Strengthening of reinforced concrete beam-column joints using chamfers

*Eddie Siu-shu Lam\textsuperscript{1)}, Kam-leung Tse\textsuperscript{2)}, Siu-sum Leung\textsuperscript{2)}, Ivan Man-lung Sham\textsuperscript{3)}, Jeffery Yuet-kee Lam\textsuperscript{3)}, Shuai Fang\textsuperscript{3)} and Bo Li\textsuperscript{4)}

\textsuperscript{1)} Department of Civil & Environmental Engineering, PolyU, Hong Kong, China
\textsuperscript{2)} Architectural Services Department, the HKSAR Government, Hong Kong, China
\textsuperscript{3)} Nano and Advanced Materials Institute Limited, Hong Kong, China
\textsuperscript{4)} Department of Civil Engineering, The University of Nottingham Ningbo China, Ningbo, China
\textsuperscript{1)} cesslam@polyu.edu.hk

ABSTRACT

Failure of reinforced concrete beam-column joints (“BCJ”) in a moment-resisting structure may cause partial, if not an overall, collapse of the structure. Hence, adequate joint strength has to be properly ensured. As BCJ designed according to non-seismic design codes (e.g. old design codes) have no seismic provisions, they exhibit potential joint-shear failure when subjected to seismic action and strengthening is inevitable.

This paper represents part of an on-going effort to strengthen non-seismically designed BCJ by installing chamfers at beam-column corners and reports the main findings based on the experiments conducted in early 2017. Specifically, four 2/3-scale BCJ, comprising one control specimen and three strengthened specimens, were tested to failure under quasi-static cyclic loading. They were subjected to moderate level of axial load with axial load factor at 0.25. Parameters to be considered included chamfers with and without reinforcements and size of chamfers. It has been shown that chamfers are effective to protect a non-seismically designed BCJ against failure at joint core. Mode of failure is shifted from joint-shear in the control specimen to column-flexure in the strengthened specimens. To enhance the performance of BCJ, size of chamfer is more crucial in comparison with reinforcements in chamfers.

1. INTRODUCTION

In Hong Kong, there is no provisions on seismic resistant design of buildings (Lee et al. 2000; Lam et al. 2002). Our buildings are traditionally designed to gravity loads and wind load only. Since the implementation of Code of Practice for Structural Use of Concrete in 2004 (and the same applies to subsequent revisions including the latest 2013 edition), certain requirements on seismic detailing have been introduced. For instance, joint shear reinforcement is specified in beam-column joints (“BCJ”). This
is based on unbalanced gravity load in beams when not subjected to seismic force as per required under NZ3101: Part 1: 1995 Concrete Structures Standard. As Hong Kong is recognized as a region of moderate seismic risk with peak ground acceleration at 0.15g in accordance with GB50011-2010, Chinese Code for Seismic Design of Buildings, the HKSAR Government is planning to implement seismic resistant design for buildings (Legislative Council Panel on Development 2014). In the forthcoming future, there will be in need of strengthening programs to upgrade our building stock, especially those designed according to the pre-2004 concrete codes.

In Hong Kong, there are thousands of low-rise buildings, like hospitals, facilities for public transportation, police stations, fire stations, etc. These are reinforced concrete frames and most of them were designed to the pre-2004 concrete code. Thus, BCJ are absence of joint shear reinforcement. As key structural elements in frame structures, shear failure of BCJ under seismic action has to be prevented.

Over the years, many strengthening techniques were proposed to upgrade non-seismically designed BCJ. A recent review is referred to Li et al. (2015a). Concrete jacketing (e.g. Hakuto et al. 2000; Karayannis et al. 1998) and shotcrete jacketing (Tsonos 2010) require substantial increase in member size and may not be feasible to implement. To reduce the increase in member size, Beschi et al. (2012) applied high performance fiber reinforced concrete jacketing (only 30-40 mm thickness) to external BCJ. When strength limit of jacket was reached at about 0.5% drift, horizontal load dropped significantly and load carrying capacity was reduced to a level compatible to that of non-jacketing specimens. Despite, ductility and energy dissipation were improved by 100% and 30%, respectively. Steel jacketing (e.g. Ghobarah et al. 1997) requires additional protection against fire and corrosion. Fiber reinforced polymer wrapping (e.g. Ghoborah and Said 2002; Li and Chua 2009; Li and Kai 2011) has eliminated most of the above-mentioned limitations. It fails to perform under moderate temperature and has poor fire rating. Sasmal et al. (2011) proposed a combination of the above. Eom et al. (2015) incorporated special detailing to relocate the plastic hinges but it is also subjected to the above-mentioned limitations. Other alternatives include metallic haunches (Pampanin et al. 2006), passive energy dissipation devices (Chung et al. 2009), shape memory alloys (Dolce et al. 2000), etc.

In recognizing the requirement on fire rating and to minimize increase in member size, concrete jacketing has been revisited and evolved to local enlargement. Chaimahawan and Pimanmas (2009) strengthened two BCJ by incorporating rectangular or triangular/chamfer expansion. Shear failure was suppressed. Noticeably, performance of triangular expansion was almost the same as that of rectangular expansion. Subsequently (Chaimahawan and Pimanmas 2010), triangular expansion was modified with thickened ends to improve confinement and performance was comparable to that of earlier studies. Other alternatives include pre-stressed steel angles for joint enlargement (Shafaei et al. 2014) and ferrocement jackets with diagonal reinforcement (Li et al. 2013, 2015a-b). Li et al. (2015c) applied chamfers to external BCJ. As shown from the test results, chamfers could be provided without skeleton reinforcement. Further, external BCJ with one chamfer exhibited slight decrease in peak strength and energy dissipation as compared to two chamfers. This sheds light on an attractive strengthening scheme using only one chamfer under the soffit of each beam so as to minimize the impact on the use of space.
In this study, an effective and viable method to strengthen interior BCJ is proposed by installing chamfers at the soffit of beams. This paper represents part of an on-going effort to upgrade non-seismically designed BCJ using chamfers and reports the main findings based on the experiments conducted in early 2017.

2. EXPERIMENTAL PROGRAM

The study began with a survey on a number of government buildings. These were all cast in-situ low-rise reinforced concrete structures with building height ranging from three to six stories. Typical BCJ of these buildings were analyzed to select a representable interior BCJ for the experimental program. Criteria of selection are based on joint shear stress greater than joint shear capacity and geometries typical to these buildings.

Based on the record drawings, a typical interior BCJ P9 at the 1st floor of TEC building (better known as Trauma & Emergency Centre) is selected. As shown in Figure 1, lateral load resisting system is contributed by a combination of beam-column frames and shear walls in proportional to relative stiffness.
Figure 2 shows the 1st floor framing plan and the corresponding reinforcement details. Joint shear stress is computed based on joint configuration and reinforcement details. It is found to be much greater than joint shear capacity and joint shear failure will likely occur prior to beam/column failure.
Figure 3 shows geometry of the as-built BCJ P9. Recognizing the geometric limitation (beam span and column height) and loading capacity of the multi-purpose testing system available to the study, a scale factor of 2/3 is applied with adjustment of dimensions and reinforcement details.

Figure 3 Geometry of as-built BCJ P9

Figure 4 Specimen IJ-NC (unit: mm)
2.1 The specimens

There are four specimens representing an interior BCJ of a sub-frame with points of contra-flexure at the supports similar to the as-built drawing as shown in Figure 3. These are the respective control specimen IJ-NC and three strengthened specimens IJ-C150R, IJ-C300R and IJ-C300WR.

Dimension and reinforcement details of the specimens are given in Figure 4. Columns are 2,585 mm height with a cross section of 300 mm × 300 mm. Beams are 3,100 mm long with a cross section of 275 mm × 400 mm (breadth x depth). 20 mm diameter (T20) and 25 mm diameter (T25) deformed bars are used as longitudinal reinforcements. Plain bars with diameters of 8 mm (R8) and 6 mm (R6) are used as stirrups and reinforcements in chamfers, respectively. Measured strength of reinforcements are listed in Table 1. Transverse reinforcement is not provided in the joint zone. Concrete cover is 25 mm to stirrups.

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Yield strength (MPa)</th>
<th>Ultimate strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T25</td>
<td>557.1</td>
<td>662.6</td>
</tr>
<tr>
<td>T20</td>
<td>586.0</td>
<td>684.6</td>
</tr>
<tr>
<td>R8</td>
<td>387.9</td>
<td>462.9</td>
</tr>
<tr>
<td>R6</td>
<td>461.0</td>
<td>528.2</td>
</tr>
</tbody>
</table>

Ready-mixed concrete at 150mm slump is used. All specimens are air-cured after demoulding. Compressive strength of concrete $f_{cu}$ of each specimen is estimated by testing 100mm cubes at the date of testing. The results are listed in Table 2.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>IJ-NC</th>
<th>IJ-C150R</th>
<th>IJ-C300R</th>
<th>IJ-C300WR</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{cu}$ (MPa)</td>
<td>53.2</td>
<td>65.9</td>
<td>57.3</td>
<td>67.1</td>
</tr>
</tbody>
</table>

2.2 Method of strengthening

The strengthening scheme proposed in this study is by means of installing chamfers at the corners of BCJ. To minimize the impact to building plans, chamfers are installed at the soffit of beams. Two parameters are considered and these are the respective dimension of chamfers ($L_c$) and with (R) or without (WR) reinforcement inside the chamfers. Here, $L_c$ is based on the least dimension of beam depth and column width. In this study, $L_c$ is taken as either 300 mm (i.e. column width) or 150 mm (i.e. half column width). R6 U-bars at 50mm spacing are installed inside the chamfers as reinforcement.

As shown in Figure 5, strengthened specimens are identified as IJ-C150R, IJ-C300R and IJ-C300WR for the respective 150 mm chamfers with reinforcement, 300 mm chamfers with reinforcement and 300 mm chamfers without reinforcement.
Chamfers are casted using mortar with w/c at 0.45. Mix proportion of mortar is given in Table 3. It achieves a slump flow of 210mm.

Table 3 Mix proportion of mortar

<table>
<thead>
<tr>
<th></th>
<th>OPC</th>
<th>Silica Fume</th>
<th>Sand</th>
<th>Water</th>
<th>Superplasticizer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>0.05</td>
<td>2.5</td>
<td>0.45</td>
<td>0.016</td>
</tr>
</tbody>
</table>

Flexural and compressive strength of mortar are measured at the date of testing and the results are given in Table 4.

Table 4 Measured flexural and compressive strength of mortar

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Flexural strength (MPa)</th>
<th>Compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>IJ-C150R</td>
<td>6.8</td>
<td>44.8</td>
</tr>
<tr>
<td>IJ-C300R</td>
<td>6.3</td>
<td>42.5</td>
</tr>
<tr>
<td>IJ-C300WR</td>
<td>7.5</td>
<td>54.0</td>
</tr>
</tbody>
</table>

Method of strengthening is illustrated in Figure 6 taking IJ-C150R as an example and a brief description is as follows.

1. Concrete cover of joint core and 400mm length from the adjacent beams and columns is removed.
2. Surface preparation is by chiseling the exposed surface using an impact hammer (Figure 6(a)).
3. R6 U-bars at 50mm spacing are installed (Figure 6(b)).
4. Acrylic formwork is installed and mortar is applied and compacted by hammering (Figure 6(c)).
5. Acrylic formwork is dismantled 24 hours after casting and chamfers are air-cured.
2.3 Test setup and instrumentations

Figure 7 shows the test setup of a multi-purpose testing system (Li et al. 2013). Beams were connected to the strong floor through two hinges one at each end to restrain the vertical displacement. Bottom column was hinged to the strong floor. Axial load and horizontal displacement/force at upper column were provided by two actuators and monitored by two load cells and one LVDT.
Figure 8(a)-(d) show instrumentation of the respective specimen. Strain gauges were installed on reinforcements close to BCJ and on the surface. A pair of LVDTs was installed diagonally on BCJ to monitor the shear deformation. Four pairs of LVDTs were installed on the upper column to estimate the curvature.
2.4 Loading sequence

After imposing the axial load, each specimen was tested under reversed horizontal displacement. Firstly, axial load at $0.25f_{cu}A_g$ was applied vertically on the upper column at a rate of 100 kN/min. It was kept constant throughout the test. Here, $f_{cu}$ is compressive strength of concrete at the time of testing and $A_g$ is gross cross-sectional area of column. Subsequently, horizontal force was applied to reach 75% moment capacity of beam. The corresponding horizontal displacement was defined as $\Delta_{0.75}$. Yield displacement is $\Delta_y = \Delta_{0.75}/0.75$ based on linear extrapolation. Strain of longitudinal reinforcements was also checked against yielding. Afterwards, horizontal displacement was applied at increments of $0.5\Delta_y$ at pre-peak stages and at increments of $\Delta_y$ at post-peak stages, respectively. Each cycle was repeated twice at each displacement increment. Test was terminated when horizontal force reduced to 85% from the maximum. Post-peak displacement at 85% peak value is regarded as the ultimate deformation (Li et al. 2015b).

3. EXPERIMENTAL RESULTS

In what follows, drift ratio is the ratio of horizontal displacement at the tip of upper column to distance between the supports of columns or 2600 mm as shown in Figure 7.

3.1 General behaviour and modes of failure

**Specimen IJ-NC**

Flexural cracks were first observed in the beams and vertical cracks appeared in the joint. At a drift ratio of 0.82%, diagonal cracks developed in the joint. With progressive increase in the drift ratio, more flexural cracks emerged on the beams and the existing cracks propagated. When the horizontal force reached its peak value, diagonal cracks were fully developed in the joint and the specimen failed by joint shear failure.

**Specimen IJ-150R**

Cracks were first observed at the beams. When the drift ratio was increased to 0.39%, cracks developed at the corners of chamfers and propagated into the joint. When the horizontal force approached its peak value, there was spalling of mortar at upper column. When the drift ratio reached 2.17%, failure of upper column was observed.

**Specimen IJ-300R**

Flexural cracks were first observed at the beams and at the corners of chamfers. When the drift ratio reached 0.72%, one crack was observed in the joint. Subsequently, more cracks were generated at the corners of chamfers. They propagated and intersected with diagonal cracks in the joint. When the drift ratio reached 1.7%,
longitudinal cracks appeared on the mortar at upper column. This was followed by spalling of mortar and failure of column.

**Specimen IJ-300WR**

Compared to specimen IJ-300R, cracks extended rapidly in the chamfers probably due to the absence of U-bars inside the chamfers and propagated to the beams and the joint. Cracks originated from one corner of chamfers developed horizontal along beam-chamfer interface. When the horizontal force reached its peak value, spalling of mortar was observed at the upper column and extended into the joint. When the drift ratio reached 1.8%, bottom reinforcements of beams yielded. Specimen failed by a combination of column failure and flexural failure of beams.

![Figure 9 Condition of specimens at failure](image)
3.2 Hysteresis behaviour

Since constant axial load $P_v$ was applied throughout the loading history, additional horizontal force $\Delta P_h$ was induced on BCJ due to the $P-\Delta$ effect. $\Delta P_h = P_v \delta / L$, where $\delta$ is horizontal displacement and $L$ is length of column. Figures 10(a)-(d) plot horizontal force against lateral displacement at column tip. As shown, $\Delta P_h$ is significant at large drift ratio.

As compared with specimen IJ-NC, specimen IJ-C300R achieved higher peak load and higher stiffness. It also experienced slower degeneration of stiffness in post-peak stage. In specimen IJ-C300R, shear failure of BCJ was prevented by chamfers. This resulted in improving the loading capacity of BCJ, enhancing the stiffness and reducing the peak deformation.

Specimen IJ-C300WR performed better than specimen IJ-C150R in reaching higher strength and higher stiffness even though there was no U-bar in the chamfers of the former. This suggests that dimension of chamfer $L_c$ is a dominant factor affecting the performance of a strengthened BCJ. As compared with $L_c$, chamfers with or without reinforcement is of secondary importance.
In recognising the difference in concrete strength, adjustment is made to the horizontal force acting on specimen X by the following equation.

Equivalent horizontal force = horizontal force of specimen X × \( \frac{f_{cu,NC}}{f_{cu,X}} \)  

Here, \( f_{cu,X} \) and \( f_{cu,NC} \) are compressive strength of concrete of specimen X and specimen IJ-NC, respectively.

Figure 11 shows envelopes of the hysteretic loops of specimens against equivalent horizontal force.

Figure 11 Envelopes of the hysteretic loops

At the initial stage, the envelopes are close to each other. When the equivalent horizontal force exceeds 70 kN, control specimen suffers progressive loss of stiffness whereas strengthened specimens are able to maintain the stiffness at higher equivalent horizontal force. Highest loading capacities are obtained from specimens IJ-C300WR and IJ-C300R in the direction of pulling and pushing, respectively. Specimen IJ-C150R performs better than control specimen IJ-NC.

Table 5 shows peak values of equivalent horizontal force achieved by the specimens. Loading capacities increase with increasing \( \frac{L_C}{L_{BC}} \) ratio. Here, \( L_C \) and \( L_{BC} \) are the length of chamfer and the depth of beam, respectively. Peak value of specimen IJ-C150R \( (\frac{L_C}{L_{BC}} \) ratio at 0.5 or \( L_C = 150 \) mm) was increased by 8.1%. When \( \frac{L_C}{L_{BC}} \) ratio was increased to 1.0 (i.e. \( L_C = 300 \) mm), peak value was increased by 30% as compared with that of control specimen. Further, peak value of specimen IJ-C300WR was 20% more than that of specimen IJ-C150R.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( \frac{L_C}{L_{BC}} ) ratio</th>
<th>Pull ( \rightarrow )</th>
<th>Push ( \leftarrow )</th>
<th>Average</th>
<th>Enhancement</th>
</tr>
</thead>
<tbody>
<tr>
<td>IJ-NC</td>
<td>-</td>
<td>121.7</td>
<td>-120.1</td>
<td>120.9</td>
<td>-</td>
</tr>
<tr>
<td>IJ-C150R</td>
<td>0.5</td>
<td>136.2</td>
<td>-125.1</td>
<td>130.7</td>
<td>8.1%</td>
</tr>
<tr>
<td>IJ-C300R</td>
<td>1.0</td>
<td>154.6</td>
<td>-159.7</td>
<td>157.1</td>
<td>30.0%</td>
</tr>
<tr>
<td>IJ-C300WR</td>
<td>1.0</td>
<td>165.5</td>
<td>-152.5</td>
<td>159.0</td>
<td>31.5%</td>
</tr>
</tbody>
</table>
4. SUMMARY OF TEST DATA

Table 6 summaries peak loads and failure modes of the specimens.

Peak loads in both pull and push directions of BCJ were enhanced by the installation of chamfers at the soffit of beams. Different from control specimen, peak loads in push or pull directions were unsymmetrical for the strengthened specimens. The occurrence of cracks in a chamfer could affect the load transfer through the chamfer. Additionally, reinforcements installed in the chamfers have marginal influence on peak horizontal force. Peak horizontal force of strengthened specimen without reinforcements (i.e. specimen IJ-C300WR) is higher than that with reinforcements (i.e. specimen IJ-C300R). This is probably attributed to higher strength of mortar used in the chamfers for specimen IJ-C300WR as shown in Table 4.

Failure modes of BCJ were shifted from shear failure in the joint to flexural failure in column/beam. It demonstrates that installation of chamfers at the soffit of beams are effective to prevent shear failure of joint.

Chamfers installed at the soffit of beams remained intact or partially damaged under cyclic loading. This indicates that dimension of chamfer is the dominant factor affecting the performance of a strengthened specimen.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Peak Load (kN)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pull →</td>
<td>Push ←</td>
<td>Members</td>
</tr>
<tr>
<td>IJ-NC</td>
<td>121.7</td>
<td>-120.1</td>
</tr>
<tr>
<td>IJ-C150R</td>
<td>189.8</td>
<td>-155.0</td>
</tr>
<tr>
<td>IJ-C300R</td>
<td>166.5</td>
<td>-172.0</td>
</tr>
<tr>
<td>IJ-C300WR</td>
<td>208.7</td>
<td>-192.3</td>
</tr>
</tbody>
</table>

5. CONCLUSIONS

An experimental investigation on reinforced concrete interior beam-column joints strengthened by chamfers was conducted. Influence of chamfers with or without reinforcements and size of chamfer on the performance of strengthened specimens were evaluated. Based on the test results, the following conclusions could be drawn.

(1) Chamfers installed at the soffit of beams are effective to prevent joint shear failure and to enhance the loading capacity of non-seismically designed reinforced concrete beam-column joints.

(2) Loading capacity of a beam-column joint is enhanced by 30% after installing 300 mm chamfers at the soffit of beams regardless of provision of U-bars in the chamfers.

(3) To enhance the performance of beam-column joints, size of chamfer is more crucial in comparison with reinforcements in chamfers.

(4) Failure mode of a beam-column joint is shifted from joint shear in the control specimen to column/beam flexural in the strengthened specimens. Installation of
chamfers at the soffit of beams can effectively prevent shear failure in the joint core.

ACKNOWLEDGEMENT

The authors wish to express their gratitude and sincere appreciation to the support from the Architectural Services Department of the HKSAR Government. Further, the authors would like to thank Dr Yang Zhi-dong and technical staff of Structural Engineering Research Laboratory, Department of Civil and Environmental Engineering, The Hong Kong Polytechnic University, for their support to the experiments.

REFERENCES


