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Basis for the design of lateral reinforcement for high-strength concrete columns

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Abstract. This paper attempts to provide a theoretical basis for the design of high-strength concrete columns in terms of the spacing of lateral reinforcement. In order to achieve this, important concepts had to be addressed such as the choice of a measure of ductile behaviour and a realistic high-strength concrete stress-strain model, as well as limiting factors such as longitudinal steel buckling and lateral steel fracture. A design method incorporating above factors are suggested in the paper. It is shown that both buckling of longitudinal steel and hoop fracture will not demand a reduction in spacing of lateral ties with increase in compressive strength of concrete.

Key words: high-strength concrete; columns; design; ductility; lateral reinforcement; buckling.

1. Introduction

High-strength concrete with compressive strength over 100 MPa is being used in the construction industry. The use of high-strength concrete columns for high rise buildings has proven most popular with economy, superior strength, stiffness and durability being the major advantages.

The application of high-strength concrete has preceded research and therefore the behaviour of the material cannot yet be predicted with reasonable accuracy. As a consequence, important issues related to design and construction of high-strength concrete structures are not adequately addressed in building codes. Structural designers are unable to take the full advantage of the material because of insufficient information.

A major concern regarding the use of high-strength concrete in axial compression is the increase in brittleness observed with increase in compressive strength of concrete. This observation was based on the observed rapid reduction in load carrying capacity of unconfined high-strength

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concrete specimens after reaching the peak stress. However, the same conclusion cannot be directly extended to laterally confined reinforced concrete columns under axial loads. Failure of a confined concrete column may be caused by several factors such as failure of confined concrete, fracture of lateral steel or buckling of compression steel. Some researchers (Attard and Mendis 1993, Saatcioglu and Razvi 1993, Cusson and Paultre 1994, Sheikh *et al.* 1994) have studied the extra confinement provided by the stirrups in high-strength concrete columns. However no researchers have studied the other failure modes, i.e., fracture of lateral steel and buckling of compression steel and the combination of all three possible failure modes.

In this paper, the aforementioned failure modes of laterally confined high-strength concrete columns are investigated and a method of predicting the failure of a high-strength concrete column is suggested. A case study is presented to illustrate the difference in behaviour between normal and high-strength concrete columns.

2. Stress-strain relationship of confined high-strength concrete

The main parameters required to predict the stress-strain relationship of confined concrete are; effective confining pressure, strain at the peak stress, peak stress, slope of the descending branch and the residual stress. The stress-strain model proposed by Scott *et al.* (1982) was modified by calibrating each of the above parameters using reported experimental results (Pendyala *et al.* 1996). Full details of the development of the model are presented elsewhere (Pendyala 1997). Following equations represent the proposed model which is shown graphically in Fig. 1.

Confined Concrete

$$f = Kf_c'\left[\frac{2\varepsilon}{\varepsilon_{cc}} - \left(\frac{\varepsilon}{\varepsilon_{cc}}\right)^2\right] \quad \text{for } \varepsilon \le \varepsilon_{cc} \tag{1}$$

$$f = Kf_c'[1 - Z_m(\varepsilon - \varepsilon_{cc})] \ge f_{res} \quad \text{for } \varepsilon > \varepsilon_{cc} \tag{2}$$

in which: $K = 1 + 3\dfrac{f_l}{f_c'}$

f_l is the confinement stress obtained using the Mander approach (see Section 3).

$$\varepsilon_{cc} = (0.24K^3 + 0.76)\varepsilon_c$$

$$\varepsilon_c = \frac{4.26}{\sqrt[4]{f_c'}}\,\frac{f_c'}{E_c}$$

$$f_{res} = Kf_c'(0.28 - 0.0032f_c') \ge 0$$

$$Z_m = \frac{0.5(0.018f_c' + 0.55)}{\dfrac{3 + 0.29f_c'}{145f_c' - 1000} + \dfrac{3}{4}\rho_s\sqrt{\dfrac{h''}{s}} - \varepsilon_{cc}}$$

where: h'' is the core width measured from outer-to-outer ties.

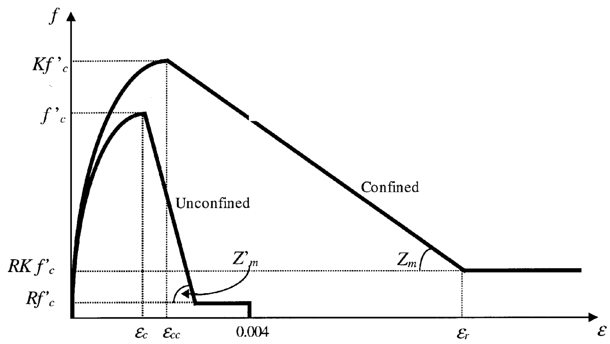

Fig. 1 Modified Scott model

Unconfined Concrete

$$f = f_c'\left[\frac{2\varepsilon}{\varepsilon_c} - \left(\frac{\varepsilon}{\varepsilon_c}\right)^2\right] \quad \text{for } \varepsilon \le \varepsilon_c \tag{3}$$

$$f = f_c'[1 - Z_m'(\varepsilon - \varepsilon_c)] \ge f_{res}' \quad \text{for } \varepsilon > \varepsilon_c \tag{4}$$

where:

$$f_{res}' = f_c'(0.28 - 0.0032 f_c') \ge 0$$

$$Z_m' = \frac{0.5(0.018 f_c' + 0.55)}{\left(\frac{3 + 0.29 f_c'}{145 f_c' - 1000} - \varepsilon_c\right)}$$

3. Evaluation of the actual confining pressure

Many researchers have shown that the confining pressure induced by rectangular ties dissipates between longitudinal bars in the plane of the cross-section and longitudinally between ties, as shown in Fig. 2. This reduction in confining pressure is taken into account by defining an effectively confined core area (Sheikh and Uzumeri 1982, Mander *et al.* 1988). The method proposed by Mander *et al.* (1988) was adopted by the authors to calculate the confining pressure. The relevant equations for the square column shown in Fig. 2 are given below. The derivation of these equations are given by Mander *et al.* (1988).

$$f_l = 0.5 k_e \rho_s f_{yt} \tag{5}$$

where: ρ_s = volume of lateral steel / volume of core concrete,
f_{yt} = yield stress of stirrups, and
k_e = the confinement effectiveness coefficient
= A_e / A_{core}

in which:

$$Ae = \left[b_s d_s - \sum_{i=1}^{n} (wi)^2/6\right]\left(1 - \frac{s^*}{2b_s}\right)\left(1 - \frac{s^*}{2d_s}\right)$$

b_s and d_s are the width and the depth of the core respectively.

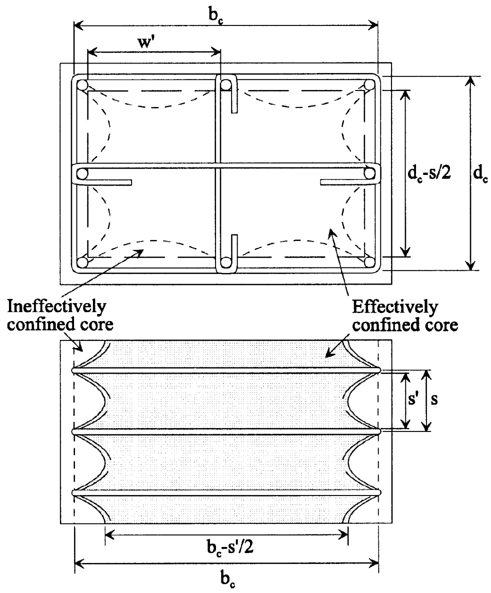

Fig. 2 Confined concrete core

4. Fracture of lateral steel

Premature fracture of hoop steel is a deficiency expected in high-strength concrete columns. Unconfined cover concrete of high-strength concrete columns deteriorate at a higher rate compared to low strength concrete. This may cause a sudden transfer of energy to the confining steel which may lead to the fracture of steel. In order to investigate this, an energy balance method, similar to that adopted by Mander *et al.* (1984) for normal strength concrete, was modified and adopted in this study. The basic energy balance equation is written as follows:

$$U_{sf} = U_g - U_{co} \tag{6}$$

where: U_{sf} = the energy required for fracture of hoop steel at one cross-section,
U_g = the total energy absorbed by the column until fracture of hoop steel, and
U_{co} = the energy required for failure of an unconfined concrete column.

Using the integrals of the stress-strain curves of steel and concrete, the axial concrete strain at which fracture of lateral steel occurs, e_{cu}, can be determined. An elastic-plastic stress-strain curve was assumed for both transverse and longitudinal steel while the modified Scott model (see Section 2) was used as the concrete stress-strain model.

Prior to the first hoop fracture, the strain energy available from the transverse reinforcement is;

$$U_{sf} = \rho_{sh} A_{cc} \int_0^{\varepsilon_{sf}} f_{s(tr)} d\varepsilon \tag{7}$$

where: ρ_{sh} = the volumetric ratio of transverse confining steel to the concrete core area,
A_{cc} = the concrete core area,
ε_{sf} = the tensile fracture strain of steel, and

$f_{s(tr)}$ = the steel stress of transverse reinforcement.

The total work done on the column is determined by integrating the force-strain curves for both confined and unconfined concrete as well as the longitudinal steel;

$$U_g = A_{cc}\int_0^{\varepsilon_{cu}} f_{cc}\,d\varepsilon + (A_g - A_{cc})\int_0^{\varepsilon_{spall}} f_c\,d\varepsilon + A_{st}\int_0^{\varepsilon_{cu}} f_{s(lo)}\,d\varepsilon \tag{8}$$

where: f_{cc} = the confined concrete stress,
A_g = the gross cross-sectional area,
ε_{spall} = the strain at which unconfined concrete carries no stress,
f_c = the unconfined concrete stress,
A_{st} = the total area of longitudinal reinforcement, and
$f_{s(lo)}$ = the steel stress of longitudinal reinforcement.

The strain energy of the unconfined plain concrete is given by;

$$U_{co} = A_g\int_0^{\varepsilon_{spall}} f_c\,d\varepsilon \tag{9}$$

Substituting Eqs. (7), (8) and (9) into Eq. (6) gives;

$$\rho_{sh}A_{cc}\int_0^{\varepsilon_{sf}} f_{s(tr)}\,d\varepsilon = A_{cc}\int_0^{\varepsilon_{cu}} f_{cc}\,d\varepsilon - A_{cc}\int_0^{\varepsilon_{spall}} f_c\,d\varepsilon + A_{st}\int_0^{\varepsilon_{cu}} f_{s(lo)}\,d\varepsilon \tag{10}$$

The derivation of integrals in Eq. (10) is described in detail below.

The integral $\int_0^{\varepsilon_{sf}} f_{s(tr)}\,d\varepsilon$ can be derived by assuming an elastic-plastic stress-strain model for transverse steel.

$$\int_0^{\varepsilon_{sf}} f_{s(tr)}\,d\varepsilon = f_{sy(tr)}\left(\varepsilon_{sf} - \frac{\varepsilon_{sy(tr)}}{2}\right) \tag{11}$$

where $f_{sy(tr)}$ is the yield stress of transverse reinforcement.

Similarly the integral $\int_0^{\varepsilon_{cu}} f_{s(lo)}\,d\varepsilon$ can be derived by assuming an elastic-plastic stress-strain model for longitudinal steel.

$$\int_0^{\varepsilon_{cu}} f_{s(lo)}\,d\varepsilon = f_{sy(lo)}\left(\varepsilon_{cu} - \frac{\varepsilon_{sy(lo)}}{2}\right) \tag{12}$$

where $f_{sy(lo)}$ is the yield stress of longitudinal reinforcement.

When determining $\int_0^{\varepsilon_{cu}} f_{cc}\,d\varepsilon$ using the modified Scott model for confined concrete (Fig. 1), two different conditions have to be considered. The first condition is if the concrete strain at hoop fracture is reached during the softening portion of the concrete stress-strain curve, while the second condition is if this strain is reached after the concrete has stopped softening and the residual stress is maintained;

$$\int_0^{\varepsilon_{cu}} f_{cc}\,d\varepsilon = A + B \qquad \text{for } \varepsilon_{cu} \le \varepsilon_r \tag{13}$$

$$\int_0^{\varepsilon_{cu}} f_{cc}\,d\varepsilon = A + C + D \qquad \text{for } \varepsilon_{cu} > \varepsilon_r \tag{14}$$

in which;
$$A = Kf_c'\int_0^{\varepsilon_{cc}}\left[\frac{2\varepsilon}{\varepsilon_{cc}} - \left(\frac{\varepsilon}{\varepsilon_{cc}}\right)^2\right]d\varepsilon \quad (15)$$

$$B = Kf_c'\int_{\varepsilon_{cc}}^{\varepsilon_{cu}}[1 - Z_m(\varepsilon - \varepsilon_{cc})^2]d\varepsilon \quad (16)$$

$$C = Kf_c'\int_{\varepsilon_{cc}}^{\varepsilon_r}[1 - Z_m(\varepsilon - \varepsilon_{cc})^2]d\varepsilon \quad (17)$$

$$D = Kf_c'\int_{\varepsilon_r}^{\varepsilon_{cu}} f_{res}d\varepsilon \quad (18)$$

At ε_r, $\quad Kf_c'[1 - Z_m(\varepsilon_r - \varepsilon_{cc})] = Kf_c'f_{res}$

Therefore,
$$\varepsilon_r = \frac{1 - f_{res}}{Z_m} + \varepsilon_{cc} \quad (19)$$

Hence, evaluating Eq. (13) for $\varepsilon_{cu} \le \varepsilon_r$;

$$\int_{\varepsilon_r}^{\varepsilon_{cu}} f_{cc}d\varepsilon = \frac{2}{3}Kf_c'\varepsilon_{cc} + \frac{1}{2}Kf_c'a(2 - Z_m a) \quad (20)$$

where:
$$a = \varepsilon_{cu} - \varepsilon_{cc} \quad (21)$$

Evaluating Eq. (14) for $\varepsilon_{cu} > \varepsilon_r$;

$$\int_0^{\varepsilon_{cu}} f_{cc}d\varepsilon = \frac{2}{3}Kf_c'\varepsilon_{cc} + Kf_c'a\frac{(1 - f_{res}^2)}{2Z_m} + \cdots f_{res}Kf_c'\left[\varepsilon_{cu} - \varepsilon_{cc} - \frac{(1 - f_{res})}{Z_m}\right] \quad (22)$$

The integral $\int_0^{\varepsilon_{spall}} f_c d\varepsilon$ is derived using the modified Scott model for unconfined concrete as shown in Fig. 1 and assuming that steel hoop fracture occurs after the cover reaches zero stress.

$$\int_0^{\varepsilon_{spall}} f_c d\varepsilon = A' + C' + D' \quad (23)$$

in which:
$$A' = f_c'\int_0^{\varepsilon_c}\left[\frac{2\varepsilon}{\varepsilon_c} - \left(\frac{\varepsilon}{\varepsilon_c}\right)^2\right]d\varepsilon \quad (24)$$

$$C' = f_c'\int_{\varepsilon_c}^{\varepsilon_r'}[1 - Z_m'(\varepsilon - \varepsilon_c)]d\varepsilon \quad (25)$$

$$D' = f_c'\int_{\varepsilon_r'}^{\varepsilon_{spall}} f_{res}d\varepsilon \quad (26)$$

At ε_r', $f_c'[1 - Z_m'(\varepsilon_r' - \varepsilon_c)] = f_c'f_{res}$

Therefore,
$$\varepsilon_r' = \frac{1 - f_{res}}{Z_m'} + \varepsilon_c \quad (27)$$

Evaluating Eq. (23);

$$\int_0^{\varepsilon_{spall}} f_c d\varepsilon = \frac{2}{3}f_c'\varepsilon_c + f_c'\frac{(1 - f_{res}^2)}{2Z_m'} + \cdots f_{res}f_c'\left[\varepsilon_{spall} - \varepsilon_c - \frac{(1 - f_{res})}{Z_m'}\right] \quad (28)$$

Therefore the solution for the concrete strain at which hoop fracture occurs can be evaluated;
For $\varepsilon_{cu} \le \varepsilon_r$;

$$\varepsilon_{cu} = \frac{E \pm \sqrt{E^2 - 4F\left[G + Kf_c'\varepsilon_{cc}\left(\frac{Z_m}{2}\varepsilon_{cc} + \frac{1}{3}\right) + \frac{2}{3}f_c'\varepsilon_c\right]}}{2F} \quad (29)$$

For $\varepsilon_{cu} > \varepsilon_r$;

$$\varepsilon_{cu} = \frac{1}{f_{res}Kf_c' + \frac{A_{st}}{A_{cc}}f_{sy(lo)}}[G + Kf_c'H] \tag{30}$$

in which:
$$E = Kf_c'(1 + Z_m\varepsilon_{cc}) + \frac{A_{st}}{A_{cc}}f_{sy(lo)} \tag{31}$$

$$F = \frac{1}{2}Kf_c'Z_m \tag{32}$$

$$G = \rho_{sh}f_{sy(tr)}\left(\varepsilon_{sf} - \frac{\varepsilon_{sy(tr)}}{2}\right) + \frac{A_{st}}{A_{cc}}f_{sy(lo)}\frac{\varepsilon_{sy(lo)}}{2} + \dots f_c'\left[\frac{(f_{res}-1)^2}{2Z_m'} + f_{res}(\varepsilon_{spall} - \varepsilon_c)\right] \tag{33}$$

$$H = \left(\frac{2}{3} - f_{res}\right)\varepsilon_{cc} + \frac{(f_{res}-1)^2}{2Z_m} - \frac{2}{3}\frac{\varepsilon_c}{K} \tag{34}$$

It is assumed in this method that the strain at which the unconfined concrete carries no more stress, ε_{spall}, is 0.004.

5. Buckling of compression steel

An upper limit on the stirrup spacing should be placed to ensure that buckling of compression steel does not occur before failure of the column by hoop fracture or crushing of concrete.

The strain at which compression buckling occurs has been evaluated by trial and error using a spreadsheet program. A trial buckling strain is entered into the program, and the solver routine is then activated to calculate the strain at which both the theoretical buckling stress (from the Euler buckling formula) and steel stress from the stress-strain model are equal.

The steel stress-strain model used was originally developed by Samra (1990) and is shown in Fig. 3:

Region AB

$\varepsilon_s \le \varepsilon_y$
$f_s = \varepsilon_s \cdot E_s$

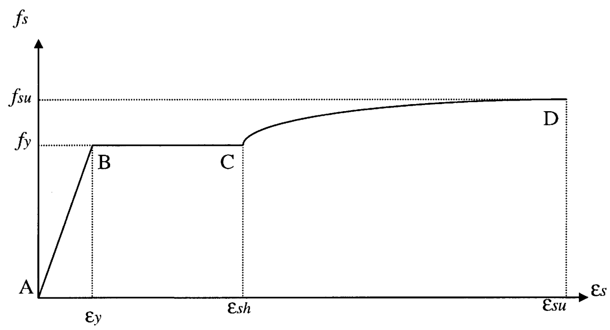

Fig. 3 Stress-strain model for steel reinforcement

Region BC

$\varepsilon_y \le \varepsilon_s \le \varepsilon_{sh}$
$f_s = f_y$

Region CD

$\varepsilon_{sh} \le \varepsilon_s \le \varepsilon_{su}$

$$f_s = f_y\left[\frac{m(\varepsilon_s - \varepsilon_{sh}) + 2}{60(\varepsilon_s - \varepsilon_{sh}) + 2} + \frac{(\varepsilon_s - \varepsilon_{sh})(60 - m)}{2(30r + 1)^2}\right]$$

where:
$$m = \frac{\left(\frac{f_{su}}{f_y}\right)(30r + 1)^2 - 60r - 1}{15r^2}$$

$$r = \varepsilon_{su} - \varepsilon_{sh}$$

Once a trial strain is entered, the corresponding steel stress is calculated using the above equations.
The trial strain is also entered into the theoretical Euler buckling stress formula. The Euler buckling load (P_{cr}) for the end conditions adopted to simulate the longitudinal steel (as shown in Fig. 4) is:

$P_{cr} = \frac{\pi^2 EI}{L'^2}$ where: L' is the effective length. From Fig. 4, $L' = L/2$.

L is the spacing of the stirrups.
Therefore the Euler buckling stress is:

$$f_{cr} = \frac{4\pi^2 EI}{s^2 A}$$

Now, if the trial strain ε(trial) is less than ε_y, then $E = E_s$ (the elastic modulus). Otherwise if ε (trial) is greater than ε_{sh}, then $E = E_r$ (the double modulus or reduced modulus).

The reduced modulus, $E_r = \frac{4E_s E_t}{(\sqrt{E_s} + \sqrt{E_t})^2}$

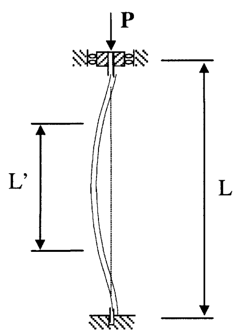

Fig. 4 Longitudinal steel model

where: E_t (the tangent modulus)=$\dfrac{df_s}{d\varepsilon_s}$

For $\varepsilon(\text{trial}) \geq \varepsilon_{sh}$, $\dfrac{df_s}{d\varepsilon_s}=f_y\left[\dfrac{(m-60)}{2(30\varepsilon(\text{trial})-30\varepsilon_{sh}+1)^2}+\dfrac{(60-m)}{2(30r+1)^2}\right]$

The values of f_{cr} and f_s are then made equal by changing ε(trial). Once this condition is satisfied, ε(trial) is then equal to the strain at which buckling of the longitudinal steel takes place.

6. Comparison of behaviour of high and normal strength columns with different configurations of lateral steel

A Column section shown in Fig. 5 was selected for comparison. Different configurations of lateral steel were obtained by calculating the spacing by three different codes, the Australian Concrete Structures Code, AS3600 (1994), American Concrete Code, ACI 318-95 (clauses suggested for seismic design) and Norwegian Concrete Code, NS 3473 (1992). AS3600 presently covers only normal strength concrete. ACI 318 formula is used by some engineers to find the stirrup spacing of high-strength concrete columns. NS 3473 covers high-strength concrete up to 90 MPa. Two different compressive strengths of concrete and two types of stirrups, R10 (10 mm diameter plain bars with yield strength of 250 MPa) and Y12 (12 mm diameter plain bars with yield strength of 400 MPa) were used for the comparison. Longitudinal strains of the column at the point of buckling of longitudinal steel (ε_b) and also fracture of lateral steel (ε_u) are tabulated in Table 1. Equivalent confining pressure exerted by lateral ties on the core of the column and the ratio of confined to unconfined compressive strength of concrete are also tabulated in Table 1. As seen from Table 1,

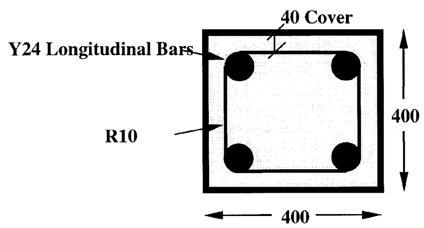

Fig. 5 Design example - column cross-section

Table 1 Ultimate concrete strains

	f_c' = 80 MPa					f_c' = 50 MPa				
	s	f_l	$K=f_{cc}/f_c$	ε_u	ε_b	s	f_l	$K=f_{cc}/f_c$	ε_u	ε_b
AS3600-R10	360	0.04	1.002	0.083	0.02	360	0.04	1.004	0.031	0.02
ACI-S-R10	9	9.37	1.469	0.078	0.19	15	5.51	1.441	0.080	0.18
NS3473-R10	240	0.14	1.007	0.104	0.02	360	0.04	1.004	0.031	0.02
AS3600-Y12	360	0.08	1.004	0.156	0.02	360	0.08	1.007	0.056	0.02
ACI-S-Y12	20	7.67	1.384	0.077	0.17	30	4.95	1.396	0.095	0.15
NS3473-Y12	240	0.26	1.013	0.206	0.02	360	0.08	1.007	0.056	0.02

ACI 318 formulae give unrealistic stirrup spacings for this example.

Data shown in Table 1 indicate that for 50 MPa concrete the provision of lateral ties according to the requirements of AS3600 result in buckling strains much lower than the strain at which hoop fracture occurs. 50 MPa is the limit specified in AS3600 for normal strength concrete. The same observation is made when the column is of 80 MPa concrete. Therefore it may be concluded that buckling of longitudinal steel will occur much earlier than the fracture of lateral steel, if the lateral ties were provided according to AS3600 requirements. However, if the spacing of lateral ties is much closer, as for ACI - Seismic requirements, the hoop fracture strain becomes lower than the buckling strain indicating failure by fracture of ties.

The strain at which buckling of longitudinal steel occurs is independent of the compressive strength of concrete. Therefore an increase in compressive strength of concrete does not require a closer spacing of stirrups to prevent buckling of longitudinal steel. If the spacing of ties is small, hoop fracture may occur before the buckling of longitudinal steel. However, increasing compressive strength of concrete from 50 to 80 MPa causes only a small reduction in the hoop fracture strain. Based on a limited theoretical study and studying other configurations, it was shown that both buckling of longitudinal steel or hoop fracture will not demand a significant reduction in spacing of lateral ties with increase in compressive strength of concrete (Kovacic 1995).

A reduction in tie spacing is required with an increase in compressive strength, if the same level of ductility is to be maintained in a confined concrete column. A method of arriving at a suitable spacing of ties is described in the next section.

7. Flexural ductility

Adequate ductility for a high-strength concrete column is provided by comparing its moment-curvature characteristics with that of a normal strength concrete column. The level of ductility obtained from a 50 MPa concrete column is therefore also ensured for a similar column made with high-strength concrete.

Moment-curvature (M-ϕ) curves for column cross-sections can be determined by considering the stress-strain distributions of concrete and steel and compatibility equations (Park and Paulay 1975). The Fortran program MEMPHI was written to generate M-ϕ curves using the modified Scott model. The ultimate curvature $\phi_{0.8}$ is defined as the curvature at which the moment has reduced to 0.8 of its maximum peak (Fig. 6). The ductility ratio is defined as $\phi_{0.8}/\phi_y$.

The yield curvature ϕ_y is defined as $\phi_y = \dfrac{\phi_y' Mi}{M_y'}$

where ϕ_y' and M_y' are the curvature and the corresponding moment calculated when either the steel at the extreme tension face of the section is yielding, or when the extreme fibre concrete compression strain reaches 0.002, whichever occurs first. M_i is the maximum moment.

To illustrate the application of this design method, the column cross-section shown in Fig. 5 was selected. As given in Table 1 (Section 6), for this column AS3600 gives a stirrup spacing of 360 mm. An 80 MPa column was selected for comparison.

In order to determine the most appropriate stirrup spacing for the 80 MPa column in terms of adequate ductility provision (equivalent to the standard 50MPa concrete column with 360 mm ligature spacing as specified by AS3600) as well as the prevention of both longitudinal steel buckling and lateral steel fracture, a plot of the axial load level versus the ductility factor $\phi_{0.8}/\phi_y$ was

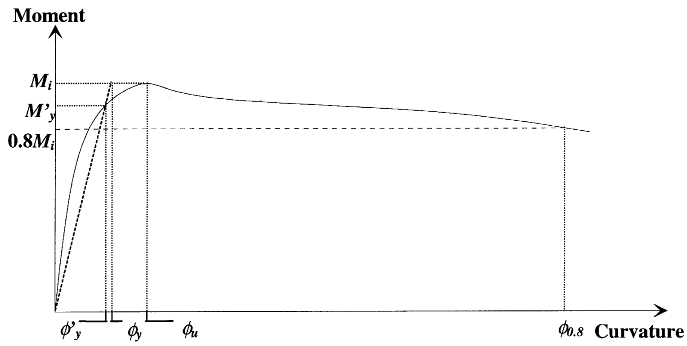

Fig. 6 Moment-curvature relationship for a cross-section

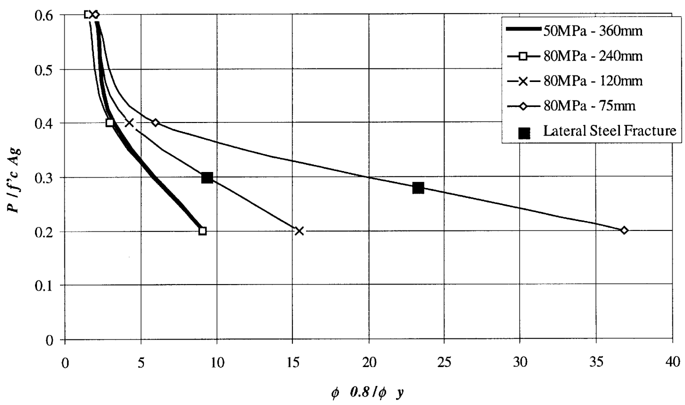

Fig. 7 Plot of axial load level versus ductility

developed, as shown in Fig. 7.

The M-ϕ curves were developed by entering different axial load levels into the Fortran computer program MEMPHI. The output data from the moment-curvature curves were then checked to see if either the buckling or fracture strain had been reached before the curvature had reached $\phi_{0.8}$. The fracture strain had been reached for the 80 MPa column with a ligature spacing of both 75 and 120 mm when the axial load level was approximately 0.3 and lower, as shown in Fig. 7. The buckling strain was never reached in any of the columns.

From Fig. 7, it is clear that an 80 MPa column with an axial load level of 0.2 can reach the same ductility level as a 50 MPa column designed under the current Australian Code provisions, if the lateral steel spacing is reduced from 360 mm to 240 mm. Similarly, an 80 MPa column with an axial load level of 0.6 can reach the same ductility level as a 50 MPa column designed under the current Australian Code provisions, if the lateral steel spacing is reduced from 360 mm to 75 mm.

The lateral steel fracture in high-strength concrete columns subjected to low axial load levels is clearly a limiting factor in the design of these elements. At lower axial load levels, the full strength capacity is more likely to be reached. Once the cover spalls off, the transfer of stress from the concrete cover to the steel hoops is therefore very large, causing hoop fracture.

The above procedure can be used to determine the most appropriate stirrup spacing for a high-strength concrete column.

8. Conclusions

The following conclusions are drawn from this study.

1. The strain at which buckling of longitudinal steel occurs is independent of the compressive strength of concrete. Therefore an increase in compressive strength of concrete does not require a closer spacing of stirrups to prevent buckling of longitudinal steel. If the spacing of ties is small, hoop fracture may occur before the buckling of longitudinal steel. However, increasing compressive strength of concrete from 50 to 80 MPa causes only a small reduction in the hoop fracture strain. Based on the limited theoretical study, it may be concluded that both buckling of longitudinal steel or hoop fracture will not demand a significant reduction in spacing of lateral ties with increase in compressive strength of concrete.

2. A reduction in tie spacing is required with an increase in compressive strength, if the same level of ductility is to be maintained in a confined concrete column. The method suggested in this paper considering buckling of bars, fracture of stirrups and confinement can be used to find the stirrup spacing of HSC columns.

3. This paper attempts to provide only a theoretical basis for the design of lateral reinforcement for high-strength concrete columns. More work is required to compare and validate the theoretical results with experimental results.

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