PROPOSED CONFINEMENT STEEL DESIGN WITHIN CRITICAL REGION FOR LIMITED DUCTILE HIGH-STRENGTH REINFORCED CONCRETE COLUMNS

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ABSTRACT:

In high seismic risk region, the design of high-strength reinforced concrete (HSRC) columns in fully ductile buildings or bridges requires a large amount of confinement reinforcement within the critical region to avert brittle failure. However, in regions of low to moderate seismic risk, the design of these columns, which have a smaller seismic resistance demand, could be designed for reduced limited ductility. The advantage of this is that it saves considerably the construction cost and alleviates steel congestion problem within the beam-column or pier - pile cap joints. In a previous study conducted by the authors, an equation was proposed for the confinement steel design within the column's critical region for achieving limited ductile behaviour. This paper investigates experimentally the flexural ductility of three HSRC columns containing the proposed confinement steel content within their critical region and subjected to various compressive axial load levels. By comparing with fully ductile HSRC columns, it is seen that limited ductility HSRC columns can be confined effectively by less amount of confinement steel over a shorter critical region length. Lastly, some simplified design guidelines for limited ductility HSRC columns are proposed.

KEYWORDS: Columns, confinement, curvature, high-strength concrete, limited ductility

1. INTRODUCTION

Because of its higher strength to weight ratio, high-strength concrete (HSC) has increasingly been used in tall buildings and long span bridges. However, HSC is more brittle than normal-strength concrete (NSC). For example, it was shown that high-strength reinforced concrete (HSRC) beams failed in a very brittle manner when containing large amount of tension steel [1,2] even if they were under-reinforced. It was also reported that HSRC columns would be extremely brittle if they were not confined by adequate confinement steel, which should be more than that provided in normal-strength reinforced concrete (NSRC) columns for preserving the same flexural ductility [3-6]. The above experimental studies indicated that the design of HSRC columns is different from NSRC columns. Therefore, the deemed-to-satisfy rules for ductility provision stipulated in the existing RC design codes, which were established based on behaviour of NSRC members, should not be adopted for ductility design of HSRC members.

From structural safety point of view, flexural ductility should be regarded as important as strength [7]. Adequate ductility enables a large plastic rotation to be developed in the plastic hinge region and thus allows moment redistribution to occur in the member effectively. It also prevents structures from brittle collapse under accidental impacts or earthquake attacks. In a typical moment-resisting framed building, the plastic hinges would normally form in the beams, and therefore the beams should be designed for adequate ductility. However, in some structures, it is not desirable for the plastic hinges to form in the superstructures, e.g. bridges or transfer plate structures. For these structures, the plastic hinges would develop at the base of the piers or columns, and hence the piers or columns should be designed for adequate ductility. Since the piers or columns in these structures are usually subjected to large axial load, they are generally constructed using HSC such that the size of the piers or columns could be reduced. Therefore, the ductility of these HSRC piers or columns should be ensured in the design, which might not follow the traditional deemed-to-satisfy rules.

In designing for ductility, different levels of flexural ductility are normally assigned for columns according to the seismic risk of the region where the structures are located. In regions of high seismic risk, columns should be designed for a higher level of flexural ductility, while in regions of low to moderate seismic risk, columns could be designed for a reduced or limited ductility demand. Up to now, a lot of experimental tests have been conducted on fully ductile RC columns [8-9], however, little research has been done on limited ductile RC columns, especially HSRC columns. To establish the design method for limited ductile HSRC columns, factors affecting the ductility of HSRC columns should be investigated first.

Unlike flexural strength, which can be evaluated rapidly using ordinary beam bending theory, flexural ductility of RC members can only be obtained by nonlinear moment-curvature analysis based on the stress-strain curves of concrete and steel. In HSRC columns, it has been found in the previous study [10] conducted by the authors that the major factors affecting their ductility are concrete strength, axial load level and confinement steel ratio. The ductility of HSRC columns increases with either the decrease of concrete strength, the decrease of axial load level or the increase of confinement steel ratio. From the parametric study conducted in the same study, an equation incorporating the above factors for the design of confinement steel within the critical region of HSRC columns to achieve limited ductile behaviour has been proposed.

In this paper, the flexural ductility of three HSRC columns with confinement steel content within the critical region designed according to this authors' proposed equation is investigated experimentally. The HSRC columns contain target concrete cube strength of about 90MPa and are subjected to simultaneous axial compressive load and reversed cyclic lateral deformations. It will be shown from the test results that all the columns behave in a limited ductile manner, reaching a curvature ductility factor of about 10. Lastly, simplified guidelines for the design of limited ductile HSRC columns are developed for regions of low to moderate seismic risk, such as Hong Kong.

2. EXPERIMENTAL PROGRAMME

Each of the three column specimens was tested in a 6600kN self-reaction steel-loading frame under constant axial compressive load and increasing reversed cyclic displacement into inelastic stage. Ranges of properties in the specimens are: concrete cube strength (f_{cu}) from 86 to 100MPa, compressive axial load level ($P/A_g f_{cu}$) from 0.11 to 0.55, longitudinal steel ratio (ρ) from 0.9 to 6.1% and transverse steel volumetric ratio (ρ_s) from 1.73 to 3.2% (from 0.38 to 1%) within (outside) the critical region. The specified yield strengths of the reinforcing bars are 250MPa for mild steel (prefixed "R") and 460MPa for high yield steel (prefixed "T"). The reversed cyclic bending moment and displacement were applied by bending the column via a horizontal rigid beam, which was cast monolithically with the column. Figure 1 shows the test set-up and Table 1 summarises the details of the column test specimens.

Figure 1 Test set-up

Table 1 Section properties of columns

Unit	Specimen Code	Actual f_{cu} [f_c'] (MPa)	Average $P/A_g f_{cu}$	Longitudinal steel Content	Longitudinal steel ρ (%)	Transverse steel within [outside] critical region d (mm)	s (mm)	ρ_s (%)
1	100-06-61-C	100.0 [85.0]	0.55	8T32	6.1	T16 [R8]	120 [100]	3.20 [1.00]
2	80-03-24-C	90.4 [80.6]	0.28	8T20	2.4	T12 [R8]	105 [150]	2.10 [0.66]
3	80-01-09-S	85.9 [77.8]	0.11	8T12	0.9	R12 [R6]	85 [100]	1.73 [0.38]

2.1. Test Specimens

Figure 2 shows a typical test specimen consisting of a column, a horizontal rigid beam and a top flange. The cross-section dimensions of column are 325×325mm and the height is 1515mm. Each of the test specimens represents a real column in an RC moment-resisting framed building between the contra-flexure and the maximum bending moment points, which are located around the mid-height and at the face of the beam-column joint respectively. The transverse steel content within critical region of the columns was calculated using Equation (2.1), while that outside the critical region was designed only to resist the ultimate shear force.

$$\rho_s = \left(\frac{A_g}{A_c}\right)\left(0.2 - 0.2\frac{\rho\ f_y}{f_{cu}}\right)\left(\frac{P}{A_g f_{cu}}\right)^{0.9}\left(\frac{f_{cu}}{f_{ys}}\right) + 0.008 \qquad (2.1)$$

where ρ_s is the volumetric ratio of transverse steel, A_g and A_c are the gross and core concrete section areas respectively, ρ is the area ratio of longitudinal steel, f_y and f_{ys} are the yield strengths of longitudinal and transverse steel respectively, f_{cu} is the concrete cube strength and P is the compressive axial load. In the above equation, the target values of concrete strength, target axial load level, and specified yield strengths of longitudinal and transverse steel have been adopted to calculate the required ratio of confinement steel. A predetermined critical region length [11] was adopted for each specimen, i.e. 650mm for Unit 1; 500mm for Unit 2; and 325mm for Unit 3.

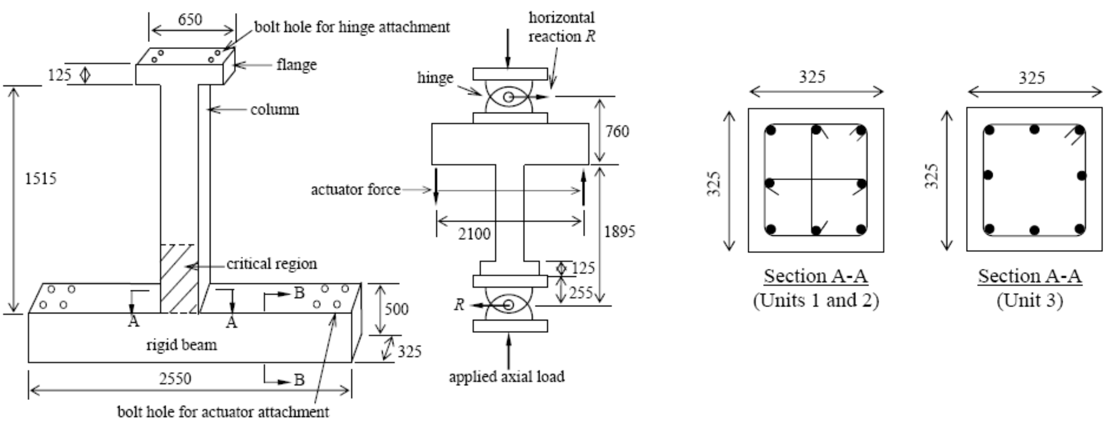


Figure 2 Detail of test specimens and loading arrangement

In order to provide a fixed support to one of the column ends, where the maximum moment occurred, the horizontal rigid beam was designed to behave elastically throughout the test. Similarly for the flange at the other end of the column to resist the flexure and shear as well as to facilitate attachment to the hinge of the loading frame.

2.2. Instrumentation

(1) Strain gauges
Strain gauges were attached on both longitudinal and transverse steel to measure the bending and shear as well as confining strains respectively. The locations of strain gauges and their test results are described elsewhere [12].
(2) Linear variable displacement transducers (LVDTs)
All the LVDTs are described in Figure 3. Seven pairs of LVDTs were installed on both the extreme tension and compression fibres of the column test specimens. The pair of LVDTs located at 25mm above the beam-column interface were used to obtain the maximum column curvature. One LVDT (±150mm stroke) was installed at the column tip to measure column lateral deflections.
(3) Load cells
A built-in load cell was available in each of the MTS servo hydraulic actuators to measure the load applied. An external load cell was installed on the hydraulic actuator that applied axial load to the column.

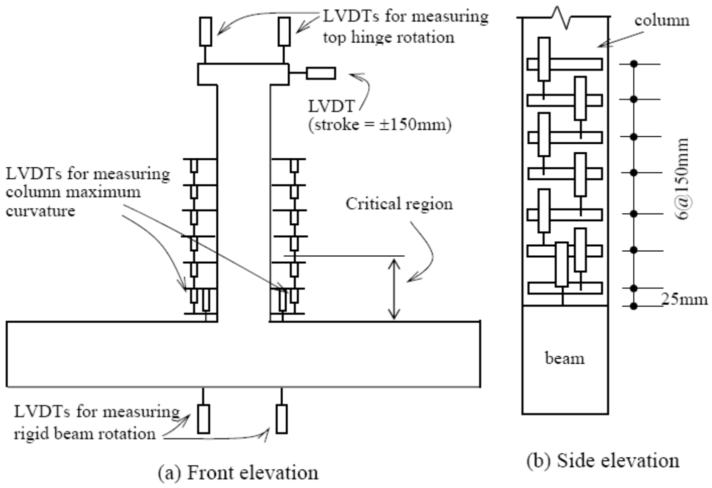


Figure 3 LVDT arrangement

2.3. Test Procedure

The first cycle was load-controlled, in which the column was loaded to subsequently $+0.75M_u$ and $-0.75M_u$, where positive indicates clockwise direction and M_u is the column flexural strength calculated according to the Design Guidance for HSC [13]. The lateral displacements at the column tip were recorded as Δ_1 and Δ_2 respectively, where the nominal yield displacement Δ_y was determined by the following equation:

$$\Delta_y = \frac{4}{3}\left(\frac{\Delta_1 + |\Delta_2|}{2}\right) \tag{2.2}$$

The subsequent cycles were displacement-controlled. In the second cycle, the lateral displacements at the column tip were increased to $+\Delta_y$ and $-\Delta_y$ to reach $\mu = +1$ and -1 respectively, where μ is the nominal displacement ductility factor defined as:

$$\mu = \Delta / \Delta_y \tag{2.3}$$

In Equation (2.3), Δ is the measured lateral displacement at the column tip. Starting from the third cycle reaching $\mu = 2$, the column was subjected to two full cycles. At the completion of every two cycles, μ was increased by one. The process was repeated until the measured moment capacity was smaller than 80% of the maximum measured flexural capacity.

2.4. Test Observation

During the first load-controlled elastic cycle for Units 1 and 2, no flexural crack was formed, and the first flexural cracks occurred on the respective extreme tension fibres at the second cycle when $\mu = \pm 1$. For Unit 3, which was subjected to the lowest axial load level, the first flexural cracks occurred in the first elastic cycle when $\mu = \pm 0.75$. The average spacings for these cracks for all the columns are summarised in Table 2.

In Units 1 and 2, concrete compression crushes at the respective extreme compressive fibre took place when the lateral displacement at the column tip was increased on the way to reach $\mu = \pm 2$. However for Unit 3, the concrete cover spalled later in the cycle of $\mu = \pm 3$. As the lateral displacement increased in successive inelastic cycles, the concrete cover continued to spall so that finally the longitudinal steel buckled owing to the loss of anchorage provided by the concrete cover. The first visible signs of compression spalling and longitudinal steel buckling are summarised in Table 2 for all the columns in terms of column drift (δ) and μ.

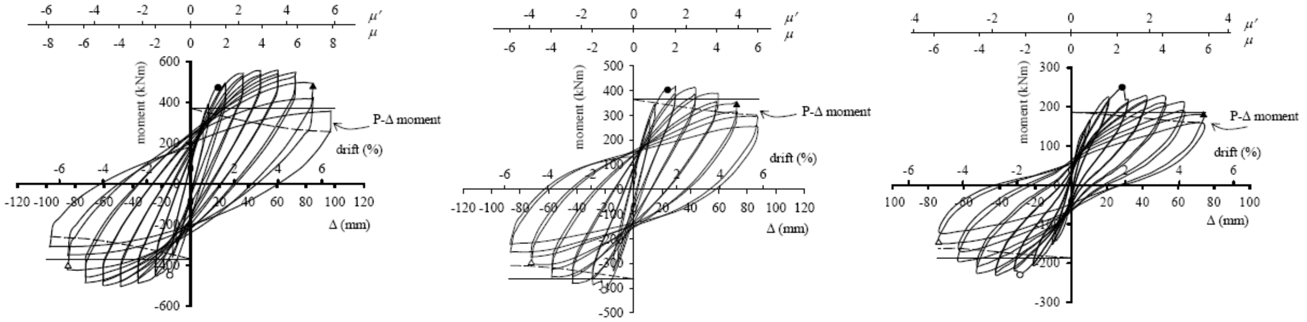
Table 2 Summary of crack spacing, δ and μ at first cover spalling and steel buckling

Unit	Specimen code	Average crack spacing (mm)	At first concrete cover spalling		At first longitudinal steel buckling	
			δ (%)	μ	δ (%)	μ
1	100-06-61-C	106	+1.1	+1.4	+5.6	+7×1
2	80-03-24-C	95	+1.5	+1.6	+4.8	+5×1
3	80-01-09-S	100	+2.0	+2.7	+4.9	+6×1

3. TEST RESULTS

3.1. Moment – lateral displacement and moment-curvature hysteresis curves

The moment – lateral displacement and moment-curvature hysteresis curves of the column specimens are shown in Figures 4 and 5 respectively. The theoretical moment capacity calculated based on the Design Guidance for HSC [13], M_u, is shown as a solid horizontal line in Figure 4, where the drop between this line and the dotted line refers to the secondary moment due to P-Δ effect. Also shown in Figure 4 are the scales of nominal displacement ductility factor (μ), given by Equation (2.3), and actual displacement ductility factor (μ'), which will be defined in Equation (3.2).



(a) 100-06-61-C (b) 80-03-24-C (c) 80-01-09-S

Figure 4 Moment – lateral displacement hysteresis curves

From the hysteresis curves presented in Figure 4, it is observed that:

(1) The maximum moment capacity of the column always occurred within inelastic range. Unit 1, which was subjected to the largest axial load, attained its maximum positive and negative moment capacities on the way to reach the first cycle of $\mu = \pm 4$; whereas Units 2 and 3, subjected to respectively moderate and small axial load level, reached their flexural strength in the cycle of $\mu = \pm 2$. The occurrence of maximum moment capacities beyond the elastic cycle is mainly due to the confining effect provided by the transverse reinforcement in the post-elastic range [14].

(2) The proposed content of transverse reinforcement caused all the HSRC columns, which were subjected to a complete range of compressive axial load levels (represented in this study by Unit 1 for high axial load level, Unit 2 for moderate axial load level and Unit 3 for low axial load level), to behave in a limited ductile manner, in that they managed to reach $\mu = \pm 6$ or drift = ±5%.

(3) The results suggested that the amount of transverse reinforcement calculated using Equation (2.1) could prevent brittle failure of HSRC columns having concrete compressive cube strength as high as 100MPa.

From Figure 5, it can be observed that the column curvature increased rapidly after the first two cycles due to: (1) development of flexural cracks, (2) extensive spalling of concrete cover, (3) inelastic buckling of longitudinal steel, (4) large residual strains accumulated in the columns due to inelastic behaviour, and (5) formation of critical region. It is also evident that all the column specimens could reach $\mu_c = 10$ (see Equation 3.2 for definition of μ_c) prior to failure. This phenomenon is similar to the limited ductile behaviour shown in Figure 4 when the displacement ductility factor and drift ratio reached respectively ±6 and ±5%. The limited ductile behaviour of the columns is believed to be contributed by the provision of adequate confinement steel calculated according to Equation (2.1) within the column critical region.

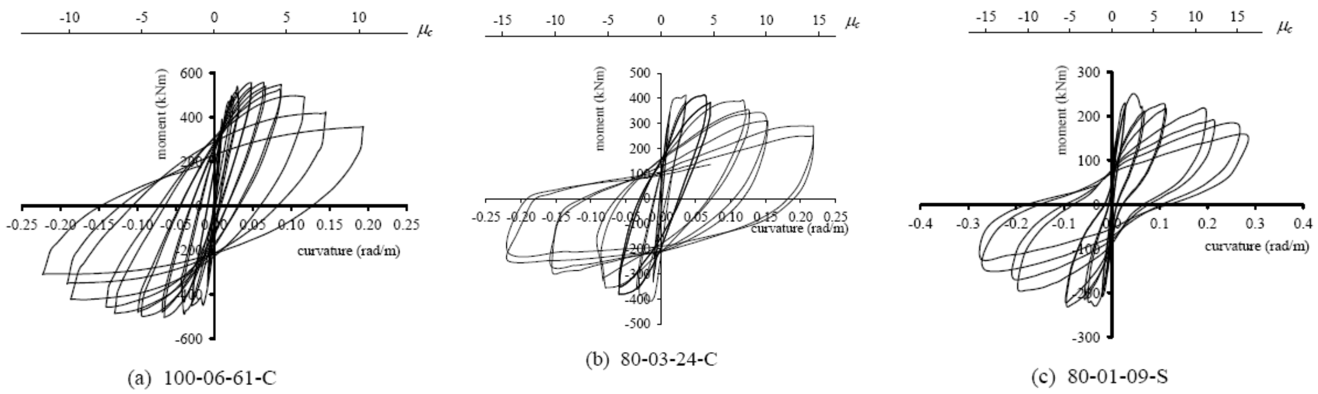


Figure 5 Moment-curvature hysteresis curves

3.2. Ultimate displacement and curvature ductility factors

Displacement and curvature ductility factors are used in this experimental study to evaluate the flexural ductility performance of HSRC columns. Displacement ductility factor refers to the member overall ductility including elastic and plastic deformations outside and within the critical region respectively. Curvature ductility factor, on the other hand, refers to the section ductility, which depends on the section geometry and material strengths.

There are two types of displacement ductility factor used in this study. The first one is nominal displacement ductility factor (μ), which has been explained in Section 2.3 and expressed in Equation (2.3). The second definition is actual displacement ductility factor (μ'), in which the actual yield displacement (Δ_y') is obtained from extrapolation of Δ_1 and Δ_2 (Equation 2.2) at respectively $0.75M_p$ and $-0.75M_p$, where M_p is the measured maximum moment. The actual displacement ductility factor μ' is introduced because M_p is always larger than M_u, and therefore μ tends to overestimate the displacement ductility factor of the column specimens.

To evaluate the flexural ductility performance of the column specimens, ultimate actual displacement ductility factor (μ_d) and ultimate curvature ductility factor (μ_c) have been adopted in this study. Both ductility factors reveal the extent of respectively the lateral displacement at the column tip and the maximum column curvature that could be reached when the flexural strength has degraded by 20% of M_p. The values of μ_d and μ_c can be calculated from Equations (3.1) and (3.2) respectively [15]:

$$\mu_d = \Delta_u / \Delta_y' \tag{3.1}$$

$$\mu_c = \phi_u / \phi_y \tag{3.2}$$

where Δ_u and ϕ_u are the ultimate displacement and ultimate curvature respectively, measured at the column tip when the moment reached $0.8M_p$ in the post-peak range, ϕ_y is the yield curvature, which is equal to 4/3 of the column curvature measured at $0.75M_p$ before reaching the peak moment.

The ultimate actual displacement ductility factor μ_d and ultimate curvature ductility factor μ_c together with their respective yield values are listed in Table 3. It can be observed from the table that the obtained values of μ_c for all the column specimens are fairly close to 10, which could be regarded as the measure of limited ductility. Such design would be most suitable for buildings located in regions having low to moderate seismic risk or in structures that prohibit the development of fully ductile response.

Table 3 Ultimate actual displacement and curvature ductility factors

Unit	Specimen code	Average P/A_gf_{cu}	ρ_s (%)	Δ_y' (mm)	Δ_u (mm)	μ_d	ϕ_y (rad/m)	ϕ_u (rad/m)	μ_c
1	100-06-61-C	0.55	3.20	19.9	89.7	4.5	0.0188	0.1944	10.4
2	80-03-24-C	0.28	2.10	18.4	77.5	4.2	0.0151	0.1481	9.8
3	80-01-09-S	0.11	1.73	18.4	67.1	3.7	0.0175	0.2233	12.8

4. DESIGN GUIDELINES

The following guidelines are proposed for the design of limited ductility HSRC columns:

(1) The transverse steel within the critical region of column should be calculated in accordance with Equation (2.1) and provided with 135° end hooks.

(2) The transverse steel outside the critical region of column could be designed solely to resist the ultimate shear force. Also, the 135° end hooks in the transverse steel could be replaced by 90° hooks.

(3) The following extent of critical region (or critical region length) in columns should be adopted:
   - (a) For $0 \le P/(A_gf_{cu}) \le 0.1$, the critical region length is taken as 1.0 times the greater dimension of the cross-section or where the moment exceeds 0.85 of the maximum moment, whichever is larger;
   - (b) For $0.1 < P/(A_gf_{cu}) \le 0.3$, the critical region length is taken as 1.5 times the greater dimension of the cross-section or where the moment exceeds 0.75 of the maximum moment, whichever is larger; and
   - (c) For $0.3 < P/(A_gf_{cu}) \le 0.6$, the critical region length is taken as 2.0 times the greater dimension of the cross-section or where the moment exceeds 0.65 of the maximum moment, whichever is larger.

5. CONCLUSIONS

The flexural ductility performance of three HSRC columns containing transverse reinforcement designed according to the authors' previously proposed formula was investigated experimentally. The HSRC columns contained concrete cube strength, transverse steel volumetric ratio and longitudinal steel ratio ranging respectively from 86-100MPa, 1.73-3.20% (0.38-1.00% outside the critical region) and 0.9-6.1%. They were subjected to respectively low, moderate and high levels of axial load as well as reversed cyclic inelastic displacement. From the obtained experimental results, it was observed that:

(1) The flexural strength of all the column specimens was larger than their respective theoretical strength due to confinement effect.
(2) The ultimate curvature ductility factor of all the column specimens was about 10, which is considered the standard for limited ductile behaviour. Such design is suitable for regions having low to moderate seismic risk, or where the design of fully ductile structures is not necessary.
(3) Lastly, some design guidelines for limited ductile HSRC columns were proposed.

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