Behavior of Confined High Strength Concrete Columns under Axial Compression

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Abstract

An experimental study was carried out to investigate the behavior of high strength concrete short columns confined by circular spirals and square ties under monotonically increasing concentric compression. The test variables included volumetric ratio, spacing and yield strength of transverse reinforcement, longitudinal reinforcement ratio, lateral steel configuration, shape of cross section and concrete compressive strength. The effects of these variables on the uniaxial behavior of high strength concrete columns are presented and discussed. The results indicate that more confinement is required in columns of high strength concrete than in columns of low strength concrete to achieve the desired post-peak deformability. The behavior of high strength concrete columns is characterized by the sudden spalling of concrete cover, leading to a loss of axial capacity. A comparative study of existing confinement models of high strength concrete columns was also conducted to assess their capabilities of predicting the actual test behavior. To this end, the stress-strain curves of the specimens tested in the present study were compared with the ones predicted by the various models. It is shown that Legeron & Paultré (2003) model estimates the actual experimental curves more closely as compared to the other models employed in the study.

1. Introduction

Inelastic deformability of reinforced concrete columns is essential for overall stability of structures in order to sustain strong earthquakes. Deformability of columns can be achieved through proper confinement of the core concrete. It is now well documented that the desired ductility can be attained in case of normal strength concrete columns by providing well-detailed lateral confinement reinforcement (Richart et al. 1928; Sheikh & Uzmeri 1980; Park et al. 1982; Mander et al. 1988). However, the gradual development of concrete technology has promoted the use of high strength concrete owing to its wide range of advantages over normal strength concrete and as a result, concrete strengths much higher than 60Mpa have gained acceptance in the construction industry. The high strength concrete is significantly more brittle than conventional normal strength concrete. While, existing code provisions for minimum amount of confinement reinforcement are based on experiences with normal strength concrete, questions have been asked as to whether similar amount of confinement is suitable for high strength concrete columns (Razvi & Saatchiouglu 1994; Pessiki & Pieroni 1997; Foster 2001). The few studies carried out on confinement of high strength concrete columns in the recent past indicate that the confinement steel requirements for such columns have not yet been fully developed (Bjerklie et al. 1990; Muguruma et al. 1993; Li et al. 1994; Cusson & Paultré 1994; Razví & Saattcioglu 1996; Foster & Attard 2001). This is also evidenced by the lack of design provisions in the current relevant building codes of various countries. ACI Committee 441 (1997) also emphasized that more data on the confinement of high strength concrete columns are needed. Therefore, as a part of ongoing combined experimental and analytical study to fully discover the strength and ductility of high strength concrete columns, this paper reports the results of concentrically tested circular and square high strength concrete columns. An attempt has also been made to evaluate the capabilities of existing stress-strain models for high strength concrete by comparing the experimentally found curves with the ones estimated by the various models.

2. Experimental program

2.1 Test specimens

A total of 44 high strength concrete columns, 600 mm in length, were tested under concentric compression. They included 18 tie confined 150 mm square specimens and equal numbers of spiral confined 150 mm diameter circular section columns. The remaining 8 columns were prepared as 4 square and 4 circular plain (unconfined) concrete specimens to establish the properties of unconfined concrete and thereby to get comparison of in place strength of concrete with standard cylinder compressive strength. To properly investigate the behavior of high strength concrete columns, the specimens were cast and

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tested in duplicate for each case in order to get the average of two results thus making 22 independent column designs. All the details regarding the various column specimens are illustrated in Table 1 and Fig. 1. Most of the results reported here in this study are the averages of two tests with a few exceptions of SG, SC, CH and SF columns where duplicate results could not be taken due to the following reasons. One of the SG specimens failed rather more suddenly compared to the other and all the measuring instruments like LVDTs and strain gauges got dislodged and further readings after peak could not be taken. In case of one of the CH specimens the top end concrete cover of the column got crushed and the loads came directly on the end steel collars which got damaged and therefore the test was stopped. Whereas one of the results of each of SC and SF specimens were rejected due to the reason that in spite of exercising extra care to avoid any eccentric loading of the specimens, these columns seemed to suffer from this problem as evidenced by their considerable tilting and entire spalling on one side only.

The specimens were cast in total eleven sets. Each set consisted of two duplicate circular and two duplicate square columns of one design. For example two specimens of CA and SA each were cast in one set. The columns were designed to investigate the parameters of confinement, including volumetric ratio and spacing of transverse reinforcement, yield strength of transverse reinforcement, longitudinal reinforcement ratio, and lateral steel configuration in addition to shape of cross section and concrete compressive strength. Concrete cover of 10 mm was provided in all the confined specimens. A concrete cover of 15 mm was also provided between the ends of the longitudinal bars and the top and bottom surfaces of the specimens to prevent direct loading of the bars. In all the specimens, the ratio of gross area of the section, , to the core area, , measured to the outside of the lateral reinforcement was 1.33. Longitudinal reinforcement was provided in all the confined column specimens. The lateral ties of all square specimens had 135° hooks and a development length as per the ACI 318 code provisions. Fig. 1 and Table 1 show the various properties of longitudinal and transverse reinforcement. The notation shows standard cylinder compressive strength on the day of testing of columns, and are respectively longitudinal steel ratio and yield strength of longitudinal steel, , and are respectively spacing, volumetric ratio and yield strength of lateral steel. Two different specified concrete strengths, two lateral steel yield strengths and two longitudinal steel ratios and resulting lateral steel configurations were employed. The spacing of spirals and ties were varied from approximately one fourth to half of the total lateral dimension of column in order to give varying volumetric ratios of lateral confining steel.

Failure of the specimens was forced in the test region, which was equal to 300 mm in the middle of the specimen height by reducing the spacing of the lateral reinforcement out side the test region to half of the specified spacing in the test region. However, in the case of the specimens where specified spacing of lateral steel was 30 mm, the spacing in the end zones was reduced to only three fourth of that in test region to facilitate proper placing of concrete in the cover of columns in these end regions. All the specimens were also externally confined by 10mm thick mild steel collars in the end regions of 150 mm to further prevent premature failure.

Fig. 1 Details of column specimens, reinforcement arrangement and location of strain gauges.
The fabricated steel cages were fixed in the steel moulds and the casting was carried out in the laboratory. After 24 hours, the specimens were removed from the moulds and submerged in a water tank for curing. The water-curing period lasted for 28 days, after which the specimens were left in the laboratory at ambient temperature until the time of testing. The testing was commenced after 90 days of ageing.

### 2.2 Material properties

Two high strength concrete mixes were employed in the study. The materials consisted of Ordinary Portland Cement, natural river sand, crushed stone aggregate of maximum size 10mm, tap water for mixing and curing, silica fume and superplasticizer admixture to maintain adequate workability of mix. To obtain the desired strength, several preliminary mix designs were practiced and then optimal mix designs were determined after testing 28 days strength. The final mixes had 28 days average compressive strengths of 58.03 and 76.80 MPa. **Table 2** shows the mix proportions and 28 days compressive strengths for both the mixes. The measured tangent elastic moduli ($E_t$) were 31645 MPa and 34465 MPa for lower and higher strength concrete mixes respectively. The slump value ranging from 100-120mm was maintained to ensure that the concrete could be placed through the dense reinforcement cages.

Three standard concrete cylinders (100 x 200 mm) were cast along with each set of four columns. They were tested at various ages to determine the strength of concrete $f'_c$, on the day of testing of columns. The properties of unconfined concrete were obtained by testing plain unreinforced column specimens (CPL, CPH, SPL & SPH). It was observed that concrete strength of column specimens were generally lower than the concentric strength measured on standard cylinders. The average unreinforced specimen concrete strength was measured as 88% and 90% of the average concrete cylinder strength for the lower high strength (CPL & SPL) and upper high strength (CPH and SPH) concrete respectively. The commonly used ratio of 0.85 was then used for evaluating the concrete section capacity of the specimens tested in the present study.

The longitudinal reinforcement consisted of 8 mm and 12mm deformed bars for circular columns and 12 mm deformed bars for square columns. Two different grades (412 MPa and 520 MPa) of reinforcing steel with a diameter of 8 mm were used as lateral reinforcement.
Stress-strain relationships, established by performing at least three coupon tests for each reinforcement, are illustrated in Fig. 2. The yield strength of the lateral reinforcement is defined as an offset strain of 0.2%.

2.3 Instrumentation and testing procedure

Longitudinal and lateral steel strains were measured by electrical resistance strain gauges glued to the steel bars. The strain gauges were pasted to the two opposite longitudinal steel bars at their middle lengths. Similarly two gauges were glued at the two locations on the lateral steel at approximately middle length of the specimen as shown in the Fig. 1. The axial displacement of the specimens was recorded using four linear variable differential transducers (LVDTs). Two LVDTs were attached on the opposite faces to a top and bottom steel clamps to the specimens to give a gauge length of 250 mm. Whereas, two LVDTs were mounted on the other two faces of the specimen and were held by the attachments provided to the end steel collars. If a sudden failure occurs due to spalling, the steel clamps get dislodged, the LVDTs mounted on the end collars could be used instead of the gauge length over the central 250 mm. LVDTs were of 50 mm stroke. Loads were recorded through a 3000 kN load cell. The recorded strain data from the four LVDTs, four strain gauges and corresponding load data from load cell were fed to a data acquisition system and stored on a computer. Overall view of the test setup is shown in Fig. 3.

The top and bottom ends of the specimens were slightly ground by a grinding machine to remove any surface unevenness. But, at the same time it was ensured that the excessive grinding is not carried and the required cover is maintained to the longitudinal bars at the ends. In addition, 6mm wooden ply pieces were put at the top and bottom ends of the specimens to ensure parallelism of specimen end surfaces and uniform distribution of the load on the specimen. The test specimens were loaded using a 5000 kN capacity hydraulic universal testing machine with load controlled capabilities. The monotonic concentric compression was applied at very slow rate to capture the post-peak part of the measured load deformation curves by manually controlling the oil pressure. The load was applied from zero to failure, which was determined primarily by either rupture of the lateral reinforcement or excessive crushing of the core, together with buckling of the longitudinal bars. The time taken to complete each test ranged from 30 minutes to 1 hour depending upon the degree of confinement in the specimen. To ensure concentric loading, an initial load of approximately 20% of the total ultimate load was applied and the readings of the four LVDTs were monitored. If the readings of the LVDTs were not approximately equal, this signals that the load is not concentric. The column was then unloaded and the additional packing was given under the wooden plies unless the load becomes almost concentric. In spite of the rigorous procedure followed for aligning the specimens, some eccentricities were unavoidable.

![Fig. 2 Stress-strain curves for reinforcing bars.](image)

![Fig. 3 Experimental setup.](image)

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement Kg/m³</th>
<th>Water Kg/m³</th>
<th>Coarse Aggregate Kg/m³</th>
<th>Sand Kg/m³</th>
<th>Silica Fume Kg/m³</th>
<th>Super-plasticizer Kg/m³</th>
<th>28 Days Cylinder Compressive Strength f'_{cy}, MPa</th>
<th>28 Days Cube Compressive Strength f'_{ck}, MPa</th>
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<tr>
<td>Mix 1</td>
<td>545</td>
<td>169</td>
<td>1105</td>
<td>700</td>
<td>50</td>
<td>5.45</td>
<td>58.03</td>
<td>68.40</td>
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<tr>
<td>Mix 2</td>
<td>600</td>
<td>168</td>
<td>1055</td>
<td>625</td>
<td>50</td>
<td>7.5</td>
<td>76.80</td>
<td>87.50</td>
</tr>
</tbody>
</table>

* Average of 5 Specimens.
3. Observed behavior

All columns initially behaved in a similar manner and exhibited relatively linear load deformation behavior in the ascending part, which is typical of high strength concretes. The plain concrete specimens CPL, CPH, SPL & SPH, had a sudden explosive type of failure at the maximum axial load. The complete load deflection curves for these specimens could not be obtained with the present testing facility, and no readings could be taken after their brittle failure. The strains corresponding to the peak loads were 0.00232, 0.00256, 0.00237 and 0.0026 for CPL, CPH, SPL and SPH specimens respectively.

The general behavior of confined specimens was comparatively ductile and complex unlike plain unconfined columns. These columns were characterized sequentially by the development of surface cracks, cover spalling, yielding of longitudinal steel, yielding of lateral steel, fracture of spiral or ties, buckling of longitudinal bars and crushing of core concrete. The vertical longitudinal cracks were noticed invariably first of all for almost all the columns just before the cover spalling. These cracks thereafter eventually led to the spalling of cover concrete, which was marked by the separation of large pieces of cover from core concrete. 

Figures 4 and 6 illustrate the appearance of a typical specimen at various stages of loading. The spalling was comparatively more sudden and explosive for columns with higher strength concrete mix. The beginning of cover spalling was carefully observed and marked during testing. The strain data (Table 3) indicated that spalling of cover occurred prior to the development of strains associated with the failure of unconfined concrete specimens.

The cover spalling often resulted in a sudden drop in load, which was also more pronounced in higher concrete strength columns. Load resistance of columns again increased to a second peak normally for well-confined specimens only. This behavior indicates that the passive confinement becomes active only after the cover has spalled and post-spalling behavior depended solely upon the confinement level of specimens. Figures 5 and 6 show the total column axial load normalized to unconfined column capacity versus longitudinal strain curves for circular and square specimens respectively.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Axial Loads</th>
<th>Axial Strains</th>
<th>( I_{10} )</th>
<th>( f_{sec} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circuler</td>
<td>( P_{max} )</td>
<td>( P_{max}/P_{oc} )</td>
<td>( P_c )</td>
<td>( P_c/P_{oc} )</td>
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<tr>
<td>CA</td>
<td>1109</td>
<td>1.063</td>
<td>986</td>
<td>1.074</td>
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<tr>
<td>CB</td>
<td>1059</td>
<td>1.006</td>
<td>947</td>
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<tr>
<td>CC</td>
<td>1148</td>
<td>1.106</td>
<td>1024</td>
<td>1.115</td>
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<tr>
<td>CD</td>
<td>1241</td>
<td>1.049</td>
<td>973</td>
<td>1.063</td>
</tr>
<tr>
<td>CPL</td>
<td>963</td>
<td>1.035</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>CE</td>
<td>1381</td>
<td>1.07</td>
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</tr>
<tr>
<td>CF</td>
<td>1294</td>
<td>0.972</td>
<td>1175</td>
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<tr>
<td>CG</td>
<td>1352</td>
<td>0.980</td>
<td>1206</td>
<td>0.982</td>
</tr>
<tr>
<td>CH</td>
<td>1321</td>
<td>0.992</td>
<td>1202</td>
<td>0.995</td>
</tr>
<tr>
<td>CI</td>
<td>1379</td>
<td>0.956</td>
<td>1111</td>
<td>0.932</td>
</tr>
<tr>
<td>CPLH</td>
<td>1312</td>
<td>1.062</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Square</td>
<td>( P_{max} )</td>
<td>( P_{max}/P_{oc} )</td>
<td>( P_c )</td>
<td>( P_c/P_{oc} )</td>
</tr>
<tr>
<td>SA</td>
<td>1334</td>
<td>0.992</td>
<td>1155</td>
<td>0.990</td>
</tr>
<tr>
<td>SB</td>
<td>1364</td>
<td>1.005</td>
<td>1185</td>
<td>1.006</td>
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<td>SC</td>
<td>1308</td>
<td>0.977</td>
<td>1133</td>
<td>0.977</td>
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<tr>
<td>SD</td>
<td>1626</td>
<td>1.069</td>
<td>1268</td>
<td>1.090</td>
</tr>
<tr>
<td>SPL</td>
<td>1239</td>
<td>1.046</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>SE</td>
<td>1641</td>
<td>0.951</td>
<td>1463</td>
<td>0.946</td>
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<tr>
<td>SF</td>
<td>1604</td>
<td>0.937</td>
<td>1433</td>
<td>0.935</td>
</tr>
<tr>
<td>SG</td>
<td>1730</td>
<td>0.995</td>
<td>1551</td>
<td>0.995</td>
</tr>
<tr>
<td>SH</td>
<td>1621</td>
<td>0.946</td>
<td>1446</td>
<td>0.943</td>
</tr>
<tr>
<td>SI</td>
<td>1819</td>
<td>0.971</td>
<td>1461</td>
<td>0.964</td>
</tr>
<tr>
<td>SPH</td>
<td>1684</td>
<td>1.070</td>
<td>--</td>
<td>--</td>
</tr>
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</table>

--- Not applicable, * Could not be measured.
The second peak was observed for many circular columns CA, CC, CD, CE, CH and CI, and a few square columns SA, SD, SE and SI which have better efficiently confined core than the other specimens of the study as implied by their respective volumetric ratios, spacing and distribution of lateral steel. In the case of CE and SD specimens the load values at second peak were in fact more than those at respective first peaks. This may be attributed due to the fact that CE specimen has a very high volumetric ratio and closer spacing of spirals. Square column SD had also high volumetric ratio and the lateral tie arrangement of eight bars, which provided better confinement than an arrangement of four bars. However, the SG, CG and SB specimens failed immediately after the first maximum load was reached, indicating that the spacing of the lateral steel was too wide to provide lateral confinement. Overall, it was observed that for the similar confinement circular columns had a better post peak behavior than the square tie confined columns and the lower high strength mix columns failed slowly in a better ductile fashion than the similarly confined higher concrete strength mix columns.

At the stage of cover spalling, in all the specimens longitudinal steel yielded but, the level of stress in lateral confining steel was considerably less than their respective yield strengths. Similar observations were made in some earlier studies also (Cusson & Paultre 1994; Foster 1999). Lateral steel yielded either at the second peak or even after that in the descending part of load-deformation behavior. In fact yielding of lateral steel at second peak was noticed only for well-confined columns and mostly for lower high strength mix specimens. They were CA, CC, CD, and CE circular columns and SD and SI square column. Fracture of transverse reinforcement was observed only for the specimens with closer spacing of spirals or ties i.e. 30 and 50 mm. The stage of spiral or tie fracture was invariably at the moment when the load dropped to approximately 40-50% of the maximum value in the descending segment. It was observed that the fracture of lateral steel caused a sudden drop in load. In the case of columns with 75 mm spacing of transverse steel, buckling of longitudinal steel was observed prior to the fracture of lateral steel, which ultimately lead to the fracture of ties and crushing of core.

### 4. Test results

The columns were analyzed to obtain the stress-strain curves of confined concrete, as suggested by Sheikh & Uzumeri (1980) and Cusson & Paultre (1994). The concrete contribution $P_c$ at a certain deformation was determined by subtracting the contribution of longitudinal steel from the applied load $P$. The load carried by the longitudinal steel was determined from the stress-strain curves obtained from the tension test. In computing the load carried by the longitudinal steel, the strain hardening regions of the curves were assumed to be a straight line that resulted in a maximum error of less than 5% in the computed concrete force. The concrete contribution curves were nondimensionalized with respect to gross concrete area force $P_{oc}$ and core concrete area force $P_{occ}$. 

![Graph](image1.png)

Fig. 5 (a) & (b) Column axial load versus axial strain curves (circular columns).

![Graph](image2.png)

![Graph](image3.png)

![Graph](image4.png)

Fig.6 (a) and (b) Column axial load versus axial strain curves (square columns).
While the gross concrete area $A_c$ represented column behavior before the cover started spalling, only the core area $A_{cc}$ resisted the applied load after concrete cover was completely spalled. A smooth transition was assumed to take place from the lower curve to the upper curve. In Fig. 7, dark line shows the behavior of confined concrete. The load sustained by the confined core concrete $P_{cc}$ was calculated by subtracting the load carried by the cover concrete from $P_o$. The maximum loads $P_{max}$, $P_o$, and $P_{cc}$ were normalized by $P_{oc}$, $P_{cc}$ and $P_{occ}$ respectively and these values were compared with each other as shown in Table 3, where

$$P_o = 0.85 f'c A_c$$

$$P_{cc} = 0.85 f'c A_{cc}$$

The ratio $P_{occ}/P_o$ ranges from 0.95 to 1.10 for circular columns and 0.93 to 1.07 for square columns. The lower ratios are observed for specimens with higher strength concrete and closer spacing of lateral steel indicating loss in column capacity due to spalling. Similarly $P_c/P_{occ}$ varies from 0.93 to 1.11 for circular columns and 0.93 to 1.09 for square columns. The stress ratio $\varepsilon_c/\varepsilon_{cc}$ ranges from 0.89 to 1.07 for circular columns and 0.84 to 1.10 for square columns indicating premature failure of cover for higher strength concrete columns with dense lateral steel spacing. Here $\varepsilon_c$ stands for axial strain corresponding to the first peak i.e. beginning of spalling and $\varepsilon_{cc}$ is peak strain of unconfined concrete column. The load ratio $P_{cc}/P_{occ}$ ranges from 1.19 to 1.68 for circular columns and 1.10 to 1.68 for square columns. The ratio of peak confined strain $\varepsilon_{cc}$ to $\varepsilon_{co}$ varies from 1.32 to 3.5 for circular columns and 1.09 to 3.0 for square columns. The stress ratio $\varepsilon_{cc}/\varepsilon_{co}$ ranges from 3.12 to 13.92 for circular columns and 1.61 to 13.50 for square columns. Where, $\varepsilon_{cc}$ and $\varepsilon_{co}$ are axial strain corresponding to 0.5 $P_{cc}$ in the descending part. The values of ratios $P_{cc}/P_{occ}$, $\varepsilon_{cc}/\varepsilon_{co}$ and $\varepsilon_{cc}/\varepsilon_{co}$ prove that the important gains in deformability of core concrete can be achieved following the spalling of the cover concrete depending upon the level of confinement.

To quantify the effect of test variables on the post-peak deformability of confined high strength concrete columns more effectively $I_{10}$ ductility index can be used (Foster 1999; Foster & Attard 2001). The definition of $I_{10}$ ductility index is adopted from ASTM C1018 (26) for the measurement of toughness. The $I_{10}$ index is the ratio of the area under the load versus strain curve up to a strain of 5.5 times the yield strain to the area under the curve for a strain equal to the yield strain. The yield strain is taken as 1.33 times the strain corresponding to a load of 0.75 times the maximum load on the ascending side. The significance of taking area under the curve up to 5.5 times the yield strain is that for a perfectly elasto-plastic material $I_{10} = 10$, while for a perfectly elastic-brittle material $I_{10} = 1$. The $I_{10}$ values were computed for all the specimens of the study and the same are given in Table 3.

### 5. Effect of test variables

#### 5.1 Concrete compressive strength

The concrete compressive strength was the most important and one of the primary variables investigated extensively in the test program. The behavior of specimens with same volumetric ratio, spacing, configuration and yield strength of lateral steel but with different concrete strengths was compared to quantify the effects of this parameter. Fig. 8 shows comparisons of both circular and square columns with different strength concretes. The post peak curves of the higher strength concrete mix columns are steeper indicating faster rate of strength decay as compared to the lower strength concrete specimens. The decreasing trend of strength enhancement ($P_{cc}/P_{occ}$) and $I_{10}$ ductility values for both circular and square columns from lower strength concrete mix specimens to higher strength mix specimens show that effectiveness of confinement decreases as the concrete strength increases (Table 3). For example, if similarly confined CC (61.85 MPa) and CH (81.80 MPa) specimens are compared, the percentage increase in strength reduced from 68 to 41% and $I_{10}$ ductility decreased from 8.58 to 7.64 from former to latter. Therefore, if the same levels of strength and ductility enhancements are desired, higher strength concrete columns shall require more confinement than lower strength concrete columns.

#### 5.2 Volumetric ratios and spacing of lateral steel

The importance of the amount of lateral confining steel as a factor that affects the behavior of confined concrete is well recognized. An increase in the volumetric ratio of confinement steel may be directly translated into a proportional increase in lateral confining pressure. Also, the spacing of transverse reinforcement is an important parameter that affects the distribution of confinement pressure on the confined core in addition to the stability of
5.3 Yield strength of transverse steel

The effect of yield strength of transverse confining steel was investigated by comparing results of two pairs each of circular and square columns. The compared specimens of each pair had the same concrete strength as well as the same volumetric ratio, arrangement, and spacing of lateral steel but different yield strengths (Fig. 10). In general, it is observed that as the yield strength is increased from 412 MPa to 520 MPa, the strength and deformability of confined concrete get improved. However, for the range of steel grades used in this study, the results show that the effect of increasing yield strength of lateral steel has not very significant effect on the behavior of high strength concrete columns. The maximum enhancements of 5% in strength and 10% in $I_{t0}$ ductility were observed. This may be due to the reason and as mentioned in the earlier section also, that the transverse steel of only well confined specimens reached its yield value at peak-confined stress of concrete and that too for mostly lower concrete strength circular columns only. The values of stress, $f_{yhl}$, in lateral steel at peak confined stress of different specimens are given in Table 3. The passive confinement pressure is generated from the tensile forces that develop in the lateral confining steel. However, tensile stress in transverse steel is generated as a result of lateral expansion of the core concrete, which in turn is dependent upon the characteristics of concrete. The higher strength concrete exhibits less lateral expansion than lower strength concrete under axial loads due to its higher modulus of elasticity and its lower internal micro cracking. Therefore, the advantages of providing higher yield strength lateral steel in high strength concrete columns can be realized only if considerably higher...
amount of confinement is provided to generate enough lateral expansion of core so that yield strength of high yield strength lateral steel is fully developed prior to appreciable post-peak strength decay.

5.4 Configuration of lateral steel
The transverse steel configuration and the resulting distribution of the longitudinal steel play a significant role in the confinement of concrete. If the lateral confining pressure applied by the transverse reinforcement on concrete is well distributed around the perimeter of the core concrete, the efficiency of confinement is improved. In this study it was possible to observe the effect of varying the lateral steel configuration by comparing the behavior of square column specimens SE and SI, which have almost equal volumetric ratio of lateral steel (Fig. 11). The specimen SE was constructed using single perimeter tie (4 bar arrangement), and SI specimen had double tie configuration (8 bar arrangement). The ductility indices $I_{10}$ of the specimens SE and SI were 6.44 and 7.86, respectively indicating a 22% improvement in ductility of specimen SI which has better steel configuration. However, the ratio $P_{cc}/P_{occ}$ is equal to 1.382 and 1.43 for specimens SE and SI respectively showing strength enhancement of only 3.5%. This indicates that the improvements in deformability are more marked than in strength due to better configuration of transverse steel. Therefore, according to the test results, transverse steel configuration and the resulting distribution of longitudinal steel has a considerable effect on the ductility of the confined concrete but insignificant effect on its strength.

5.5 Amount of longitudinal reinforcement
Fig. 12 shows the two different pairs of circular specimens, and for each matched pair, two specimens varying only in their ratios of longitudinal reinforcement are compared. It may be mentioned here that this type of comparison was not possible in case of square columns because, in addition to amount of longitudinal steel, volumetric ratios of lateral ties were also varying for any two comparable specimens. The results show (Table 3) that the strength and deformability gains are equal to 0.4% and 6.4% for the pair CA-CD and 1.2% and 11% for the pair CF-CI, respectively. Therefore, based upon the results it can be said that the amount of longitudinal steel has only little effect on the behavior of confined concrete, which might be because of the reason that larger longitudinal bar diameters prevent their premature buckling.

5.6 Section geometry
It is well established now that circular spirals are more effective in confining concrete than rectilinear ties owing to their better uniform distribution of lateral confining pressure around the core compared to the case of square or rectangular ties (Mander 1988; Razvi & Saatcioglu 1996). In the present study this variable was investigated extensively to quantify its effect in case of confinement of high strength concrete columns. However, only a few representative comparisons are shown in Fig. 13 with respect to this parameter. The specimens of each matched pair have similar concrete strength and volumetric ratios of lateral steel. It is clear from this figure that circular confinement is substantially more effective than rectilinear confinement in case of high strength concrete also as evidenced by the margin of increase in

![Fig. 11 Effect of configuration of lateral ties.](image1)

![Fig. 12 Effect of amount of longitudinal steel.](image2)

![Fig. 13 Effect of section geometry on confined concrete.](image3)
strength and ductility from square to circular columns. The square columns SC and SE failed in a comparatively brittle fashion at 0.0042 and 0.0051 axial strains respectively, while the matched circular columns CC and CE were able to develop axial strains of 0.0081 and 0.0074 respectively without any strength decay. The maximum gains of 28% in confined strength (CC vs SC) and 66% in I₁₀ ductility (CB vs SB) were noticed due to the change in section geometry. Therefore, if the same percentages of strength and ductility enhancements are desired, square or rectilinearly confined columns are required to be confined more vigorously than circular columns.

6. Design implications for ductility

As a result of some field observations regarding the performance of columns after major earthquakes, it is now well established that despite following the strong-column weak-beam concept in design, damage could occur at ends of the columns. Therefore, ductile detailing of these potential plastic hinge regions of columns becomes even more important when high strength concrete is employed. One of the ways in which most of the building codes of various countries ensure ductility in concrete columns, is by specifying the amount of confining transverse reinforcement in critical regions of columns. However, these code equations specifying the minimum amount of transverse steel for columns are empirical and have been developed based upon the experimental data obtained from the testing of normal strength concrete columns. An attempt has been made in the present study to work out the confinement requirements of high strength concrete columns, based upon the present test results. Ideally, the behavior of columns should be studied under combined axial compression and bending to assess the ductility performance of confined reinforced concrete and the effect of cyclic loading should be taken into account. But, in this study uniaxial load-strain data of confined concrete column sections under monotonic loading has been analyzed, and this should give a reasonable assessment in the first instance considering it to be an extreme case.

A number of past studies (Sheikh & Uzmeri 1980; Mander et al 1988, Razvi & Saatcioglu 1994, Foster & Attard 2001) have indicated that ductility is a function of effective confinement index \( k_c \rho_s f_{sh} / f'_c \), where \( k_c \) is a confinement effectiveness parameter, which accounts for configuration of lateral steel and resulting longitudinal steel distribution. However, the coefficient \( k_c \) has been ignored by the present seismic code requirements as indicated by the following relevant expressions of ACI 318 code for rectilinearly confined columns:

\[
\rho_s f_{sh} / f'_c \geq 0.18
\]  
\[
\rho_s f_{sh} / f'_c \geq 0.6 \left( \frac{A_s}{A_c} - 1 \right)
\]

In Fig. 14, the \( I_{10} \) ductility index for all the specimens is plotted against the effective confinement parameter \( k_c \rho_s f_{sh} / f'_c \). The \( k_c \) for both spirally confined circular columns and tie confined square columns was computed using the procedure suggested by Mander et al (1988). The computed values of confinement index \( k_c \rho_s f_{sh} / f'_c \) for the various confined specimens are given in Table 1. In a few columns (CG, SB, SF and SG) tested in this study, a faster rate of strength decay was observed after peak. For these columns, an actual value of \( I_{10} \) ductility could not be obtained. Therefore, for such cases average values of \( I_{10} \) were calculated by obtaining the upper and lower bounds (Foster & Attard 2001). A best fit relationship between \( I_{10} \) and \( k_c \rho_s f_{sh} / f'_c \), was found from the plot and is given by:

\[
I_{10} = 2.89 \ln \left( 1000 k_c \rho_s f_{sh} / f'_c \right) - 0.45
\]  

Foster & Attard (2001) analyzed the test data of 40 concentrically loaded columns tested by Razvi & Saatcioglu (1996) and computed \( I_{10} \) values for all the specimens. For the specimens with \( I_{10} < 8 \) sudden changes in the load strain data and faster rate of post-peak strength decay were recorded. However, columns with \( I_{10} \geq 8 \) exhibited ductile post-peak curves. Therefore, the authors based upon the evaluation of data concluded that for the regions of moderate seismicity desirable ductility could be achieved if \( I_{10} \) is more than 8. An analysis of present test data also reveals that the columns with \( I_{10} > 8 \) showed ductile post-peak load-strain curves with slower rate of strength decay. So if the same yardstick is applied on the present test data to achieve a moderate seismic resistance in terms of ductility, the following relationship could be derived after substituting the value of \( I_{10} \) as 8 in equation (6):

\[
k_c \rho_s f_{sh} / f'_c \geq 0.136
\]

The equation (7) can be used to design the critical

![Fig. 14 I₁₀ ductility index versus effective confinement index.](image-url)
hinge regions of high strength concrete columns. It may be mentioned here that equation (7) specifies the minimum effective confinement index or ratio for high strength concrete columns, which includes confinement effectiveness coefficient ($k_c < 1$) unlike equations (4) and (5) of ACI 318 provisions.

7. Assessment of existing stress-strain confinement models

The analysis of structural members requires an analytical model for the full stress-strain relationship of concrete in compression both in confined and unconfined states. The analytical models for confined normal strength concrete based on extensive experimental data are well established. These models, based on the test results of normal strength concrete columns, might be inadequate for high strength concrete columns, which possess a less ductile stress-strain behavior. So the normal strength concrete models if applied to high strength concrete shall overestimate the ductility. As a result a number of such models specifically for high strength concrete columns have also been proposed in the recent past. The following section provides an overview of these analytical confinement models that cover high strength concrete with strengths more than 60 MPa.

**Yong, Nour and Nawy (1988)**

Yong, Nour and Nawy proposed a model for rectilinearly confined high strength concrete columns. They tested 24 square prisms that were made of high strength concrete with a compressive strength ranging from 83.6 to 93.5 MPa, confined with square ties with yield strength of 496 MPa. The variables considered were volumetric ratio of lateral ties, concrete cover, and distribution of lateral steel. A three-part stress-strain relation was proposed to predict the constitutive behavior of confined high strength concrete.

**Bjerkli, Tomaszewicz and Jansen (1990)**

Bjerkli, Tomaszewicz and Jansen proposed a three-part stress-strain curve for high strength concrete columns for both circular and rectilinear cross sectional shapes based upon their test results. They tested a large number of plain and confined high strength concrete columns with compressive strength ranging from 65 to 115 MPa. Both cross sectional shapes i.e. cylinders (150 mm diameter and 500 mm high) and prisms (150x150x500mm and 300x500x2000mm) were included. The test specimens contained longitudinal steel but no concrete cover.

**Nagashima, Sugano, Kimura and Ichikawa (1992)**

Twenty-six prism specimens (225x716mm) of high strength concrete of strengths 59 and 118 MPa were tested by Nagashima, Sugano, Kimura and Ichikawa. Based on the test results a two-part stress-strain relationship was proposed for confined high strength concrete columns. The variables taken in to account were concrete strength, yield strength of lateral steel, tie configuration and spacing of lateral steel.

**Muguruma, Nishiyama and Watanabe (1993)**

Muguruma, Nishiyama and Watanabe proposed a three-part stress-strain model for confined concrete based on their previous studies. A wide range of concrete strength ranging from 40 to 140 MPa was covered. They tested small square specimens confined laterally by square helix hoops of different yield strengths and with various volumetric ratios. The yield strengths of the hoops ranged from 161 to 1353 MPa.

**Li, Park and Tanaka (1994)**

Li, Park and Tanaka proposed a three-part stress-strain model for confined high strength concrete based on their experimental results. Forty reinforced concrete short columns of both cylindrical (240x720 mm) and square (240x240x720mm) cross sectional shapes were tested. The main parameters were in place concrete strength (35.2 to 82.5 MPa) and lateral steel grade (445 and 1318 MPa) in addition to other parameters like spacing, volumetric ratio and configuration of lateral steel.

**Cusson and Paultre (1995)**

Cusson and Paultre developed a confinement model for high strength concrete on the basis of test results of 50 large-scale high strength concrete tied columns tested under concentric loading. Out of them, 30 HSC tied columns (235x235x1400 mm) were tested by authors themselves and 20 HSC tied columns (225x225x715 mm) were tested by Nagashima et al. (1992). The concrete compressive strengths of the specimens ranged from 60-120 MPa. The ties with yield strength from 400 to 800 MPa were used. The proposed model takes into account tie yield strength, tie configuration, transverse reinforcement ratio, tie spacing, and longitudinal reinforcement ratio. The two-part stress-strain relationship included separate expressions for ascending and descending parts.

**Razvi and Saatcioglu (1999)**

Razvi and Saatcioglu proposed a model for confined normal and high strength concrete columns using extensive test data of authors own test results as well as the experimental results of other research studies. This included the test results of nearly full size specimens of different shapes, sizes, reinforcement configurations, tie yield strength (400 to 1387 MPa) and concrete strengths (30 to 130 MPa). The parameters incorporated in the model were type, volumetric ratio, spacing, yield strength, and arrangement of transverse reinforcement, distribution and amount of longitudinal steel as well as concrete strength and section geometry. The two part stress-strain model proposed by the authors was in the form of the ascending parabolic branch up to peak and a linear descending branch up to 20% of the peak stress.

**Legeron and Paultre (2003)**

Legeron and Paultre proposed a stress-strain confinement model for normal and high strength concrete columns based on the large number of test results of circular, square and rectangular columns tested under various research studies that included the studies undertaken by
themselves and a number of other researchers. The concrete compressive strengths ranged from 20 to 140 MPa and tie yield strengths ranged from 300 to 1400 MPa. The model incorporates almost all the parameters of confinement. The stress strain relationship was basically same as proposed by Cusson and Paultre (1995), but the parameters of the model were recalibrated on the basis of large number of test data collected by the authors.

A critical review of the above models indicate that most of them have limited validity in terms of concrete strengths, column geometry, transverse reinforcement yield strength and loading conditions. With the exception of the models proposed by Razvi & Saatcioglu (1999); Legeron & Paultre (2003); Li et al. (1994) and Bjerkli et al. (1990), which cover both circular and rectilinear sections, all other models are applicable to only square or rectilinear shapes. The models proposed by Yong et al. (1988); Nagashima et al. (1992); Li et al. (1994) have limited applicability for wide concrete strength ranges. It has been proved experimentally in the present study and in many earlier studies (Cusson & Paultre 1994; Razvi & Saatcioglu 1996; Foster 1999) that for high strength concrete columns, lateral-confining ties may not yield when peak of confined concrete stress-strain is reached. But, most of the models use lateral steel yield strength to calculate lateral confining pressure at peak-confined strength. Only Cusson & Paultre (1995); Razvi & Saatcioglu (1999) and Legeron & Paultre (2003) have incorporated this fact into their respective models by proposing procedures to calculate actual tie stress at peak of confined stress-strain response. Li et al. (1994) has also accounted for this indirectly by suggesting different expression for confined strength when higher grades of lateral steel are to be used, but no explicit expression for finding the actual tie stress at peak was proposed. There is hardly any model, which takes into account all the loading conditions namely monotonic, cyclic, strain rates and eccentric loading into account.

To investigate the relative performance of the various proposed analytical models (as listed above) with regards to their capabilities of predicting the experimentally observed stress-strain profile, stress-strain curves of the test specimens of present study were compared with the ones predicted by the various models. It may again be mentioned here that all the eight confinement models of the study are applicable to square sections whereas only four namely Razvi & Saatcioglu (1999); Legeron & Paultre (2003); Li et al. (1994) and Bjerkli et al. (1990) can be applied to circular columns. Figures 15 to 22 illustrate the comparisons of the experimental and predicted stress-strain curves of a few representative test specimens only. The comparisons for circular columns indicate that Razvi & Saatcioglu (1999) and Li et al. (1994) models consistently overestimate the actual test behavior, while Bjerkli et al. (1990) model underestimates the test curves. Legeron & Paultre (2003) model closely follows the experimental stress-strain curves though, with a slight overestimation, for most of the circular specimens. In the case of square columns Razvi & Saatcioglu (1999); Li et al. (1994); Bjerkli et al. (1990) and Yong et al. (1988) models always overestimated the experimental stress-strain curves by a considerable margin for all the specimens. The model proposed by Muguruma et al. (1993) produced considerably steeper stress-strain curves for all the specimens that were far below the actual test curves. Whereas, the models proposed by Cusson & Paultre (1995) and Nagashima et al. (1992) continuously underestimated the test curves though; the predictions were quite close in the latter case for few specimens. As in the case of circular columns, Legeron & Paultre (2003) model was also able to produce close predictions of stress-strain behavior for almost all the square columns except for few cases like SH specimen where it slightly overestimated the experimental stress-strain curve. The ability of Legeron & Paultre (2003) model to consistently follow the actual test behavior for both circular and square columns undoubtedly proves its superiority over the other models of the study. Therefore, the present study concludes that this model can be employed to predict the uniaxial response of high strength concrete columns with a reasonable degree of accuracy.
8. Conclusions

This paper presents the results of spirally confined circular and tie confined square high strength concrete columns subjected to concentric axial compression. The test variables are volumetric ratio and spacing of transverse reinforcement, yield strength of transverse reinforcement, longitudinal reinforcement ratio, lateral steel configuration.
tion, shape of cross section and concrete compressive strength. A comparative study of existing stress-strain models for high strength concrete columns is also reported. The following conclusions can be made based upon this study.

1. High strength concrete columns show brittle behavior unless the columns are confined with sufficient lateral reinforcement that can provide adequately high lateral confining pressure. A consistent decrease in strength enhancement and deformability of columns is observed with increasing concrete strength. Therefore, a higher degree of confinement is required in columns with higher concrete strength than in a column with lower concrete strength to achieve similar advantages.

2. High strength concrete columns suffer from the problem of premature cover spalling which reduces the column load carrying capacity to even less than unconfined strength if sufficient confinement is not provided. The post spalling behavior depends very much on the degree and efficiency of confinement.

3. Among the test variables studied, volumetric ratio and spacing of lateral steel has a more pronounced effect on the behavior of confined columns than the other parameters like yield strength of lateral steel, longitudinal steel ratio and configuration of lateral steel though, improvement in each of the variables considered, translated into enhancements in strength and ductility. However, increasing the yield strength of lateral steel seems to give benefits only when a column has high volumetric ratio and efficient arrangement of lateral steel.

4. The ductility of columns is shown to be dependent on effective confinement index \( k_s \rho_s f_{yh} / f_c \). It is concluded that for regions of moderate seismicity, the critical hinge regions of high strength concrete columns should have effective confinement index of more than 0.136 to achieve adequate ductility.

5. A comparative study is undertaken to evaluate the capabilities of the various confinement models of high strength concrete columns to predict the actual experimental behavior. The study indicated that almost all the models are able to estimate correctly ascending part of stress-strain curve. But, there are wide variations in the prediction of the post-peak part of stress-strain curves, with a few models underestimating and a few overestimating the test behavior except the Legeron and Paultre (2003) model, which consistently predicted experimental results with least amount of discrepancy. Therefore, the present study concludes that the Legeron and Paultre (2003) model can be used to analytically predict the uniaxial response of high strength concrete columns with a reasonable degree of accuracy.

**Notations**

\[ A_{sc} = \text{core concrete area} \]
\[ A_e = \text{sectional area of longitudinal reinforcement} \]
\[ E_c = \text{tangent modulus of elasticity of concrete} \]
\[ f_{c} = \text{cylinder compressive strength of concrete} \]
\[ f_{sh} = \text{yield strength of tie steel} \]
\[ f_{y} = \text{yield strength of longitudinal steel} \]
\[ f_{hc} = \text{stress in lateral steel at peak stress of confined concrete} \]
\[ P_{max} = \text{maximum applied load on the column} \]
\[ P_c = \text{concrete load corresponding to first peak} \]
\[ P_{cc} = \text{peak confined concrete load (corresponding to second peak)} \]
\[ P_{oc} = \text{theoretical load carrying capacity of column including longitudinal bars} \]
\[ P_{oc'} = \text{theoretical capacity of concrete in the column} \]
\[ P_{occ} = \text{theoretical capacity of concrete core} \]
\[ s = \text{spacing of lateral steel} \]
\[ \rho_s = \text{volumetric ratio of lateral steel} \]
\[ \rho_t = \text{volumetric ratio of longitudinal steel} \]
\[ \varepsilon_{uc} = \text{unconfined strain of concrete column} \]
\[ \varepsilon_{cc} = \text{axial strain at peak confined load, } P_{cc} \]
\[ \varepsilon_{c50}\text{a} = \text{axial strain at which the stress drops to 50% of peak in unconfined concrete} \]
\[ \varepsilon_{c50}\text{b} = \text{axial strain corresponding to the first peak, } P_c \]
\[ \varepsilon_{c50} = \text{axial strain at peak confined load, } P_{cc} \]
\[ \varepsilon_{c50}\text{b} = \text{axial strain at which the stress drops to 50% of peak in confined concrete} \]
\[ \varepsilon_{y} = \text{yield strain of reinforcement} \]

**References**


