglu and Razvi (1), and Zaina (32), together with the results from this study. It was observed that the columns tied with HSS steel performed equally well, or better, than the HSC columns with conventional strength ties from the earlier studies. It is concluded that HSS ties provide the confinement needed to meet the established ductility requirements.

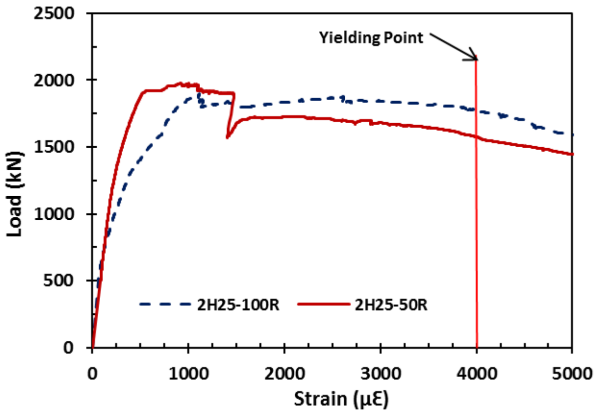
4.3 *Yield strength of tie reinforcement*

The passive confinement pressure is generated from the tensile stress that develops in tie reinforcement and the tensile stress is generated as a result of lateral expansion of concrete. Therefore, the effectiveness of HSS ties is dependent on the ability of concrete to expand laterally without failure. Tie spacing is an important parameter that affects the distribution of confinement pressure and stability of longitudinal reinforcement. The effectiveness of transverse reinforcement diminishes quickly with increasing tie spacing. The adverse effect of increased tie spacing can be offset by an increase in the volumetric ratio (1).

Strain gauges were attached to a tie located at the central section in each column. Figure 10 shows the load versus maximum tie strains of all columns. In no case did the strain in the ties reach yield strain at the peak load with the maximum strain in the ties at the peak load being 866$\mu\varepsilon$, which is consistent with observations of earlier studies of Foster and Attard (17), and Zaina (32). However, ties in most of the columns were engaged at or shortly after the peak load and it is notable that the load remained approximately constant between strains of 2000 $\mu\varepsilon$ (stress of 400 MPa) and 4000 $\mu\varepsilon$ (stress of 800 MPa).



(a) Group A columns



(b) Group B columns

Figure 10. Load versus maximum tie strain.

4.4 *Effect of tie arrangements and spacing*

In Group A rectangular and diamond shaped ties were used (Figure 4a) that resulted in spacings of 70 mm and 120 mm, respectively. The ratio of peak loads between 2H0-70S and 2H0-120D, 2H50-70S and 2H50-120D were 1.1 and 1.0, respectively. Having the same volumetric ratio, with different tie spacing both sets of columns performed consistently. In Group B, a set of columns was provided with ties at maximum spacing and another sets of columns was provided with ties at half spacing (double volumetric ratio) to study the adequacy of Standard defined maximum spacing. The ratios of peak loads between 2H12.5-50R and 2H12.5-100R, 2H25-50R and 2H25-100R were 1.02 and 1.04, respectively. From Table 3 it is observed that the ductility index I_{10} of each of these 4 columns was above 6.5. This indicates that the maximum tie spacing (100 mm) provided according to AS3600-2009 (20) with HSS tie reinforcement is sufficient to ensure strength and ductility requirements are met.

3. Conclusions

In this study 12 end-haunched HSC columns confined with HSS ties were tested to investigate the strength and ductility with reference to Australian Standard for Concrete Structures, AS3600-2009. Ties had a nominal yield strength of 750 MPa, and were placed at the maximum spacing allowed by AS3600 for a 750 MPa steel. On average tie spacing was 30% larger than required by conventional steel ties. Test variables were loading eccentricities, tie arrangement and column section. To compare the effectiveness of the maximum tie spacing, a set of columns was tested with half tie spacing required by Standard.

The ultimate strength of the columns was compared to the strength predictions based on the AS3600-2009 design models. The experimental results compared reasonably well with Standard predictions and with previous test results of HSC column confined with conventional steel ties. However, slightly lower than predicted strengths about the major axis was observed in two columns of Group B; this was due to bi-axial bending, which was observed and measured. The ultimate strength of columns did not change as the maximum tie volumetric ratio was doubled, satisfying the adequacy of present Standard models. These results indicate that HSS ties are able to provide the strengths predicted by the Standard but with 30% less tie reinforcement – potentially saving steel and costs. Significantly a reduction of 30% in the steel mass potentially results in a reduction of some 25% in greenhouse gas emissions from steel production and transport.

The I_{10} ductility measure was calculated for the columns where it was possible to trace the softening branch. Having used 30% less confinement steel, I_{10} measures for this study were well above the AS3600-2009 base line (I_{10} = 5.6). For the same confinement level, $k_e \rho_s f_{yt}/f_c^{\prime}$, the columns from this study showed higher ductility measures. The confinement parameter is based on the assumption that ties are at yield at, or shortly after, the peak load is reached. Strain measurement show that while the tie reinforcement was not at yield when the peak load was reached, the load as the tie strains increased.

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