DESIGN AND DETAILING OF HIGH STRENGTH CONCRETE COLUMNS

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UNICIV REPORT No. R-375 MARCH 1999
THE UNIVERSITY OF NEW SOUTH WALES
SYDNEY 2052 AUSTRALIA

ISBN: 85841 342 6
Design and Detailing of High Strength Concrete Columns

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High strength concrete, columns, cover spalling, strength, ductility, tie spacing, design.

In this report the behaviour, strength and ductility of high strength concrete columns is reviewed. Particular emphasis is given to the issue of cover spalling and its influence on strength and ductility. Cover spalling in columns occurs after cracking along the interface of the cover and the core is initiated by a triaxial stress condition. This cracking occurs before the concrete reaches its uniaxial compressive strength and leads to the condition of early cover spalling. If sufficient ties are provided, and detailed to provide efficient and effective confinement in the localised region, the column can maintain a sufficient level of ductility. It is shown that while using high strength ties does not increase the strength of high strength concrete columns, they can improve the ductility. Design guidelines are suggested for strength and for maximum tie spacings and tie arrangements.

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1. Introduction

The increase in strength and ductility of normal strength concrete columns afforded by well-detailed lateral confinement reinforcement is well documented (Sheikh and Uzumeri (1), Mander et al. (2), Saatcioglu and Razvi, (3)). Questions have been posed, however, as to whether or not similar detailing is suitable for high strength concrete (HSC) columns and, if not, what amount of confinement reinforcement is necessary to obtain a satisfactory level of ductility in high strength concrete columns (Razvi and Saatcioglu (4), Foster and Attard (5), Pessiki and Pieroni (6)). Existing code provisions for minimum amounts of lateral reinforcement are based on experiences with normal strength concrete, however, the influence of concrete strength on effectiveness of confinement needs to be taken into consideration.

Experimental studies on high strength concrete columns under concentric axial loading have shown that the strength is affected by spalling of the cover and the inability of the concrete core to carry increased loads after the cover is shed. Various theories have been postulated for this observed behaviour, including buckling of the cover shell (Paultre et al. (7)) and restrained shrinkage in the cover shell combined with shrinkage of the high strength concrete around the reinforcing steel (Sundaraj and Sheikh (8), Collins et al. (9)). In a recent paper by Foster et al. (10) it was shown that cracking occurs at the cover-core interface as a result of the triaxial stress condition induced by confinement of the core. It was also shown that this interface cracking occurs irrespective of the concrete’s strength.

Experimental studies (Cusson and Paultre (11), Foster and Attard (12), Razvi and Saatcioglu (13)) have shown that at the point of cover spalling, the tie steel is not at yield. This leads to the question as to what advantages can high strength ties provide to the behaviour of a column section given that at the point of cover spalling the confinement is
limited by the tie strain. This issue is addressed in this paper together with design models for the strength and ductility requirements of HSC columns.

2. The Behaviour of Concentrically Loaded HSC Columns

While all columns designed to AS3600 (14) are required to incorporate a minimum eccentricity of 0.05D, where D is the overall depth of the section, much insight is gained by reviewing the behaviour of concentrically loaded columns. The load versus deflection behaviour of a HSC column under load is shown in Figure 1. As the column is loaded the displacement increases in a near linear manner. The proportional limit, as shown in Figure 1, is dependent on the strength of the concrete and can be as high as 80 percent of the spalling load for very high strength concrete. Beyond the proportional limit, micro-cracking within the core gives the increased lateral expansion necessary to activate the confining pressure provided by the tie reinforcement. High strength concrete has a higher proportional limit than normal strength concrete leading to a lower expansion of the core at the peak load and, thus, lower tie strains at the spalling load.

![Figure 1 - Load-deflection behaviour of HSC columns](image-url)
While for HSC columns the strains in the ties at the peak load are small, they are not insignificant. In a recent paper by Foster et al. (10), it was shown that the initiation of cover spalling is an inevitable consequence of the triaxial stress state in the cover. Further, the mechanism behind the initiation of the separation exists equally for conventional strength as for high strength concretes and within the constraint $\sigma_3 < f_{cp}$; where $\sigma_3$ is the axial compression stress in the concrete and $f_{cp}$ is the uniaxial strength of the in-place concrete. In providing ties to constrain the core tension stresses are set up at the cover-core interface, as shown in Figure 2. The greater the amount of confinement provided to the core, the greater the tension stress at the cover-core interface due to the higher restraint provided to the core against Poisson’s growth. Figure 3 shows the load path for the stresses at a point at the cover-core interface for a circular column (although the same applies for the ease of rectangular sections). The point is in a triaxial tension-compression-compression stress state, with the tension in the radial direction. The circumferential compression is small, relative to the axial compression, and has only a small influence on the behaviour. Cracking occurs at the interface of the cover and the core once the internal stresses reach the failure surface. The experimental and numerical evidence (Liu et al. (15)) suggests that this is at an axial stress of the order of $0.85f_{cp}$ and at a tensile stress of the order of $0.26f_t$, where $f_t$ is the uniaxial tension strength. Once cracking occurs at the cover-core interface the cover is free to buckle or spall away from the section. This process is possibly accelerated by dilation of the longitudinal steel after yielding.

When the cover spalls the core is required to take the additional load shed by the cover. In tests on HSC columns by Cusson and Paultre (11) it was observed that the cover spalled suddenly causing a drop in load of 10-15 percent. In the finite element studies of Liu et al. (15) it was shown that the drop in load is a function of the ratio of the area of the cover to
the gross cross sectional area. In poorly confined columns, or columns with a high cover area to gross area ratio, the dynamic effects set-up by the spalling of the cover leads to a sudden and catastrophic failure. In well-detailed sections expansion of the core activates the confinement provided by the ties and failure is arrested. At this point the column may again carry increased load to a second peak. The second peak load may be higher or lower than
spalling load and depends on the volumetric ratio of the ties, the yield strength of the ties, the strength of the concrete and the efficiency of the tie arrangements. The second peak load corresponds to yielding of the ties and the axial strain at the peak load is a function of the tie yield strength.

Sheikh and Uzumeri (16) proposed a procedure to determine the capacity of the confined core based on a conceptual model in which it was postulated that the area of the effectively confined concrete in a column is less than the core area (refer Figure 4). The effectively confined concrete is assumed to arch between the points where the lateral steel exerts a confining pressure on the concrete. In the case of rectilinear lateral reinforcement, the area of effectively confined core is less than the core area even at the tie level and is further reduced away from the tie level. In the case of circular or spiral ties, the reduction of the core area to an effectively confined area takes place only along the longitudinal axis of the column. Mander et al. (2), Saatcioglu and Razvi (3) and Cusson and Paultre (17) further refined the concept advanced by Sheikh and Uzumeri. In these studies it was hypothesised that the confining pressure on the core dissipates between ties and away from longitudinal bars with the full confining pressure acting over a reduced or effective core area, $A_{eff}$. The core area, $A_{core}$, is normally defined by the enclosed area inside the perimeter of the center lines of the outer ring of spirals or ties. The effective core area is given by

$$A_{eff} = k_e A_{core}$$  \hspace{1cm} (1)

where $k_e$ is the confinement effectiveness coefficient ($k_e \leq 1$). The confining pressure is calculated by assuming the ties to have yielded and the equilibrating stresses on the core to be uniformly distributed. Examples of calculating the confining stress ($f_r$) for some common sections are shown in Figure 5.
Figure 4 - Effectively confined area in tied concrete columns; a) square and b) circular sections; c) 3D view of a square column.

Figure 5 - Calculation of confining pressures for some common section types.
For circular columns with tie or spiral reinforcement, assuming a parabolic arch between the
ties with a 45 degree tangent slope following the concept advanced by Sheikh and
Uzumeri (16), the confinement effectiveness factor is

\[ k_c = \left(1 - \frac{s^*}{2d_s}\right)^2 \]  

(2)

where \( s^* \) is the clear spacing between the ties or spirals as used by Mander et al. (2) and \( d_s \) is
the diameter of the tie or spiral reinforcement. For square or rectangular sections a modified
form of the Sheikh and Uzumeri model is used with the confinement effectiveness parameter
given by

\[ k_c = \left(1 - \frac{1}{\alpha A_{\text{core}}} \sum_{i=1}^{n} w_i^2 \right) \times \left(1 - \frac{s^*}{2b_c}\right) \times \left(1 - \frac{s^*}{2d_c}\right) \]  

(3)

where \( w_i \) is the \( i \)th clear distance between adjacent tied longitudinal bars, \( b_c \) and \( d_c \) are the
core dimensions to the centreline of the ties across the width and depth of the section,
\( A_{\text{core}} = b_c d_c \), \( n \) is the number of spaces between tied longitudinal bars and \( \alpha \) is constant
which is equal to 6 if the arches of the effectively confined concrete are assumed to be
parabolic and have an initial tangent of 45 degrees.

The core strength \( f_o \) can be obtained using a modified form of the Richart et al. (18)
equation

\[ f_o = f_{cp} + k_c f_{tr} \]  

(4)

where \( f_{cp} \) is the unconfined in-situ concrete strength and \( C \) is a confinement parameter. For
the analyses that follow, the confinement parameter was taken as \( C = 4 \) for \( f'_c \leq 80 \text{ MPa} \) and
C = 3 for $f'_c > 80 \text{ MPa}$, as recommended by the FIP/CEB (19). The capacity of the confined core including the longitudinal reinforcement is then given by

$$P_{\text{core}} = f_o (A_{\text{core}} - A_{st}) + A_{st} f_{sy}$$

(5)

where $A_{st}$ is the area of longitudinal tension reinforcement and $f_{sy}$ is the yield strength of the longitudinal reinforcement.

In Figure 6a, the data of Sheikh and Uzumeri (1), Toklucu and Sheikh (20), Cusson and Paultre (11) and Razvi and Saatcioglu (13) has been normalised against the squash load and plotted against the cylinder strength. In Figure 6a, the squash load $P_o$ is calculated as

$$P_o = k_3 f'_c A_c + A_{st} f_{sy}$$

(6)

where $f'_c$ is the measured cylinder strength, $A_c$ is the gross area of concrete in the section, and $k_3$ is a factor to account for the difference in the cylinder strength to that of the in-situ concrete ($k_3 = f_{cp} / f'_c$). Razvi and Saatcioglu (13) measured $k_3 \approx 0.9$ and this value was taken for all specimens compared in Figure 6. Figure 6a shows that for cylinder strengths greater than 80 MPa the peak load is approximately 0.85 times the squash load calculated by Eq. (6). From the evidence presented, the spalling load can be taken as

$$P_{\text{spall}} = 0.85 k_3 f'_c A_c + A_{st} f_{sy}$$

(7)

and the maximum capacity of the section is then obtained from

$$P_{\text{max}} = \max(P_{\text{spall}}, P_{\text{core}})$$

(8)
Figure 6 - Comparisons between experimental data normalised against the squash load for conventional and HSC columns loaded in concentric compression.
In Figure 6b, the experimentally obtained peak loads are normalised against the theoretical capacity including the longitudinal steel, as calculated using Eqs. 5, 7 and 8. Figure 6b indicates that for all strength concrete the cover is ineffective after the initiation of the cover-core cracking. The peak load then becomes the greater of the spalling load (taken as the load corresponding to the commencement of cover-core cracking) and the capacity of the effectively confined core (as given by Eq. 8). Comparison of Figures 6a and 6b, however, indicate that the spalling loads are only of concern for columns with $f'_c \geq 60$ MPa.

3. The Effect of Load Eccentricity Early Cover Spalling

Ibrhim and MacGregor (21) analysed data from a number of experimental studies for concentrically, and eccentrically, loaded columns. Two of their figures are reproduced in Figure 7. Comparing Figures 7a and 7b suggests that the early cover spalling effect is not significant for eccentrically loaded sections. Once sufficient bending is introduced such that $d_n \leq D$, where $d_n$ is the depth of the neutral axis measured from the extreme compression fibre, no reduction in load occurs due to early spalling of the cover. The reasons for this remain unclear.

Figure 7a shows that $k_3 = 0.85$ is a reasonable lower bound for eccentrically loaded columns. For concentrically loaded columns, Figure 7b shows a lower bound of $k_3k_4 = 0.72$, where $k_4$ is introduced to account for the effect of cover spalling on the squash load.
Figure 7 - $k_3$ versus concrete strength for (a) eccentrically and (b) concentrically loaded specimens (after Ibrahim and MacGregor, 21)
4. The Rectangular Stress Block - Theory

For any given stress distribution through a concrete beam-column section, an equivalent rectangular distribution of stresses can be determined such that

1. the total volume under the true and equivalent rectangular stress blocks are equal, and
2. the centroid of the equivalent rectangular stress block lies at the centroid of the true stress block.

The relationship between the true and rectangular stress blocks is shown in Figure 8. Point 1 is necessary to ensure equilibrium of forces and point 2 for moment equilibrium. Three parameters are required to define the equivalent rectangular stress block (refer Figure 8), namely $k_1$, $k_3$ and $\gamma$ (where $\gamma = 2k_2$). For a given stress-strain relationship, and assuming that plane sections remain plane, $k_2$ (and therefore $\gamma$) is determined such that the location of the centroids of the true and rectangular stress blocks coincide. Equilibrium of forces is then applied to determine $k_1$ such that the volume under the true and rectangular stress blocks are equal. The $k_3$ parameter represents the ratio of the strength of the in-situ concrete to that of the standard cylinder and is unvaried for both the true and equivalent rectangular stress blocks. The $k_3$ factor can be directly determined from tests on concentrically loaded specimens.

![Figure 8 - Equivalent Rectangular Stress Block.](image-url)
It is generally agreed that as the concrete strength increases the generalised stress-strain curve becomes increasingly triangular. For the case of a triangular stress-strain relationship, again assuming that plane sections remain plane, equilibrium gives $\gamma = 2/3$ and $k_1 = 3/4$. These limits are absolute if a perfectly rational model is to be established and, thus, for all strengths of concrete

$$\gamma \geq 2/3 \quad k_1 \geq 3/4$$  \hspace{1cm} (9)

The squash load of a short concrete column is given by

$$P_o = k_3k_4f_c'(A_g - A_s) + A_s f_{sy}$$  \hspace{1cm} (10)

where $A_g$ is the gross area of the section, $A_s$ is the area of steel through the cross section, $f_c'$ the cylinder strength and $f_{sy}$ the yield strength of the longitudinal reinforcement. As discussed above, the parameter $k_4$ is introduced into Eq. 10 to account for premature spalling of the cover shell.

A number of stress blocks have been proposed for the design of HSC beam-columns, including those that appear in the design codes CSA-94 (22) and NZS-95 (23). The parameters developed for these and other models such as those presented by Ibrahim and MacGregor (21), and Attard and Stewart (24) were developed from a pool of experimental data which included concentrically loaded HSC column specimens. In these tests it was observed that the cover shell failed “prematurely” (as measured against expectations developed from theories developed for conventional strength concrete columns). The parameters for the different stress block models developed are compared in Table 1, including the parameters for a modified AS3600 model presented in this report.
Table 1 - Rectangular stress block parameters for HSC ($f'_c$ in MPa).

<table>
<thead>
<tr>
<th>Reference</th>
<th>$\alpha_1 = k_1 k_3$</th>
<th>$\gamma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CSA-A23.3-94 (22)</td>
<td>$0.85 - 0.0015 f'_c \geq 0.67$</td>
<td>$0.97 - 0.0025 f'_c \geq 0.67$</td>
</tr>
<tr>
<td>NZS3101-1995 (23)</td>
<td>$1.07 - 0.004 f'_c$</td>
<td>$1.09 - 0.008 f'_c$</td>
</tr>
<tr>
<td></td>
<td>$0.75 \leq \alpha_1 \leq 0.85$</td>
<td>$0.65 \leq \gamma \leq 0.85$</td>
</tr>
<tr>
<td>Ibrahim and Macgregor (21)</td>
<td>$0.85 - 0.00125 f'_c \geq 0.725$</td>
<td>$0.95 - 0.0025 f'_c$</td>
</tr>
<tr>
<td>Attard and Stewart (24)</td>
<td>$1.29 (f'_c)^{0.01} \geq 0.71$</td>
<td>$\gamma = 1.09 (f'_c)^{-0.091} \geq 0.67$</td>
</tr>
<tr>
<td>Modified AS3600 (refer Section 5)</td>
<td>$1.02 - 0.0032 f'_c$</td>
<td>$1.09 - 0.008 f'_c$</td>
</tr>
<tr>
<td></td>
<td>$0.64 \leq \alpha_1 \leq 0.85$</td>
<td>$0.67 \leq \gamma \leq 0.85$</td>
</tr>
</tbody>
</table>

The Canadian code model (22) was calibrated against a range of experimental data including data for concentrically loaded HSC columns failing in concentric compression. The squash load given by CSA-94 is

$$P_o = \alpha_1 f'_c (A_g - A_s) + A_s f_{sy}$$

(11)

with

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

(12)

The depth of the stress block, for sections in combined bending and compression, is given by

$$\lambda = 0.97 - 0.0025 f'_c \geq 0.67$$

(13)
Comparing Eqs. 10 and 11 gives \( \alpha_1 = k_3 k_4 \). With the rectangular stress block (RSB) shown in Figure 8 it can then be concluded that Eq. 11 and the RSB are consistent provided \( k_1 \approx k_4 \).

Further, noting that in CSA-94 \( k_3 = 0.9 \), then the CSA-94 formulation leads to

\[
k_1 = k_4 = 0.94 - 0.00167 f'_{c_{cm}} \geq 0.75
\]

which satisfies the limits given by Eq. 9. A similar analogy can be made for the models given by NZS-95 (23) and by Ibrahim and MacGregor (21).

Attard and Stewart (24) proposed a set of RSB parameters based on a probabilistic model. The model varies from previous models in that the interaction curve gives the mean section strength. Thus, if all else was equal, 50 percent of the experimental data would fall each side of the interaction curve. This differs from other models where the characteristic cylinder strength in the field is substituted for the mean cylinder strength used in the laboratory. Attard and Stewart use the squash load given by Eq. 10, with

\[
k_3 = 1.05 - 0.0009 f_{cm}
\]

\[
k_3 k_4 = 0.92 - 0.0009 f_{cm}
\]

and where \( f_{cm} \) is the mean cylinder strength in MPa. The mean cylinder strength is taken as

\[
f_{cm} = f'_{c} + 7.5 \text{ MPa}
\]

Like previous reviews of the data, the calibration of the Attard and Stewart spalling factor does not give any insight into the mechanics of cover spalling. The generalisation of the Attard and Stewart spalling factor to other than rectangular sections and for sections with
various cover/core ratios is yet to be demonstrated. This is discussed further in Section 6 below.

The height of the Attard and Stewart stress block (refer Figure 8) for eccentrically loaded sections is given by

\[ k_1 k_3 = 1.29 (f'_c)^{-0.1} \geq 0.71 \]  \hspace{1cm} (18)

with a depth of

\[ \gamma = 1.09 (f'_c)^{-0.091} \geq 0.67 \]  \hspace{1cm} (19)

Reviewing the data Attard and Stewart used in developing Eq. 15, a constant value of \( k_3 = 0.95 \) represents the data reasonably well. Substituting \( k_3 = 0.95 \) into Eq 18 gives

\[ k_1 = 1.36 (f'_c)^{-0.1} \geq 0.75 \]  \hspace{1cm} (20)

The limit on Eq. 20, however, is not reached until \( f'_c = 380 \text{ MPa} \).

5. A Modified RSB Model for AS3600.

Much of the work on the development of a rational RSB model for inclusion in the Australian building code for concrete structures, AS3600 (14), has started with the premise that a new relationship needs to be determined for all strengths of concrete. This requires calibration of the models against existing conventional strength concrete data. In this paper a modified AS3600 RSB model is developed starting with the premise that the existing AS3600 model performs well over the range of concrete strengths for which it is calibrated; and, presupposing that a minimalist approach is desirable. It is also noted that the current AS3600
model meets all the boundary constraints for a rational model with \( k_3 = 0.85 \) and \( k_4 = 1.0 \).

The requirements then are for a rational model that meets the known boundary constraints (implied by Eq. 9) and should be valid for concrete strengths beyond that currently produced commercially.

In Figure 6 data is plotted for a number of reinforced columns tested in concentric compression for a wide range of concrete strengths. The data indicates that inclusion of a cover spalling mechanism into a design model becomes more important as the concrete cylinder strength is increased beyond 60 MPa. For reinforced columns cast with concrete strengths of less than 60 MPa other mechanisms, such as the increase in strength due to confinement, compensate for the reduction in section capacity due to the onset of spalling.

The effects on capacity may be further disguised by the better ductility of conventional strength concrete compared to that of HSC. The squash load is taken as the maximum capacity of the section under concentric loading and is the greater of the capacity of the section at the point of cover spalling or the capacity of the confined core (see Section 2).

Figure 6 indicates that \( k_4 \) can be given by

\[
k_4 = 1.2 - 0.0038 f'_c \\
0.85 \leq k_4 \leq 1.0
\]  

In the squash load equation in AS3600 (14) \( k_3 = 0.85 \) and \( k_4 = 1.0 \). Therefore with the RSB parameters used in AS3600-1994 it can be interpreted that \( k_1 = 1.0 \) and

\[
2k_2 = 1.09 - 0.008 f'_c \\
0.67 \leq 2k_2 \leq 0.85
\]

The triangular stress block limit is reached at \( f'_c = 54.2 \) MPa. The axial load carried by a section is given by
\[ C = k_1 k_2 (k_3 f_c') d_n b + \sum_{i=1}^{n} (A_s f_s) d_i \]  

where \( d_n \) is the distance from the extreme compression fibre to the neutral axis, \( A_s \) is the area of reinforcing steel at level \( i \), \( f_s \) is the stress in the steel at level \( i \) and \( n \) is the number of layers of reinforcing steel. The moment carried by a symmetric section is

\[ M = k_1 k_2 (1-k_2) (k_3 f_c') d_n^2 b + \sum_{i=1}^{n} (A_s f_s) d_i \]  

where \( d_i \) is the distance from the plastic centroid of the section to the reinforcing steel at level \( i \). The extension to HSC can be obtained by manipulating \( k_1 \) such that in the limit the stress block area factor \( k_1 k_2 = 0.5 \). The \( k_1 k_2 \) factor is the multiplication factor necessary to convert the area under the true stress block to an equivalent rectangle. The lower limit of \( k_1 k_2 = 0.5 \) represents the conversion of a triangle to an equivalent rectangle. Of equal importance is the factor \( k_1 k_2 (1-k_2) \) which represents the moment carried by the concrete.

Taking \( 2k_2 \) given by Eq. 14, and setting the triangular stress block limits at \( f_c' = 120 \text{ MPa} \) (which appears reasonable when reviewing the stress strain curves obtained on standard cylinders) leads to

\[ k_1 = 1.2 - 0.0038 f_c' \]
\[ 0.75 \leq k_1 \leq 1.0 \]  

The stress block can now be defined for the case where the neutral axis lies within the section where \( \gamma = 2k_2 \) is given by Eq. 22, and

\[ \alpha_1 = k_1 k_3 = 1.02 - 0.0032 f_c' \]
\[ 0.64 \leq \alpha_1 \leq 0.85 \]  

\[ (23) \]
The squash load is given by Eq. 10 with

\[
\alpha_2 = k_3k_4 = 1.02 - 0.0032f' c \\
0.72 \leq \alpha_2 \leq 0.85
\] (27)

A straight line is used to join the squash load given by Eqs. 10 and 27 to the point on the interaction curve defined by Eqs. 22 and 26 with \( c = D \), where \( D \) is the depth of section. The axial compression-bending interaction diagram defined by Eqs. 10, 22, 26 and 27 give identical curves to the current AS3600 for \( f' c \leq 54.2 \text{ MPa} \) and obey the triangular stress block boundary limits set out in Eq. 9 for the extension to high, and very high, strength concretes.

6. Comparison of RSB Models

In Figure 9a the factor defining the area under the rectangular stress block \( k_1 2k_2 \) is plotted for a number of RSB models. The change in slope in the NZS-95 and in the proposed modified AS3600 model, for increasing concrete strengths, is due to reaching the limits in \( k_2 \) and \( k_1 \), respectively. Thus while the limits of Eq. 9 are satisfied in that each parameter approaches that for the triangular stress block, they do so at different concrete strengths. In the Canadian code (CSA-94) model, the \( k_1 \) and \( k_2 \) limits are reached simultaneously at \( f' c = 120 \text{ MPa} \). In the model proposed by Attard and Stewart, the triangular stress block limit is not reached until \( f' c = 230 \text{ MPa} \), although the limits given by Eqs. 18 and 19 are not reached until \( f' c = 390 \text{ MPa} \) and \( f' c = 210 \text{ MPa} \), respectively. Figure 9b compares the moment factor \( k_1 2k_2(1 - k_2) \) for the CSA-94 (22), NZS-95 (23), Attard and Stewart (24) and modified AS3600 model. The proposed modified AS3600 model compares favourably with the CSA-94 and NZS-95 models up to \( f' c = 80 \text{ MPa} \). At \( f' c = 80 \text{ MPa} \) the NZS-95 model hits
its limit, a limit which is significantly higher than that defined by a triangular stress block. While the CSA-94 model reaches its limit at $f'_c = 120 \text{MPa}$, the calibration of the CSA-94 model was limited to $f'_c \leq 80 \text{MPa}$ (22).

In Figures 10, 11 and 12 comparisons are made for the various stress blocks for a series of 87 MPa 250 mm by 150 mm rectangular columns and 74 MPa and 92 MPa 150 mm by 150 mm square columns, respectively, tested by Foster and Attard (5, 12). Comparisons are made with $k_3 = 0.85$ and $k_3 = 1.0$ where it is shown that all the models discussed lead to similar results. The reasons for the closeness of the RSB models compared in Figures 10 to 12 is explained by a convergence of the area and moment factors shown in Figure 9 for concrete strengths in the range $f'_c = 70 \sim 100 \text{MPa}$. Larger differences can be expected between the CSA-94, NZ-95 and modified AS3600 models for concrete strengths greater than 100 MPa, and for the Attard and Stewart model for concrete strengths of less than 50 MPa and greater than 100 MPa.

In Figure 13, the modified AS3600 model is compared with 36 eccentrically loaded columns tested Lloyd and Rangan (25). A good correlation is shown between the test data and the model predictions.

7. Design for Ductility

Ductility is an important issue when it comes to the detailing of all concrete members, but is of particular importance for of HSC columns due to the brittle nature of the concrete. Again it is worth reviewing the behaviour of columns under concentric loading as this represents the extreme condition. Ductility in columns is derived from confinement provided to the core and, thus, is a function of the yield strength of the ties, the concrete strength, the volumetric
ratio of tie reinforcement and the arrangement of the ties. Liu et. al. (15) showed that while increasing the strength of the ties does not improve the load capacity of a HSC column, it does give an improvement in the ductility.

One measure of ductility is given by the $I_{10}$ index, where $I_{10}$ is calculated similar to that set out in ASTM C1018 (26) for the measurement of toughness. The $I_{10}$ parameter is the area under the load versus strain curve at a strain of 5.5 times the yield strain, relative 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 on the ascending curve of $0.75P_y$ (see Foster and Attard (5)). The area under the load versus strain curve up to 5.5 times the yield strain is chosen such that for a perfectly elasto-plastic material $I_{10} = 10$ while for a perfectly elastic-brittle material $I_{10} = 1$. 
Figure 9 - Comparison of (a) the area factor; and (b) the moment factor for various RSB models.
Figure 10 - Comparisons of various RSB models with 250 mm x 150 mm rectangular columns tested by Foster and Attard (2) (a) $k_3 = 0.85$; and (b) $k_3 = 1.0$. 
Figure 11 - Comparisons of various RSB models with 74 MPa 150 mm x 150 mm columns tested by Foster and Attard (10) (a) \( k_3 = 0.85 \); and (b) \( k_3 = 1.0 \).
Figure 12 - Comparisons of various RSB models with 92 MPa 150 mm x 150 mm columns tested by Foster and Attard (10) (a) $k_3 = 0.85$; and (b) $k_3 = 1.0$. 

- $f_{cm} = 92$ MPa
- $k_3 = 0.85$
- $f_{cm} = 92$ MPa
- $k_3 = 1.0$
Figure 13 - Comparison of peak axial loads versus predicted axial load using the modified ACI-318 model for 36 columns tested by Lloyd and Rangan (25).

A number of authors (4, 27, 28, 29) have indicated that ductility is a function of the confinement parameter $\rho_s f_{yt}/f_c'$ where $\rho_s$ is the lateral reinforcement volumetric ratio, $f_{yt}$ is the yield strength of the tie reinforcement and $f_c'$ is the concrete strength. Razvi and Saatcioglu (13) suggested that this parameter be multiplied by a second parameter representing the efficiency of the tie reinforcement arrangement. In Figure 14, the $I_{10}$ ductility index for 40 columns tested by Razvi and Saatcioglu (13) is plotted against $k_e \rho_s f_{yt}/f_c'$. The hollow data markers represent the results of columns where large changes in load were recorded. For these columns an exact value for $I_{10}$ cannot be obtained, but upper and lower bounds can be calculated. The data point is plotted at the average of the upper and
Figure 14 – I_{10} ductility parameter versus the effective confinement ratio

lower bound and have a possible error in $I_{10}$ of approximately ±0.8. From Figure 14, a relationship between the ductility index and the effective confinement parameter is obtained, that is

$$I_{10} = 1.9 \ln \left(1000 k_e \rho_s f_{yt}/f_c' \right)$$  \hspace{1cm} (28)$$

or

$$k_e \rho_s f_{yt}/f_c' = \frac{1.7 I_{10}}{1000}$$  \hspace{1cm} (29)$$

While more data is needed for columns with low amounts of confining steel, some important design conclusions can be determined from the data available. Specimens CC-17 and CC-18 (refer Figure 14) had no longitudinal reinforcement. Thus, it is concluded that the arrangement of the longitudinal reinforcement is an important parameter in obtaining ductility, even in circular columns. In column CS-5, 1000 MPa ties were used to increase the confinement ratio, however, the tie spacing was 120 mm ($D/2$). For most of the columns with
$I_{10} < 8$ sudden changes in the load-strain data were recorded. In no case where the tie spacing was greater than $(D/2.5)$, or the distance between tied longitudinal bars was greater than $(D/2.5)$, did the columns perform adequately with respect to ductility, regardless of the strength of the ties. For regions of low to moderate seismicity satisfactory ductility may be obtained with $I_{10} \geq 8$. Therefore, from Eq. 29, adequate ductility is achieved provided that the effective confinement parameter is $k_e \rho_s f_m / f'_c \geq 0.07$ and that the tie spacing, and the spacing between tied longitudinal bars, is not greater than $(D/2.5)$.

Applying the above principles for the hinge region of a 600 mm square column cast with 80 MPa concrete, having a uniform arrangement of 12 Y36 bars longitudinally, 40 mm of cover and 4 legged 16 mm diameter ties in each direction, requires 500 MPa ties at 100 mm, 700 MPa ties at 140 mm or 1000 MPa ties at 180 mm centres to achieve a ductility level of $I_{10} = 8$. Outside the localised zones the tie spacing may be increased but should not exceed the lesser of $D/2$ and 300 mm.

8. Conclusions

In this paper, research on the behaviour of high strength concrete columns under combined axial load and bending has been reviewed towards the synthesis of a rational rectangular stress block model for the design of conventional and high strength concrete columns. Tests have shown that concentrically loaded columns fabricated with HSC fail at loads lower than that their theoretical squash loads, where the squash load is calculated from the uniaxial strength of plain concrete and the total cross-sectional area. Whilst the early cover spalling of concentrically loaded HSC concrete columns has long been recognised, the mechanics of early cover spalling are not well understood. Finite element modelling of concentrically
loaded circular columns shows that a cracking plane develops at the cover core interface when the axial stress in the concrete is less than the concrete's uniaxial compressive strength. This cracking occurs when the tensile stress at the cover-core interface reaches its triaxial strength limit, which is considerably below the uniaxial tension strength of the concrete. Once cover-core interface cracks have developed, the cover concrete is free to spall or buckle away.

Analysis of experimental data on concentrically loaded square and circular columns shows that the strength of the section is limited by either the spalling load or by the capacity of the confined core. For columns with moderate to high quantities of tie reinforcement, typical of HSC columns, the spalling load can be taken as 0.85 times the squash load where the squash load is calculated on the gross cross sectional area. In this paper, a semi-rational model is developed where the early cover spalling is included in calculating the squash load, and is separated from the RSB parameters developed for eccentrically loaded sections. A modified version of the AS3600 compression-bending interaction model has been developed starting with the premise that the current model works well for conventional strength columns. Also discussed in this paper are the boundary conditions that a rational RSB model must satisfy. The modified AS3600 model developed is calibrated such that for concrete with $f'_c \geq 120$ MPa the equivalent RSB corresponds to a triangular stress block. The proposed modified AS3600 model shows a good correlation against other stress block models and against experimental data for $f'_c \leq 100$ MPa.

The ductility of reinforced concrete columns is dependent on the confinement provided by the transverse reinforcement. The confinement effectiveness is related to the concrete strength, the tie spacing and configuration, the yield strength of the ties, the tie diameter, the cover and the arrangement of the longitudinal reinforcement. A parameter that reflects the level of
confinement is the effective confinement parameter $k_e \rho_s f_{yt}/f'_c$. To obtain a ductility level of
$I_{10} \geq 8$ an effective confinement parameter of $k_e \rho_s f_{yt}/f'_c \geq 0.07$ is required but with the
additional condition that the spacing of the ties, and the spacing between tied longitudinal
bars, should not exceed the lesser of $(D/2.5)$ and 300 mm. Outside of the hinge region the
spacing of the ties can be relaxed but should not exceed the minimum requirements of
$(D/2.5)$ and 300 mm. For columns with high axial loads and low moments, the hinge region
may be difficult to identify. In these columns the requirement that $k_e \rho_s f_{yt}/f'_c \geq 0.07$ should
extend for the length of the column.
9. References


29. Sugano, S., Nagashima, T., Kimura, H., Tamura, A. and Ichikawa, A., “Experimental Studies on Seismic Behaviour of Reinforced Concrete Members of High Strength Concrete,” Utilization of High Strength Concrete - Second International Symposium, SP-121, American Concrete Institute, Detroit, 1990, pp. 61-87.