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<td>Author(s)</td>
<td>Kwan, AKH; Ho, JCM</td>
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<tr>
<td>Citation</td>
<td>Advances In Structural Engineering, 2010, v. 13 n. 4, p. 651-664</td>
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<td>Issued Date</td>
<td>2010</td>
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<tr>
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Ductility Design of High-Strength Concrete Beams and Columns

A.K.H. Kwan and J.C.M. Ho*
Department of Civil Engineering, The University of Hong Kong, Pokfulam, Hong Kong, China

(Received: 5 August 2009; Received revised form: 5 January 2010; Accepted: 14 January 2010)

Abstract: High-strength concrete (HSC) is increasingly used for the construction of tall buildings and long span bridges. However, most engineers just focus on how to better utilize the strength potential of HSC and little attention is paid to ensure that the HSC structures would have sufficient ductility. In this regard, it should be noted that the current design codes, which do not provide any guidelines for ductility design, are not applicable to HSC structures. In recent years, the authors have been conducting research on how the use of HSC would affect the flexural ductility of concrete members. Herein, an overall summary of their research is presented. It will be shown that depending on the reinforcement detailing and loading conditions, the ductility of HSC structures is not necessarily lower than that of normal concrete structures. Finally, guidelines for the ductility design of HSC beams and columns, augmented with design formulas and charts, are given.

Key words: beams, columns, ductility, high-strength concrete.

1. INTRODUCTION
Compared to normal-strength concrete (NSC), high-strength concrete (HSC) has the advantage of providing a higher strength for carrying more loads but also the disadvantage of rendering a lower ductility for resisting accidental overloading, impact and earthquake. For instance, it has been found that HSC beams, when heavily reinforced such that compression failure occurs, would fail a more brittle manner than NSC beams (Pecce and Fabbrocino 1999; Lin and Lee 2001; Pam et al. 2001a, b; Debernardi and Taliano 2002). Likewise, HSC columns, when subjected to the same axial load level (the same axial load to axial load capacity ratio), are generally more brittle than NSC columns (Li et al. 1991; Bayrak and Sheikh 1998; Ho and Pam 2003a; Pam and Ho 2009). Hence, the ductility of HSC members has been a major concern.

From the safety point of view, ductility should be regarded as crucial as strength (Park and Paulay 1975; Watson and Park 1994; Marefat et al. 2006; Xiao and Zhang 2006; Oehlers et al. 2008; Wilson 2009). Therefore, seismic resistant buildings or bridges are usually designed using performance-based approach (Li and Wu 2006; Liang 2007; Xuan et al. 2008; Xue et al. 2008). To improve the ductility of concrete beams and columns, one of the general methods is to provide significant amount of confinement in the form of reinforcing steel (Ho and Pam 2003a; Wu et al. 2004; Su and Wong 2007; Yang et al. 2009; Yalim et al. 2009; Zheng and Xie 2009), or in the form of steel tube (Han et al. 2005, 2008; Choi et al. 2006; Cai and Long 2007; Lu and Sun 2007; Shan et al. 2007; Tao et al. 2007; Zhang and Guo 2007; Choi et al. 2008; Park et al. 2008) or in the form of FRP (Al-Emrani and Kliger 2006; Haritos et al. 2006; Jiang and Teng 2007; Wong et al. 2008; Lam and Teng 2009; Ilki et al. 2009, Wu and Wei 2010). The ductility of reinforced concrete beams can also be improved significantly by compressive yielding.
of perforated P-blocks (Wu 2006, 2008). However, compared to strength analysis, ductility analysis is more difficult. To evaluate the flexural ductility, it is necessary to conduct nonlinear moment-curvature analysis extended into the post-peak range. Moreover, since strain reversal (decrease of strain despite monotonic increase of curvature) could occur, the stress-path dependence of the steel reinforcement needs to be taken into account (Pam et al. 2001a). Because of such complexities, it is not usually practical to demand ductility analysis and, therefore, in the current design codes (Standard Australia 2001; Ministry of Construction 2002; Buildings Department 2004; European Committee for Standardization 2004; ACI Committee 318 2008), only empirical deemed-to-satisfy rules, which limit the maximum tension steel or neutral axis depth in beams and impose minimum confinement in columns, are stipulated to ensure the provision of nominal ductility. However, these existing rules were developed in the past for NSC members and are not really applicable to HSC members (Li et al. 1991; Bayrak and Sheikh 1998; Ho and Pam 2003a) because the maximum tension steel/neutral axis depth and minimum confinement should vary with the concrete strength.

As HSC is becoming more and more commonly used and many engineers are just following the existing rules with little attention paid to the provision of sufficient ductility, it is now a matter of urgency to develop appropriate rules for the ductility design of HSC members. To resolve this problem, the authors have been studying in recent years the effects of various structural parameters, including the concrete strength, steel yield strength, compression steel ratio, tension steel ratio, confinement and axial load, on the flexural ductility of NSC and HSC members (Pam et al. 2001a, b; Kwan et al. 2002, 2004a, 2004b, 2006; Ho et al. 2003, 2004, 2009; Lam et al. 2009a, b). Based on these studies, design formulas for direct evaluation of the flexural ductility of beams and columns have been derived. Furthermore, to ensure the provision of a consistent minimum level of ductility to HSC members at not lower than the minimum level being provided to NSC members, a new ductility design method that is applicable at all concrete strength levels has been developed. Lastly, guidelines supplementing the existing deemed-to-satisfy detailing rules for incorporation into the design codes are proposed.

2. NONLINEAR MOMENT-CURVATURE ANALYSIS

To study the flexural behaviour and ductility of reinforced concrete members, nonlinear moment-curvature analysis was employed. For the concrete, the stress-strain curve was based on the model developed by Attard and Setunge (1996), which is applicable to both confined and unconfined concrete up to a concrete strength of 130 MPa. For the steel reinforcement, the stress-strain curve was assumed to be linearly elastic-perfectly plastic and, to allow for strain reversal, the descending branch was assumed to have the same gradient as the initial elastic branch. In the analysis, it was assumed that: (1) Plane sections remain plane after bending; (2) The tensile strength of concrete is negligible; (3) There is no slip between concrete and steel reinforcement; (4) The concrete core is confined while the concrete cover is unconfined. (5) The confining pressure provided to the concrete core by confinement is assumed to be constant throughout the concrete compression zone. Assumptions (1) to (4) are commonly accepted and have been adopted by various researchers (Pam et al. 2001a; Au and Kwan 2004; Wu et al. 2004). Assumption (5) is not exact because the confining pressure varies in the concrete compression zone with strain gradient. However, as this happens within a narrow range of concrete strain, the differences in the confined concrete compressive force and moment capacity are not significant (Ho et al. 2010). The moment-curvature behaviour of the section was analysed by applying prescribed curvatures incrementally starting from zero. At a prescribed curvature, the stresses developed in the concrete and steel were determined from the strain profile and their respective stress-strain curves. Then, the neutral axis depth and resisting moment were evaluated from the equilibrium conditions. Such procedure was repeated until the section had entered well into the post-peak stage.

Using the above moment-curvature analysis, a series of parametric studies have been carried out. The sections analysed are shown in Figure 1. In order to study the effects of the various structural parameters, the in-situ concrete strength $f_{cc}$ was varied from 40 to 100 MPa, the steel yield strength $f_y$ was varied from 250 to 600 MPa, the confining pressure $f_c$ was varied from 0 to 4 MPa (the confining pressure may be evaluated using the method developed by Mander et al. 1988), the compression steel ratio $\rho_c$ of beam section was varied from 0 to 2%, the tension steel ratio $\rho_t$ of beam section was varied from 0.4 to 2 times the balanced steel ratio $\rho_b$, the longitudinal steel ratio $\rho$ of column section was varied from 1 to 6%, and the axial load level $P/A_{f_{co}}$ of column section was varied from 0.1 to 0.6.

Three failure modes have been observed: (1) Tension failure, in which the tension steel yields during failure; (2) Balanced failure, in which the most highly stressed tension steel just yields during failure; and (3) Compression failure, in which none of the tension steel yields during failure. In beams, the three failure modes occur when the tension steel ratio $\rho_t$ is smaller than,
equal to and larger than the balanced steel ratio \( \rho_b \), respectively. It has been found that the balanced steel ratio \( \rho_b \) of a beam section is related to the balanced steel ratio \( \rho_{bo} \) of the same beam section with no compression reinforcement by

\[
\rho_b = \rho_{bo} + \rho_c.
\]

Using regression analysis, a formula for direct evaluation of \( \rho_{bo} \) when high-yield steel (\( f_y = 460 \) MPa) is used has been derived as:

\[
\rho_{bo} = 0.005 (f_{co})^{0.58} (1 + 1.2 f_y)^{0.3}
\]

In columns, the three failure modes occur when the axial load level \( P/Ag_{fco} \) is lower than, equal to and higher than the balanced axial load level \( (P/Ag_{fco})_b \), respectively. Using regression analysis, a formula for direct evaluation of \( (P/Ag_{fco})_b \) when high-yield steel \( (f_y = 460 \) MPa) is used has been derived as:

\[
(P/Ag_{fco})_b = 3.1 (f_{co})^{-0.5} (1 + 2 f_y)^{0.3}
\]

Both the above two formulas are accurate to within 10% error.

From the moment-curvature curve, the flexural ductility of each section may be evaluated in terms of the curvature ductility factor \( \mu \) defined by Park and Paulay (1975) as \( \mu = \phi_u/\phi_y \), where \( \phi_u \) and \( \phi_y \) are the ultimate and yield curvatures, respectively. The ultimate curvature \( \phi_u \) is taken as the curvature when the resisting moment has, after reaching the peak moment \( M_p \), dropped to 0.8 \( M_p \). The yield curvature \( \phi_y \) is taken as the curvature at which the peak moment \( M_p \) would be reached if the stiffness of the section is equal to the secant stiffness at 0.75 \( M_p \) (Watson and Park 1994). Based on the curvature ductility factors so evaluated, the effects of various structural parameters on the flexural ductility of beams and columns have been studied, as presented in the following sections.

3. FLEXURAL DUCTILITY OF BEAMS

In the case of beams, the effect of the concrete strength \( f_{co} \) has been found to be dependent on the amounts of tension and compression reinforcement, which determine the degree of reinforcement of the section. Herein, the degree of reinforcement is denoted by \( \lambda \) and explicitly defined as \( \lambda = (\rho_t - \rho_c)/\rho_{bo} \). When \( \lambda < 1 \), \( \lambda = 1 \) and \( \lambda > 1 \), the section is under-reinforced, balanced and over-reinforced, respectively. To study the effect of the concrete strength \( f_{co} \), the curvature ductility factor \( \mu \) is plotted for different concrete strengths of \( f_{co} = 40, 70 \) and 100 MPa against the degree of reinforcement \( \lambda \) in Figure 2(a) and against the tension steel ratio \( \rho_t \) in

Figure 1. Beam and column sections analysed

Figure 2. Flexural ductility of beams at different concrete strengths

Confined zone

Unconfined zone

Section properties

Beams
- \( b = 300 \) mm
- \( h = 600 \) mm
- \( d_1 = 50 \) mm
- \( d_2 = 550 \) mm
- \( d_2 \) and \( d_3 \) not exist
- \( \rho_t = A_{st}/bd = 0.4 \) to 2.0\%\( \rho_{bo} \)
- \( \rho_c = A_{ct}/bd = 0 \) to 2\%

Columns
- \( b = h = 1000 \) mm
- \( A_g = bh = 1.0 \) m²
- \( d_1 = 80 \) mm
- \( d_2 = 360 \) mm
- \( d_3 = 640 \) mm
- \( d_4 = 920 \) mm
- \( P/A_{fco} = 0.1 \) to 0.6
- \( \rho = A_s/A_g = 1 \) to 6\%

\( \mu = \phi_u/\phi_y \)
Figure 2(b). From these figures, it can be seen that as $\lambda$ or $\rho_t$ increases, $\mu$ decreases until it reaches a relatively low and constant value when the section becomes over-reinforced. It can also be seen that at the same degree of reinforcement $\lambda$, the ductility factor $\mu$ is lower at a higher concrete strength $f_{co}$. This is because of the gradual reduction in material ductility as the concrete strength $f_{co}$ increases. However, at the same tension steel ratio $\rho_c$, the ductility factor $\mu$ is higher at a higher concrete strength $f_{co}$. This is because as the concrete strength $f_{co}$ increases, the balanced steel ratio $\rho_{bo}$ also increases, leading to decrease in the degree of reinforcement $\lambda$ and increase in the ductility factor $\mu$. Hence, the flexural ductility of a HSC beam is not necessarily lower, albeit the HSC is more brittle per se.

To study the effect of the confining pressure $f_r$, the curvature ductility factor $\mu$ is plotted for different confining pressures of $f_r = 0$, 1 and 2 MPa against the degree of reinforcement $\lambda$ in Figure 3(a) and against the tension steel ratio $\rho_t$ in Figure 3(b). It is observed that at a fixed degree of reinforcement $\lambda$, the ductility factor $\mu$ increases significantly with the confining pressure $f_r$. This is because of the gradual increase in material ductility of the confined concrete as the confining pressure $f_r$ increases. It is also observed that at a fixed tension steel ratio $\rho_t$, the ductility factor $\mu$ increases substantially with the confining pressure $f_r$. This is because apart from the material ductility of the confined concrete, the balanced steel ratio $\rho_{bo}$ also increases with the confining pressure $f_r$, leading to decrease in the degree of reinforcement $\lambda$ and further increase in the ductility factor $\mu$. Hence, in general, the provision of confinement is an effective means of improving the flexural ductility of beams.

To enable direct evaluation of the flexural ductility of beams without conducting any nonlinear moment-curvature analysis, the values of $\mu$ obtained from the parametric studies are correlated to the various structural parameters using regression analysis to produce the following formulas:

$$\mu = 10.7m(\lambda)^{-1.25n}(f_{co})^{-0.45}(f_s/460)^{-0.25}$$ (3a)

$$m = 1 + 2.5(f_{co})^{0.5}(f_r/f_{co})$$ (3b)

$$n = 1 + 5.0(f_r/f_{co})$$ (3c)

where all strength values are in MPa and $\lambda$ is to be taken as 1.0 when it is larger than 1.0. Within the range of parameters studied, the above formulas for direct evaluation of $\mu$ are accurate to within 10% error.

As the use of HSC in place of NSC could increase the flexural strength but at the same time decrease the flexural ductility, some engineers may query how much net benefit could be derived from HSC. To appraise the benefit of using HSC, it is necessary to consider the concurrent flexural strength and ductility that could be achieved. In Figure 4, the concurrent flexural strength and ductility that could be achieved at different concrete strengths with or without compression reinforcement provided are plotted in the form of $\mu$ vs. $\lambda$ and $f_{co}$ vs. $f_r$ curves. From the figure, it is apparent that the $\mu$ vs. $f_{co}$ curve is generally higher at a higher concrete strength. This implies that the use of HSC could increase the flexural strength at a given flexural ductility, increase the flexural ductility at a given flexural strength or increase both the flexural strength and flexural ductility. Likewise, the $\mu$ vs. $f_r$ curve is also generally higher when compression reinforcement is provided. Hence, the provision of compression reinforcement could also increase the flexural strength and/or flexural
ductility. For designing beams to meet with concurrent flexural strength and ductility requirements, Figure 4 may be used as a design chart.

4. FLEXURAL DUCTILITY OF COLUMNS

In the case of columns, the effect of the concrete strength $f_{co}$ has been found to be dependent on the axial load. To study the effect of the concrete strength $f_{co}$, the curvature ductility factor $\mu$ is plotted for different concrete strengths of $f_{co} = 40$, $70$ and $100$ MPa against the axial load level $P/Ag_{fco}$ in Figure 5(a) and against the axial stress level $P/Ag$ in Figure 5(b). From these figures, it can be seen that as $P/Ag_{fco}$ or $P/Ag$ increases, $\mu$ decreases at a gradually decreasing rate. It can also be seen that at the same axial load level $P/Ag_{fco}$, the ductility factor $\mu$ is lower at a higher concrete strength $f_{co}$. This is because as the concrete strength $f_{co}$ increases, the axial load capacity $A_{sfco}$ also increases, leading to decrease in the axial load level $P/Ag_{fco}$ and increase in the ductility factor $\mu$. Hence, the flexural ductility of a HSC column is not necessarily lower, albeit the HSC is more brittle per se.

To study the effect of the longitudinal steel ratio $\rho$, the curvature ductility factor $\mu$ is plotted against the longitudinal steel ratio $\rho$ at constant axial load level $P/Ag_{fco}$ in Figure 6(a) and at constant axial stress level $P/Ag$ in Figure 6(b). From both figures, it can be seen that when $P/Ag_{fco} < 0.3$ or $P/Ag < 20$ MPa, the ductility factor $\mu$ is relatively high and gradually decreases as the longitudinal steel ratio $\rho$ increases but when $P/Ag_{fco} \geq 0.3$ or $P/Ag \geq 20$ MPa, the ductility factor $\mu$ is relatively low and almost independent of the longitudinal steel ratio $\rho$. Hence, the longitudinal steel ratio $\rho$ does have some effect when the flexural ductility is relatively high but has little effect when the flexural ductility is relatively low and causing concern (Wu et al. 2004).
To study the effect of the confining pressure \( f_{rc} \), the curvature ductility factor \( \mu \) is plotted against the confining pressure \( f_{rc} \) at constant concrete strength \( f_{co} \) in Figure 7(a) and at constant axial load level \( P/Ag f_{co} \) in Figure 7(b). From both figures, it is obvious that in all cases, the provision of confinement is an effective means of improving the flexural ductility of columns. However, the effectiveness of providing confinement is generally higher at lower concrete strength or lower axial load level. When the concrete strength and/or axial load level is relatively high, in which case the ductility tends to be low, the effectiveness of providing confinement is lower and thus a disproportionately larger amount of confinement may be needed for ductility improvement.

From the above, it may be concluded that the major factors affecting the flexural ductility of columns are the concrete strength, axial load level, confining pressure and longitudinal steel ratio. However, their effects are dependent on the failure mode and thus when evaluating the flexural ductility of columns, columns failing in tension and columns failing in compression have to be dealt with separately. Using regression analysis of the data obtained from the parametric studies, the following formulas for direct evaluation of the flexural ductility of columns have been developed:

When tension failure or balanced failure occurs:

\[
\mu = 10.7m \left( \frac{\rho - \rho_{co} + \frac{P}{A_{g}f_{y}}}{{\rho_{co}}} \right)^{-1.25m} \left( f_{co} \right)^{-0.45} \left( f_{y}/460 \right)^{-0.25} \]  

(4)

When compression failure occurs:

\[
\mu = 14.0 \left( \frac{P/A_{g}f_{co}}{P/A_{g}f_{co}} \right)^{-0.45} \left( f_{co} \right)^{-0.45} \left( 1 + 30(f_{y}/f_{co}) \right)
\]  

(5)

where \( m \) and \( n \) are the same as those given by Eqns 3b and 3c, and \( A_{sb} \) is the balanced steel area (the area of tension steel causing balanced failure). Both the above formulas are accurate to within 15%.
5. COMPARISON WITH EXPERIMENTAL RESULTS

To verify the validity of the above formula for concrete beams and columns, the flexural ductility predicted by Eqn 3 for concrete beams, and Eqns 4 and 5 for concrete columns, have been compared with experimental results obtained by other researchers. For the sake of comparison, it should be noted that the value of $f_{co}$ for each beam and column is taken as $0.85\eta f'_{c}$, where $\eta$ is the ratio of the average concrete stress of the equivalent rectangular stress block to the cylinder strength $f'_{c}$ as stipulated in EC2 (European Committee for Standardization 2004). However, if only the concrete cube strength $f_{cu}$ is provided, $f_{co}$ is then taken as $0.72 f_{cu}$ (Ho et al. 2002). On the other hand, the value of confining pressure is evaluated according to the formula provided by Mander et al. (1988). For symmetrically reinforced concrete columns, the following equation can be used:

$$f_{c} = 0.5 k_e \rho_s f_{ys}$$

where $k_e$ is the confinement effectiveness factor, $\rho_s$ is the volumetric ratio of confinement and $f_{ys}$ is the yield strength of confinement.

For concrete beams, the curvature ductility factors evaluated by Eqn 3 are compared with the curvature ductility factors obtained by Pecce and Fabbrocino (1999), Lin and Lee (2001) and Debernardi and Taliano (2002). The comparison is summarised in Table 1. From the table, it is evident that the predicted curvature ductility factors using Eqn 3 are all very close to the respective curvature ductility factors obtained by other researchers from experiment. It thus verifies the accuracy and applicability of Eqn 3 in predicting the ductility of normal- and high-strength concrete beams.

For unconfined concrete columns, where $f_r$ is negligible, the curvature ductility factors evaluated by Eqn 4 or 5 are compared with the curvature ductility factors obtained by Sheikh and Yeh (1990) and Ho and Pam (2003a, b). The comparison is summarised in Table 2. From the table, it is evident that the predicted curvature ductility factors using Eqn 4 for columns failing in tension, and Eqn 5 for columns failing in compression, are all very close to the respective curvature ductility factors obtained by other researchers from experiment. It thus verifies the accuracy and applicability of Eqns 4 and 5 in predicting the ductility of unconfined normal- and high-strength concrete columns.

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</tbody>
</table>
### Table 2. Comparison of predicted ductility factors of unconfined concrete columns

<table>
<thead>
<tr>
<th>Specimen code</th>
<th>$f_{cu}$ or $f_{c'}$ (MPa)</th>
<th>$f_{co}$ (MPa)</th>
<th>$(P/A_{g f_{co}})$</th>
<th>$(P/A_{g f_{co}})_{b}$</th>
<th>$\mu$ By Eqn</th>
<th>Test Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ho and Pam (2003a)</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>BS-80-01-09-R6</td>
<td>89.6</td>
<td>64.5</td>
<td>0.150</td>
<td>0.472</td>
<td>6.5</td>
<td>7.3</td>
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<td>BS-80-01-09-R8</td>
<td>85.4</td>
<td>61.5</td>
<td>0.149</td>
<td>0.484</td>
<td>6.8</td>
<td>7.2</td>
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<td>BS-80-01-09-R10</td>
<td>83.2</td>
<td>59.9</td>
<td>0.154</td>
<td>0.506</td>
<td>6.6</td>
<td>7.7</td>
</tr>
<tr>
<td>Ho and Pam (2003b)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BS-60-06-61-S</td>
<td>56.5</td>
<td>40.7</td>
<td>0.173</td>
<td>0.596</td>
<td>2.2</td>
<td>2.3</td>
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<td>BS-60-06-61-C</td>
<td>60.4</td>
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<td>0.595</td>
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<tr>
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<tr>
<td>Sheikh and Yeh (1990)</td>
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</tr>
<tr>
<td>E-8</td>
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<td>0.989</td>
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<td>A-11</td>
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<td>F-12</td>
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<td>D-14</td>
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<td>0.706</td>
<td>0.876</td>
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</tr>
</tbody>
</table>

*If $(P/A_{g f_{co}}) \leq (P/A_{g f_{co}})_b$, then Eqn 4 is used.*
*If $(P/A_{g f_{co}}) > (P/A_{g f_{co}})_b$, then Eqn 5 is used.*

### Table 3. Comparison of predicted ductility factors of confined concrete columns

<table>
<thead>
<tr>
<th>Specimen code</th>
<th>$f_{cu}$ or $f_{c'}$ (MPa)</th>
<th>$f_{co}$ (MPa)</th>
<th>$(P/A_{g f_{co}})$</th>
<th>$(P/A_{g f_{co}})_{b}$</th>
<th>$f_r$ (MPa)</th>
<th>$\mu$ By Eqn</th>
<th>Test Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ho (2003)</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NEW-100-03-61-C</td>
<td>108.8</td>
<td>78.4</td>
<td>0.417</td>
<td>0.681</td>
<td>4.1</td>
<td>9.8</td>
<td>11.3</td>
</tr>
<tr>
<td>NEW-80-03-24-S</td>
<td>90.4</td>
<td>65.1</td>
<td>0.389</td>
<td>0.712</td>
<td>3.4</td>
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<td>9.8</td>
</tr>
<tr>
<td>Ho and Pam (2003a)</td>
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<td>85.9</td>
<td>61.8</td>
<td>0.151</td>
<td>0.589</td>
<td>1.2</td>
<td>12.5</td>
<td>12.8</td>
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<tr>
<td>Ho and Pam (2003b)</td>
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<td>NEW-100-03-24-C</td>
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<td>78.0</td>
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<td>0.870</td>
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<td>8.4</td>
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<tr>
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<td>41.1</td>
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<td>3.3</td>
<td>9.0</td>
<td>8.3</td>
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<tr>
<td>Sheikh et al. (1994)</td>
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<tr>
<td>AS-3H</td>
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<td>0.728</td>
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<td>46.5</td>
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<td>Sheikh and Khoury (1993)</td>
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<td>3.1</td>
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<td>0.880</td>
<td>1.152</td>
<td>2.9</td>
<td>4.8</td>
<td>5.5</td>
</tr>
</tbody>
</table>

*If $(P/A_{g f_{co}}) \leq (P/A_{g f_{co}})_b$, then Eqn 4 is used.*
*If $(P/A_{g f_{co}}) > (P/A_{g f_{co}})_b$, then Eqn 5 is used.*
For confined concrete columns, where \( f_r \) is significant, the curvature ductility factors evaluated by Eqn 4 or 5 are compared with the curvature ductility factors obtained by Sheikh and Yeh (1990), Sheikh and Khoury (1993), Sheikh et al. (1994) and Ho and Pam (2003a, b). The comparison is summarised in Table 3. From the table, it is evident that the predicted curvature ductility factors using Eqn 4 for columns failing in tension, and Eqn 5 for columns failing in compression, are all very close to the respective curvature ductility factors obtained by other researchers from experiment. It thus verifies the accuracy and applicability of Eqns 4 and 5 in predicting the ductility of confined normal- and high-strength concrete columns.

### 6. MINIMUM DUCTILITY DESIGN

In the current design codes, there are no specific guidelines for the ductility design of any concrete members. Only nominal ductility is provided by stipulating some empirical deemed-to-satisfy detailing rules. The curvature ductility factors achieved by following these rules vary significantly and are generally lower at a higher concrete strength. With a view to maintain the flexural ductility when using HSC in place of NSC, it is advocated that instead of following the existing empirical rules, the members should be designed to achieve a consistent minimum curvature ductility factor, denoted herein by \( \mu_{\text{min}} \) which should not be lower than that being provided to NSC members. Based on previous studies by the authors (Au and Kwan 2004; Ho et al. 2004; Kwan et al. 2006; Lam et al. 2009b), it is proposed to set \( \mu_{\text{min}} = 3.32 \) for non-earthquake resistant structures. For earthquake resistant structures, a higher value of \( \mu_{\text{min}} \) should be adopted, depending on the actual ductility demand of the member.

For beams, the ductility design may be carried out by imposing the condition that the value of \( \mu \) evaluated by Eqn 3 must be higher than \( \mu_{\text{min}} \). This condition implies that for any given values of \( f_{co}, f_r \), and \( f_r \), there is a maximum allowable value of \( \lambda \) and a maximum allowable value of \( (\rho_c - \rho) \), which are denoted by \( \lambda_{\text{max}} \) and \( (\rho_c - \rho)_{\text{max}} \) respectively. For illustration, the values of \( (\rho_c - \rho)_{\text{max}} \) for achieving \( \mu_{\text{min}} = 3.32 \) when no confinement is provided are listed in Table 4. These values indicate that the value of \((\rho_c - \rho)_{\text{max}} \) increases as the concrete strength \( f_{co} \) increases. An alternative ductility design method is to limit the neutral axis depth \( d_n \) to not more than a certain fraction of the effective depth \( d \), as in most of the current design codes (Standard Australia 2001; Ministry of Construction 2002; Buildings Department 2004; European Committee for Standardization 2004). The maximum allowable values of \( d_n/d \) for achieving \( \mu_{\text{min}} = 3.32 \) when no confinement is provided are also listed in the table. It is clear from these values that the maximum allowable value of \( d_n/d \) decreases significantly as the concrete strength \( f_{co} \) and/or steel yield strength \( f_r \) increases. For comparison, the respective flexural strengths that can be achieved with no compression reinforcement provided are also presented in the table. From these strength values, it is evident that the use of a higher strength concrete would allow a higher flexural strength to be attained while maintaining the same \( \mu_{\text{min}} \), whereas the use of a higher strength steel would reduce the flexural strength that can be attained at the same \( \mu_{\text{min}} \).

For columns, the ductility design may be carried out by imposing the condition that the value of \( \mu \) evaluated by Eqn 4 or 5 must be higher than \( \mu_{\text{min}} \). It follows from this condition that for any given values of \( f_{co}, f_r \), and \( f_r \), there is a maximum allowable value of \( P/A_{f_{co}} \) and for any given values of \( f_{co}, f_r \) and \( P/A_{f_{co}} \), there is a minimum allowable value of \( f_r \). The maximum allowable value of \( P/A_{f_{co}} \) and the minimum allowable value of \( f_r \) are denoted by \((P/A_{f_{co}})_{\text{max}} \) and \((f_r)_{\text{min}} \).
respectively. The values of \( (P/A_g f_{co})_{\text{max}} \) and \( (f_r)_{\text{min}} \) for achieving \( \mu_{\text{min}} = 3.32 \) at different concrete strengths and a fixed steel yield strength of 460 MPa are presented in Tables 5 and 6, respectively. From Table 5, it can be seen that the value of \( (P/A_g f_{co})_{\text{max}} \) decreases as the concrete strength \( f_{co} \) increases but increases as the confining pressure \( f_r \) increases. Hence, when HSC is used in place of NSC with no corresponding increase in the confining pressure, the maximum axial load level has to be reduced, thus limiting the beneficial use of HSC. From Table 6, it can also be seen that the value of \( (f_r)_{\text{min}} \) increases as the concrete strength \( f_{co} \) or the axial load level \( P/A_g f_{co} \) increases. Hence, for heavily load HSC columns, it is particularly important to provide a sufficiently high confining pressure to maintain a minimum level of flexural ductility.

### 7. SUPPLEMENTARY GUIDELINES

To remedy the situation that the existing deemed-to-satisfy detailing rules would yield a lower flexural ductility at a higher concrete strength, supplementary design guidelines for incorporation into the design codes are formulated herein.

For the design of beams, the current practices (ACI Committee 318 2008; Standard Australia 2001; Buildings Department 2004, European Committee for Standardization 2004) are either to limit the tension steel ratio \( \rho_t \) at not larger than 0.75 times the balanced steel ratio \( \rho_{bo} \) or to limit the neutral axis depth \( d_n \) at not larger than 0.50 times the effective depth \( d \). To ensure that HSC beams so designed would achieve a minimum flexural ductility of \( \mu_{\text{min}} = 3.32 \), it is proposed to add compression and/or confining reinforcement to make up the reduction in flexural ductility due to the use of HSC. The compression steel ratio \( \rho_c \) and confining pressure \( f_r \) required have been evaluated as:

When \( \rho_t \) is limited to \( 0.75\rho_{bo} \):

\[
3350\rho_t + 54f_r = f_{co} - 30
\]

When \( d_n \) is limited to \( 0.50d \):

\[
2570\rho_t + 39f_r = f_{co} - 30
\]

where \( f_{co} \) and \( f_r \) are in MPa and \( 40 \leq f_{co} \leq 100 \) MPa.

For the design of columns, the current practice (Standard Australia 2001; Ministry of Construction 2002; Buildings Department 2004; European Committee for Standardization 2004; ACI Committee 318 2008) is to set limits on the minimum size and maximum spacing of the confining reinforcement so as to provide certain nominal confinement. It has been

### Table 5. Maximum axial load level for minimum ductility design of columns

<table>
<thead>
<tr>
<th>( f_{co} ) (MPa)</th>
<th>( f_r = 0 ) MPa</th>
<th>( f_r = 0.5 ) MPa</th>
<th>( f_r = 1 ) MPa</th>
<th>( f_r = 2 ) MPa</th>
<th>( f_r = 3 ) MPa</th>
<th>( f_r = 4 ) MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>0.26</td>
<td>0.56</td>
<td>0.75</td>
<td>0.97</td>
<td>&gt; 1.0</td>
<td>&gt; 1.0</td>
</tr>
<tr>
<td>50</td>
<td>0.20</td>
<td>0.35</td>
<td>0.62</td>
<td>0.82</td>
<td>0.97</td>
<td>&gt; 1.0</td>
</tr>
<tr>
<td>60</td>
<td>0.16</td>
<td>0.32</td>
<td>0.53</td>
<td>0.71</td>
<td>0.86</td>
<td>0.94</td>
</tr>
<tr>
<td>70</td>
<td>0.12</td>
<td>0.27</td>
<td>0.39</td>
<td>0.63</td>
<td>0.76</td>
<td>0.85</td>
</tr>
<tr>
<td>80</td>
<td>0.10</td>
<td>0.27</td>
<td>0.32</td>
<td>0.57</td>
<td>0.68</td>
<td>0.77</td>
</tr>
<tr>
<td>90</td>
<td>0.09</td>
<td>0.23</td>
<td>0.29</td>
<td>0.50</td>
<td>0.61</td>
<td>0.70</td>
</tr>
<tr>
<td>100</td>
<td>0.08</td>
<td>0.22</td>
<td>0.26</td>
<td>0.42</td>
<td>0.56</td>
<td>0.63</td>
</tr>
</tbody>
</table>

### Table 6. Minimum confining pressure for minimum ductility design of columns

<table>
<thead>
<tr>
<th>( f_{co} ) (MPa)</th>
<th>( P/A_g f_{co} = 0.1 )</th>
<th>( P/A_g f_{co} = 0.2 )</th>
<th>( P/A_g f_{co} = 0.3 )</th>
<th>( P/A_g f_{co} = 0.4 )</th>
<th>( P/A_g f_{co} = 0.5 )</th>
<th>( P/A_g f_{co} = 0.6 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>0.00</td>
<td>0.00</td>
<td>0.06</td>
<td>0.30</td>
<td>0.39</td>
<td>0.50</td>
</tr>
<tr>
<td>50</td>
<td>0.00</td>
<td>0.00</td>
<td>0.23</td>
<td>0.52</td>
<td>0.68</td>
<td>0.98</td>
</tr>
<tr>
<td>60</td>
<td>0.00</td>
<td>0.02</td>
<td>0.39</td>
<td>0.73</td>
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<td>1.43</td>
</tr>
<tr>
<td>70</td>
<td>0.00</td>
<td>0.09</td>
<td>0.57</td>
<td>0.95</td>
<td>1.34</td>
<td>1.88</td>
</tr>
<tr>
<td>80</td>
<td>0.00</td>
<td>0.20</td>
<td>0.78</td>
<td>1.20</td>
<td>1.68</td>
<td>2.27</td>
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<tr>
<td>90</td>
<td>0.02</td>
<td>0.34</td>
<td>1.07</td>
<td>1.51</td>
<td>2.02</td>
<td>2.89</td>
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<td>0.07</td>
<td>0.46</td>
<td>1.35</td>
<td>1.85</td>
<td>2.47</td>
<td>3.47</td>
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</table>
found that the nominal confinement would provide a confining pressure of only about 0.20 MPa (Lam et al. 2009b). With such low confining pressure provided, the flexural ductility could become dangerously low when HSC is used. To resolve this problem and ensure that the HSC columns would achieve a minimum flexural ductility of $\mu_{\text{min}} = 3.32$, it is proposed to set a limit on either the maximum axial load level ($P/A_{f_{\text{co}}})_{\text{max}}$ or the minimum confining pressure ($f_r$)$_{\text{min}}$. The required limits have been tabulated in Tables 5 and 6. For easy application, these tabulated limits have been transformed into the following inequality by regression analysis:

$$\frac{(P/A_{f_{\text{co}}})}{(1 + 3.5f_r)^{1.20}} \leq 24.5 (f_{\text{co}})^{-1.20}$$

(9)

where $f_{\text{co}}$ and $f_r$ are in MPa and $40 \leq f_{\text{co}} \leq 100$ MPa.

8. CONCLUSIONS

The flexural ductility of high-strength concrete beams and columns has been studied by extensive parametric studies using nonlinear moment-curvature analysis. It was found that for beams, the major factors affecting the flexural ductility are the concrete strength, steel yield strength, degree of reinforcement and confining pressure. Generally, at the same degree of reinforcement, the flexural ductility decreases as the concrete strength increases while with the same reinforcement details, the flexural ductility increases as the concrete strength increases. More importantly, the reduction in flexural ductility due to the use of higher strength concrete and/or steel can always be compensated by adding more compression and/or confining reinforcement.

For columns, the major factors affecting the flexural ductility are the concrete strength, steel yield strength, axial load/stress level and confining pressure. Generally, at the same axial load level, the flexural ductility decreases as the concrete strength increases while at the same axial stress level, the flexural ductility increases as the concrete strength increases. As the axial load level is seldom adjusted downwards when high-strength concrete is used, the flexural ductility could become dangerously low as the concrete strength increases. Nevertheless, the reduction in flexural ductility due to the use of higher strength concrete and/or steel can always be compensated by adding more confining reinforcement.

For practical applications, formulas for direct evaluation of the balanced steel ratio and balanced axial load level, which are needed to determine the failure mode, have been developed. To avoid cumbersome nonlinear moment-curvature analysis, formulas for direct evaluation of the flexural ductility of beams and columns have also been developed. These formulas have been compared with the curvature ductility factors of normal- and high-strength concrete beams and columns obtained by other researchers from experiment. From the comparison, it is evident that the proposed formulas can predict the ductility of concrete beams and columns fairly accurately.

Based on these formulas, a minimum ductility design method for ensuring the achievement of a minimum ductility of $\mu_{\text{min}} = 3.32$ has been proposed. Lastly, in order to remedy the situation that the existing deemed-to-satisfy detailing rules could yield an unacceptably low ductility at high concrete strength, supplementary guidelines on the addition of compression and confining reinforcement to beams and on the maximum axial load level and minimum confinement to be imposed to columns have been formulated for incorporation into the design codes.

ACKNOWLEDGEMENTS

Generous support from Seed Funding Programme for Basic Research (10208121) provided by The University of Hong Kong is gratefully acknowledged.

REFERENCES

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NOTATION

\[ A_g \] area of beam or column section (= bh)
\[ A_t \] total area of longitudinal steel reinforcement
\[ A_b \] balanced steel area
\[ A_c \] area of compression reinforcement
\[ A_t \] area of tension reinforcement
\[ b \] breadth of beam or column section
\[ d \] effective depth of beam or column section
\[ d_i \] depth to centroid of steel at jth layer from extreme compressive fibre

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\( d_n \)  
depth to neutral axis

\( f_c' \)  
concrete cylinder strength

\( f_{co} \)  
peak stress on stress-strain curve of unconfined concrete

\( f_{cu} \)  
concrete cube strength

\( f_r \)  
confining pressure produced by confining reinforcement

\( (f_r)_{min} \)  
minimum allowable confining pressure

\( f_y \)  
yield strength of steel reinforcement

\( f_{ys} \)  
yield strength of confinement

\( h \)  
total depth of the beam or column section

\( k_e \)  
confinement effectiveness factor as per Mander et al. (1988)

\( M_p \)  
peak moment

\( P \)  
axial load applied at centroid

\( (P/A_g f_{co})_b \)  
balanced axial load level

\( (P/A_g f_{co})_{max} \)  
maximum allowable axial load level

\( \eta \)  
ratio of equivalent concrete stress to cylinder strength as per EC2

\( \lambda \)  
degree of reinforcement

\( \lambda_{\text{max}} \)  
maximum allowable degree of reinforcement

\( \mu \)  
curvature ductility factor

\( \mu_{\text{min}} \)  
minimum curvature ductility factor to be achieved

\( \phi_u \)  
ultimate curvature

\( \phi_y \)  
yield curvature

\( \rho \)  
longitudinal steel ratio \((= A_s/A_g)\)

\( \rho_b \)  
balanced steel ratio \((= A_{sb}/bd)\)

\( \rho_{bu} \)  
balanced steel ratio for beam section with no compression reinforcement

\( \rho_c \)  
compression steel ratio \((= A_{sc}/bd)\)

\( \rho_c \)  
volumetric ratio of confinement

\( \rho_t \)  
tension steel ratio \((= A_{st}/bd)\)