Confinement of columns with non-seismic detailing

Wilson Y.M. Chung and Eddie S.S. Lam

ABSTRACT

In Hong Kong, reinforced concrete structures are traditionally designed without seismic provisions. In particular, non-seismic detailing is specified in the columns, namely with large spacing of links, the use of $90^\circ$ hooks and links not being tied to the main reinforcements. In order to develop a mathematical model to represent this class of columns, confinement of reinforced concrete columns with non-seismic detailing was examined. Axial load tests were conducted on column specimens. Columns with non-seismic detailing have limited confinement action, lesser strength enhancement and limited ductility. It is necessary to refine the confinement model to define the extent of confinement and/or the existing stress-strain relationship of confined concrete to accommodate the type of non-seismic detailing considered in this study.

1. Introduction

In Hong Kong, reinforced concrete structures are traditionally designed without seismic provisions. As Hong Kong is now recognized as a moderate seismic zone, it is necessary to assess the seismic strength of existing structures Lam et al (2002). In particular, columns in Hong Kong are characterized by the high axial load ratio, high main reinforcement ratio, and large spacing of links. Previous studies have indicated that this class of columns has limited ductility Lam et al (2003). In order to develop a mathematical model to represent this class of columns, it is the objective of this study to examine the confinement of reinforced concrete columns with non-seismic detailing.

There exist numerous researches on the confinement of column, especially on the means to increase the strength and ductility of concrete. Among others, Mander et al (1998) proposed a mathematical model to predict the stress-strain relationship of confined concrete. ACI-318 (2002) indicates that the main differences between ductile and non-ductile detailing of columns are the spacing of links and the configuration of links. When the concrete stress increases, there is a progressive increase in the lateral strain. The concrete tends to expand laterally with the formation of micro-cracks. The links interact and provide the passive pressure to confine the lateral expansion and to prevent further cracking of concrete. Strength

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and ductility of concrete are then increased. Therefore, closely spaced links with 135° hooks are specified in the seismic detailing.

![Figure 1: Some typical detailing of columns in Hong Kong.](image)

Figure 1 shows some typical detailing of columns in Hong Kong. Firstly, end hooks of the links are at 90° rather than 135°. Main drawback of this arrangement is that the end hooks may not be able to restrain the main reinforcements from buckling. Secondly, according to the Code of Practice for the Structural Use of Concrete - 1987, “links should be secured to the longitudinal reinforcement and the ends of such transverse reinforcement shall be properly anchored”. There is no further elaboration on how to anchor the links to the main reinforcements. In Hong Kong, reinforced concrete columns are heavily reinforced. For the convenience of concreting, links may not be secured to the main reinforcements. They may be tied to the others links at one ends rather than to the main reinforcements. As a result, the stress-strain relationship of the confined concrete with local detailing could be different from that predicted by Mander et al (1988).

2. Description of the specimens

Table 1 summarizes characteristics of the specimens. Normal strength concrete was considered in all the specimens. In general, the specimens can be divided into three sets. Specimens COL1, COL2, COL3 and COL4 were examined by Lau (1999). Subsequently, Chan (2001) carried out axial load tests on specimens C-1, C-2 and C-3. Specimens 1 to 3 were tested in this study. The specimens considered by Chan (2001) and the present study were 1/4 to the prototype size, whereas 3/20 scale specimens were tested by Lau (1999). Furthermore, 90° hooks were specified in all the links.

Figure 2 shows reinforcement details of the four specimens considered by Lau (1999). The main objective of his study was to differentiate the confinement effect provided by different arrangements of links. In his study, the spacing of links remained the same for all the specimens.

The prototype model considered by Chan (2001) was 800mm square and 2400mm height, and the longitudinal reinforcement ratio was 2.52%. Reinforcement details of
specimens C-2 and C-3 were the same, as shown in Figure 3. They differed in the spacing of links, and in the lateral confinement ratios at 0.093% and 0.279% respectively. Specimen C-2 represents a typical column pursuant to the minimum design requirements of Hong Kong.

Table 1: Characteristics of specimens.

<table>
<thead>
<tr>
<th>Specimen Type</th>
<th>Dimensions (mm)</th>
<th>Cover (mm)</th>
<th>Main Reinforcement</th>
<th>links</th>
<th>( f_{cu} ) (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>COL1, Lau (1999)</td>
<td>150x150x530</td>
<td>6</td>
<td>32R6</td>
<td>φ1.5@45</td>
<td>32.8</td>
</tr>
<tr>
<td>COL2, Lau (1999)</td>
<td>150x150x530</td>
<td>6</td>
<td>32R6</td>
<td>φ1.5@45</td>
<td>66.2</td>
</tr>
<tr>
<td>COL3, Lau (1999)</td>
<td>150x150x530</td>
<td>6</td>
<td>32R6</td>
<td>φ1.5@45</td>
<td>66.2</td>
</tr>
<tr>
<td>COL4, Lau (1999)</td>
<td>150x150x530</td>
<td>6</td>
<td>32R6</td>
<td>φ1.5@45</td>
<td>66.2</td>
</tr>
<tr>
<td>C-1, Chan (2001)</td>
<td>200x200x600</td>
<td>6.25</td>
<td>Plain concrete column</td>
<td></td>
<td>29.05</td>
</tr>
<tr>
<td>C-2, Chan (2001)</td>
<td>200x200x600</td>
<td>6.25</td>
<td>20R8</td>
<td>φ2@75</td>
<td>29.05</td>
</tr>
<tr>
<td>C-3, Chan (2001)</td>
<td>200x200x600</td>
<td>6.25</td>
<td>20R8</td>
<td>φ2@25</td>
<td>29.05</td>
</tr>
<tr>
<td>Specimen 1</td>
<td>200x200x500</td>
<td>10</td>
<td>20T10</td>
<td>φ4@25</td>
<td>33.5</td>
</tr>
<tr>
<td>Specimen 2</td>
<td>200x200x500</td>
<td>10</td>
<td>20T10</td>
<td>φ4@25</td>
<td>33.5</td>
</tr>
<tr>
<td>Specimen 3</td>
<td>200x200x500</td>
<td>10</td>
<td>20T10</td>
<td>φ4@25</td>
<td>33.5</td>
</tr>
</tbody>
</table>

Figure 2: Column details (Lau, 1999). Figure 3: Specimens C-2/C-3 (Chan, 2001).

Figure 4 shows reinforcement details of the three specimens tested in this study. Maximum aggregate size of the prototype column is assumed to be 20mm, so that maximum aggregate size of the specimens is limited to 5mm. In specimen 1 (and also for specimen
COL2), the main reinforcements were restrained by links in one direction only in order to examine the effectiveness of the confinement. In specimen 2 (and also for specimens COL3 and COL4), some of the links were tied to other links rather than to the main reinforcements. All the links in specimen 3 (and also for specimen COL1) were tied to the main reinforcements.

![Figure 4: Configuration of specimens 1 to 3.](image_url)

### 3. Test setup

Figures 5, 6 and 7 show the arrangement of the reinforcements along the length of the specimens. Spacing of links at the top and bottom ends of the specimen is reduced so that failure will occur near the mid-height of the specimen.

Strain gauges were installed on the reinforcements to measure the longitudinal strains and the transverse strains, see Figures 6 and 7. Longitudinal displacements were measured by four linear variable displacement transducers (LVDT), see Figure 8. Concrete strains were measured by two pairs of strain gauges installed on the surface of the specimens, see Figure 8.

![Figure 5: Typical Detailing.](image_url) ![Figure 6: Reinforcements and strain gauges.](image_url)
To ensure full contact was achieved between the surfaces of the loading pad and the specimen, a 20mm thick hard wood was placed between the specimen and loading machine Chan (2001). Alternatively, in this study capping compounds were applied at the two ends of the specimen.

Tests were conducted using a universal loading machine with 200 ton loading capacity. Loading was terminated when the loading capacity of a specimen was reduced by more than 20% from the maximum, or whenever obvious sign of collapse was observed.

4. Experimental Results

Compressive strengths of the specimens were estimated based on the cube strength of concrete, and cylinder strength of concrete was assumed to be 80% of the cube strength Neville (1995).

Specimen C-1, or the plain concrete specimen, failed in a brittle manner and in the presence of a large diagonal crack at 24 degree to the longitudinal axis of the specimen. Similar modes of failure were observed on all the other specimens. Failure was initiated by the spalling of concrete. For specimens COL2 and specimen 1, spalling of concrete was first observed along the two surfaces with no internal links being tied to the main reinforcements. Subsequently, the main reinforcements buckled together with some degree of opening up of the 90° hooks.

Figure 9 plots the stress-strain relationships of the specimens C-1, C-2 and C-3. Also shown in the figures are the respective predictions according to Mander et al (1998). Substantial increase in the ductility was achieved by reducing the spacing of links from 75mm to 25mm. However, spacing of links does not have a significant effect to the loading capacity.
The stress-strain relationships proposed by Mander et al (1988) do not provide reasonable agreement with the experimental results.

The stress-strain relationships of the specimens COL1, COL2, COL3 and COL4 can be found elsewhere (Lau, 1999) and are not repeated herein.

Figure 9: Stress-strain relationships of specimens C-1, C-2 and C-3.

Figure 10: Modes of failure of specimens 1, 2 and 3.

Figure 10 shows the modes of failure of specimens 1, 2 and 3. Strains of the main reinforcements were used to estimate the extent of eccentricity under progressive increase in
the axial load. Assuming the eccentricities of the axial load $P$ at two perpendicular directions are $e_x$ and $e_y$, we can estimate the stress in a main reinforcement at the coordinates $x$ and $y$ by

$$\sigma = E_s \varepsilon = \frac{P}{A} + \frac{Pe_x}{I_x} + \frac{Pe_y}{I_y}$$  \hspace{1cm} (1)

$E_s$ is the modulus of elasticity of the reinforcements and $P$ is the applied force. $A$, $I_x$ and $I_y$ are the gross cross-sectional area and the respective second moments of area at the $x$ and $y$ axes. Equation (1) was applied to all the main reinforcements. The eccentricities were obtained by minimizing the error in predicting the reinforcement stresses at each stress level. The eccentricities of the specimens 1, 2 and 3 are in the range of 3mm to 5mm.

![Stress-strain relationships of specimens 1, 2 and 3.](image)

Figure 11 shows the stress-strain relationships of specimens 1, 2 and 3 obtained from the experiment and from those predicted by Mander et al (1988). In producing these plots, the stress is based on the maximum stress of concrete $\sigma$ and the strain is the strain estimated by the LVDT. There are large discrepancies between the experimental results and the predictions. Furthermore, ultimate strains of the specimens are significantly lesser than those achieved by columns with proper seismic detailing.

Table 2 compares the loading capacities of the specimens. $N$ is the ultimate load and $P_u$ is a reference loading capacity estimated by the following equation.
Here, $f'_c$, $A_c$, $f_y$ and $A_{st}$ are the compressive strength of concrete, net cross-sectional area of concrete, yield stress of the main reinforcements and area of the main reinforcements respectively. $\sigma$ is the maximum concrete stress obtained from the experimental data based on Equation (1), or the average concrete stress N/A, where $A = A_c + A_{st}$, for the specimens COL1, COL2, COL3 and COL4. $\sigma_m$ is the confined concrete stress according to Mander et al (1988).

Table 2: Loading capacities of the specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>N</th>
<th>$P_u$</th>
<th>N/P_u</th>
<th>$\sigma$</th>
<th>$\sigma_m$</th>
<th>$\sigma/\sigma_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>COL1, Lau (1999)</td>
<td>700</td>
<td>934</td>
<td>0.75</td>
<td>31.11</td>
<td>36.88</td>
<td>0.84</td>
</tr>
<tr>
<td>COL2, Lau (1999)</td>
<td>575</td>
<td>1668</td>
<td>0.34</td>
<td>25.56</td>
<td>56.49</td>
<td>0.45</td>
</tr>
<tr>
<td>COL3, Lau (1999)</td>
<td>1000</td>
<td>1668</td>
<td>0.60</td>
<td>44.44</td>
<td>65.60</td>
<td>0.68</td>
</tr>
<tr>
<td>COL4, Lau (1999)</td>
<td>1150</td>
<td>1668</td>
<td>0.68</td>
<td>51.11</td>
<td>64.70</td>
<td>0.79</td>
</tr>
<tr>
<td>C-1, Chan (2001)</td>
<td>1030</td>
<td>1162</td>
<td>0.89</td>
<td>32.56</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>C-2, Chan (2001)</td>
<td>1225</td>
<td>1622</td>
<td>0.76</td>
<td>33.59</td>
<td>34.20</td>
<td>0.98</td>
</tr>
<tr>
<td>C-3, Chan (2001)</td>
<td>1125</td>
<td>1622</td>
<td>0.69</td>
<td>37.23</td>
<td>38.60</td>
<td>0.96</td>
</tr>
<tr>
<td>Specimen 1</td>
<td>1456</td>
<td>2010</td>
<td>0.72</td>
<td>33.09</td>
<td>45.59</td>
<td>0.72</td>
</tr>
<tr>
<td>Specimen 2</td>
<td>1807</td>
<td>2010</td>
<td>0.89</td>
<td>37.88</td>
<td>50.89</td>
<td>0.74</td>
</tr>
<tr>
<td>Specimen 3</td>
<td>1850</td>
<td>2010</td>
<td>0.92</td>
<td>41.70</td>
<td>50.89</td>
<td>0.82</td>
</tr>
</tbody>
</table>

In comparing the N/P_u ratios of COL1, COL2 and COL3 and of specimens 2 and 3, the N/P_u ratios obtained from specimens with some of the links not tied to the main reinforcements are lesser than those obtained from specimens with all the links tied to the main reinforcements. As indicated by the $\sigma/\sigma_m$ ratios, the confined concrete stresses estimated by Mander et al (1988) are not in good agreement with the experimental results. Confined stresses based on the experimental results are lesser than those estimated by Mander et al (1988), except for the specimens considered by Lau (1999) which are essentially high strength concrete specimens (Mandis et al, 2000).

5. Conclusions

This study serves to illustrate that the degree of confinement provided by non-seismic details based on the use of 90° hooks and having links not tied to the main reinforcements. Columns with non-seismic detailing have limited confinement action, lesser strength enhancement and limited ductility. It is necessary to refine the confinement model to define the extent of confinement and/or the existing stress-strain relationship of confined concrete to accommodate the type of non-seismic detailing considered in this study.
6. Acknowledgement

The authors would like to thank for the financial assistance provided by The Hong Kong Polytechnic University. The work reported in this paper is based on the efforts of the staff and researchers from the Department of Civil and Structural Engineering of the Hong Kong Polytechnic University. Their invaluable contribution is gratefully acknowledged.

7. References

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