Structural Performance Evaluation and Strengthening of an Earthquake-Damaged Beam-Column Joint in Indonesia

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Abstract
In 2009, a magnitude 7.6 earthquake stroke West Sumatra, Indonesia and caused severe damage to reinforced concrete (RC) buildings. Failure of poorly-detailed joints was remarkable. In consideration of economic and technical conditions in developing countries like Indonesia, a seismic strengthening method using RC wing walls is proposed aiming to improve the moment capacity of such seismically substandard beam-column joints. In this study, three 3/4-scale substandard exterior beam-column joint specimens were prepared with the same structural details, two of which were strengthened by installing RC wing walls with different strengthening details. The test results showed that the strengthened specimens exhibited ductile failure modes with beam yielding, whereas the benchmark specimen failed at the joint showing a brittle manner. This seismic strengthening approach is applicable for improving the seismic performance of substandard RC beam-column joints.

Keywords: Developing countries; reinforced concrete; retrofitting; unreinforced joint; wing wall

INTRODUCTION
There is a substantial stock of reinforced concrete (RC) buildings with seismically substandard beam-column joints, like those containing little or no transverse reinforcement in the joint regions (unreinforced joints). The following three aspects are considered as the reasons for the existence of such substandard buildings.

- In some older design codes, no requirement on arrangement of shear reinforcement within joint regions were described. Buildings designed and constructed according to these codes still widely exist nowadays, like those constructed prior to the mid-1970’s in the U.S [1].
- According to the seismic codes and structural design drawings, shear reinforcement was directed but never arranged due to rough construction management, especially in developing countries.
- Buildings constructed by the owner, as it called “non-engineered buildings”.

Past studies have illustrated that unreinforced joints showed bad seismic performance like low strength, ductility, and energy dissipation [2]. Joints failure may cause buildings to collapse, as observed in past earthquake disasters. Meanwhile, more moderate or huge earthquakes were observed worldwide in recent years [3 etc.]. Consequently, effective and economic strengthening methods for such buildings are urgently needed.

Several studies have developed effective strengthening methods for unreinforced joints using steel props [4], GFRP [5], etc. However, for developing countries, these methods are not easily implemented in consideration of technical level, available materials, and construction cost. Installing wing walls beside the existing columns as shown in Fig. 1 seems to be a more
practical way of upgrading these seismically substandard buildings. It’s expected to be effective for strengthening not only poorly-detailed joints but also the slender columns which commonly exist in developing countries. However, this paper only focuses on the strengthening effects for the joints by a series of model tests.

![Proposed Strengthening Method by Installing RC Wing Walls.](image)

**Fig. 1:** Proposed Strengthening Method by Installing RC Wing Walls.

**FOCUSED BUILDING AND JOINT**

On the 30th of September 2009, a magnitude 7.6 earthquake stroke the west coast of Sumatra, Indonesia. According to a post-earthquake field investigation [6] conducted by a reconnaissance team of AIJ (Architectural Institute of Japan), a large number of RC buildings including many engineered ones suffered severe damage. It was reported that one of the major causes of building collapse was the joint failure.

This study focused on an exterior beam-column joint of a three-story RC moment-resisting frame structure in Padang near the epicenter. This building collapsed in the earthquake, as shown in Fig. 2. It was built in 2005 with a story height of 3,000 mm and a beam span of 7,000 mm. In the building, damage such as buckling of longitudinal reinforcement and concrete spalling was concentrated in the joints. According to the field investigation [6], there was no transverse reinforcement in the joints, and the hooks of beam/column shear reinforcement were 90 degrees, as shown in Fig. 3.

![Earthquake-Damaged Building and Beam-Column Joint](image)

**Fig. 2:** Earthquake-Damaged Building and Beam-Column Joint [2].
Figure 3: Dimensions and Reinforcing Details of the Structure [6].

**PROPOSED STRENGTHENING MECHANISM**

**Moment Capacity of Beam-Column Joint**

Shiohara [7] proposed a new resisting mechanism for beam-column joints named Nine Parameter Model and demonstrated that the joint strength depends on its moment capacity. The model described the seismic behavior of joints as follows: under seismic loading, in the case of interior joints, two corner cracks and one diagonal crack appears in joint, dividing the joint panel into four rigid plates as shown in Fig. 4(a). Every plate has 3 degrees of freedom (two translations and one rotation), and the total number is 9 except one segment for a basis of comparison. The mechanism is completely different with the traditional shear resisting ones and gave a definition of joint hinging failure. As shown in Fig. 4(b), tension and compression existing in the reinforcing bars crossing the cracks and in the concrete around the rotations of the rigid plates, forming a mechanism to resist the seismic moments subjected to the joint.

Based on the test results of a small-scale (1/3) specimen modeled from the same exterior joint of the building focused on in this study, Sashima [2] illustrated a deformation behavior for exterior beam-column joints as Fig. 5.

![Fig. 4: Nine Parameter Mechanism (some notes in the figure were added by the authors) [8].](image)

(a) Nine parameter model  
(b) Forces to one segment

![Fig. 5: Deformation Behavior of Exterior Beam-Column Joints [2].](image)

(a) Positive loading  
(b) Negative loading
Kusuhara and Shiohara [8] gave a method for calculating the strength for joint hinging failure. In the literature, equations for calculating a strength reduction factor, which is defined as the ratio of the ultimate strength for joint hinging failure to the moment at joint node when beam yields at its critical section, were given separately for interior joints, exterior joints and corner joints.

**Proposed Strengthening Mechanism**

A beam-yielding mechanism is considered as an ideal failure pattern for RC frame structures. Consequently, the joints are always designed stronger than the connected beams and columns. However, if the strength of the joints are overestimated for their substandard details, or the beams/columns are underestimated, the real strength of the joint maybe the lowest, resulting in an unexpected brittle joint failure mode.

This study aims to improve the moment capacity of the weak joints in existing RC structures by installing wing walls. Under seismic loads, the beam tends to deform as Fig. 6. The assumed cracks in joint are also shown in the figure. It seems reasonable that a compressing force \( C \) and a tensile force \( T \) will act on the beam from the compressed and pulled wing wall, respectively, resisting the deformation of beam and expansion of joint cracks like Fig. 5(a).

Moreover, these two actions \( (C \) and \( T) \) can also be expected to improve the joint moment capacity from \( M_{bu} \) to \( rM_{ju} \) as Eq. 1. If the improved joint moment capacity \( rM_{ju} \) is greater than the corresponding node moment when the beam yields at the wall end as Eq. 2, joint failure can be avoided.

\[
rM_{ju} = M_{ju} + C \cdot I_C + T \cdot I_T
\]

\[
rM_{ju} > jM_{bu}
\]

Meanings of the symbols in the equations are illustrated in Fig. 6.

![Fig. 6: Strengthening Mechanism by Installing Wing Walls.](image)

**Notes**

- Assumed cracks at joint region
- Moment diagram along beam when neglecting wall actions
- \( O \): Joint center
- \( I_C \): Distance from \( T \) to \( O \)
- \( C \): Compressive force from wall to beam
- \( I_C \): Distance from \( C \) to \( O \)
- \( T \): Tensile force from wall to beam
- \( M_{bu} \): Beam ultimate flexural moment
- \( jM_{bu} \): Corresponding moment at joint when beam yield at wall end
- \( M_{ju} \): Joint moment capacity

**EXPERIMENTAL PROGRAM**

**Specimens**

Three 3/4-scale model specimens of the exterior joint shown in Figs. 1 and 2 were prepared: J2, J2-W2, and J2-W1. J2 was the benchmark specimen, representing the original frame without retrofitting. It was modeled up to the inflection points of the upper/lower column and beam, however, a special loading method illustrated in Fig. 7 was considered. Figure 7(a) is the normal loading method when conducting exterior beam-column joint model tests. As the focused building possesses long beams, to satisfy the capacity of loading facilities, scale down of specimens is needed which is not desirable for test accuracy. Figure 7(b) shows the proposed
loading method in this study: corresponding to a shear force of $V_b$, an additional moment of $\Delta M$, which is controlled as proportional to $V_b$, is applied to the shortened (from $L_b/2$ to $L_s$) beam tip.

Figure 8 shows the dimensions and reinforcement details of the benchmark specimen. As shown in the figure, no stirrups were arranged in the joint region, representing the focused joint in Fig. 3.

![Proposed Loading Method for Exterior Beam-Column Joints with Long-Span Beams.](image)

\[ M_b = V_b \times \frac{L_b}{2} \]

(a) Normal loading method  (b) Loading method in this study

![Dimensions and Reinforcement Details of the Benchmark Specimen.](image)

J2-W2 and J2-W1 was strengthened by installing wing walls. The wing walls were designed so that the width was 340 mm, almost the same with the height of column. According to the Japanese guidelines for retrofitting RC buildings [9], the thickness of the wing wall should not be less than 200 mm for retrofitting; hence, the thickness of the wing wall was 150 mm for the 3/4-scale specimen. Beam/column anchors connecting the wing walls and the existing beam/column were installed based on the Japanese retrofitting guidelines’ [9] minimum requirements for the spacing between anchors and cover concrete, which resulted in the arrangements shown in Fig. 9. The embedding depths of the beam/column anchors into the existing frame and the wing walls were 150/156 mm ($15/12 \; d_a$, $d_a$: diameter of anchor) and 200/260 mm ($20 \; d_a$), respectively. D10 (deformed rebar with a diameter of 10 mm) rebars were provided for the wing walls with double layers. $\phi$6 spirals were installed at the boundaries between the wing walls and the existing frame to prevent the splitting failure of the concrete. J2-W2 was strengthened at both sides of the beam. To examine the strengthening effectiveness of the pulled and pushed wing wall, only one wall was installed to the lower column for J2-W1.
strengthening works was conducted after two weeks’ curing of the existing frames. Tables 1 shows the compressive strength of cylindrical concrete, and the properties of the rebars.

Table 1: Properties of Concrete, Reinforcement, and Anchor Bolts (N/mm$^2$).

<table>
<thead>
<tr>
<th>Region</th>
<th>$E_c$</th>
<th>$F_c$</th>
<th>$f_t$</th>
<th>Type</th>
<th>$E_s$</th>
<th>$F_y$</th>
<th>$F_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>J2</td>
<td></td>
<td>2.55×10$^4$</td>
<td>20.2</td>
<td>1.9</td>
<td>D16</td>
<td>1.75×10$^5$</td>
<td>373</td>
</tr>
<tr>
<td>J2-W2</td>
<td>Existing</td>
<td>2.57×10$^4$</td>
<td>22.7</td>
<td>2.0</td>
<td>$\phi$9</td>
<td>1.78×10$^5$</td>
<td>344</td>
</tr>
<tr>
<td>walls</td>
<td>2.62×10$^4$</td>
<td>26.9</td>
<td>2.5</td>
<td>D10</td>
<td>1.68×10$^5$</td>
<td>380</td>
<td>554</td>
</tr>
<tr>
<td>J2-W1</td>
<td>Existing</td>
<td>2.80×10$^4$</td>
<td>22.6</td>
<td>2.1</td>
<td>D13</td>
<td>1.65×10$^5$</td>
<td>361</td>
</tr>
<tr>
<td>wall</td>
<td>2.62×10$^4$</td>
<td>27.7</td>
<td>2.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

where, $E_c$: Young’s modulus of concrete, $F_c$: the compressive strength of concrete, $f_t$: the tensile strength of concrete, $E_s$: Young’s modulus of steel, $F_y$: the yield strength of reinforcement, and $F_u$: the ultimate tensile strength of reinforcement. The properties of the concrete and reinforcement are similar to those of the focused building [2].

**Strength Evaluation for Existing Frame by Current Japanese Standard**

The strengths of the existing frame were calculated according to current Japanese standard: flexural strengths of the beam and column by Eqs. 3 and 4 [10], respectively, and shear strength of the joint by Eq. 5 [11]. The shear strengths of the beam and column are considerably larger than their flexural strengths, and are not discussed here.

$$M_{bu} = 0.9a_t\sigma_j d$$  \( (3) \)  

$$M_{cu} = 0.8a_t\sigma_j D + 0.5ND \left( 1 - \frac{N}{bDF_c} \right)$$  \( (4) \)  

$$V_{ju} = \kappa \cdot \phi \cdot F_j \cdot b_j \cdot D_j$$  \( (5) \)  

where, $a_t$: the area of tensile bars, $\sigma_j$: the yield strength of tensile bar, $d$: the effective depth of beam, $D$: the depth of column, $N$: the axial force, $b$: the width of the member, $F_j$: the
compressive strength of concrete, $\kappa$: the joint shape factor (0.7 for exterior joint), $\varphi$: the factor depending on existence of orthogonal beams (0.85 for joint without it), $F_j$: the nominal value for the shear strength of the joint ($F_j = 0.8F_c^{0.7}$), $b_j$: the effective width ($= b$, when the beam and columns have the same width), and $D_j$: the effective depth of the joint.

The calculated ultimate strengths for the beam, column, and joint are 134 kN.m (142 kN.m), 71 kN.m (177 kN.m), and 278 kN (124 kN.m), respectively. Values in parentheses are the conversions to the joint node moment. The moment strength of the joint is the lowest, meaning that joint failure will occur first for the original frame.

**Test Set-up, Instrumentation, and Loading Program**

Figure 10 shows the test set-up. The specimens were installed on the loading facilities, rotated by 90°. The right column (lower floor column) was supported by a pin hinge, and the left was supported by a roller. In order to measure the shear force of the column, a load cell was incorporated into the roller support. Horizontal reversed cyclic displacement was applied to the beam tip. The additional moment $\Delta M$ is fulfilled by a couple of forces from two vertical hydraulic jacks. The loads in the vertical jacks were controlled according to Eq. 6. Axial load was not applied to the column. Beam drift ratio was defined as $R = \delta/L$, where $\delta$ is the horizontal displacement at the beam tip measured by a displacement transducer (Dis. in the figure), and $L$ is the distance between the transducer and joint center. The loading program was shown in Fig. 11. Figure 12 shows the strain gauge arrangement.

$$N = 3.35 \times \left(2.625 - 1.7\right)V_b = 0.264V_b$$

\[\text{(6)}\]
EXPERIMENTAL RESULTS

Figure 13 shows crack patterns to the specimens after the cycling to \( R = 1/200 \), and \( 1/67 \) rad., during which the joint diagonal crack appeared and the ultimate strength was observed, respectively, for the benchmark specimen, together with the final status. The relationships between the joint moment and the beam drift ratio together with the maximum strength are also illustrated in the same figure. The joint moments are the product of the distance between pin centers and the shear force of the column.

**Benchmark specimen, J2**

Diagonal cracks appeared at the joint panel during the cycle to \( R = \pm 1/200 \) rad., as shown in Fig. 13, and then extended along the external longitudinal column bars with increasing plastic deformation. The maximum strength was recorded at \( R = \pm 1/67 \) rad. During the cycle to \( R = \pm 1/25 \) rad., column longitudinal bars exposed and buckled. Finally, the diagonal cracks pierced the outside of the column, and a substantial amount of concrete peeled off.

**Specimen strengthened at two sides of beam, J2-W2**

During the cycle to \( R = \pm 1/200 \) rad., the first beam anchor (ABU/L1, referring to Fig. 12) yielded, and diagonal cracks appeared at the beam end where the walls were attached. During the cycle to \( R = \pm 1/133 \) rad., the beam longitudinal reinforcement yielded at the location of the first anchor (BU/L4), and diagonal cracks appeared at the existing joint panel. Finally, the longitudinal reinforcement of the beam exposed and buckled. The damage concentrated at the beam where the wing walls ended. No obvious damage to the wing walls was observed.
Specimen only strengthened at one side, J2-W1
The main characteristic for J2-W1 was the asymmetry between positive and negative directions. During the cycle to $R = +1/200$ rad., a diagonal crack appeared from the existing joint to the part of the beam where the wing wall was attached. Then, diagonal cracks appeared at the existing joint panel, and in the positive loading, the first beam anchor (ABL4) yielded. During the cycle to $R = +1/100$ rad., the third beam anchor (ABL3) yielded, then the beam reinforcement (BL4)
yielded at the location where the first anchor located, and a crack appeared along the beam depth where beam anchors were buried, as shown in Fig. 14. It seemed to be the start of cone failure of concrete. During the cycle to \( R = -1/50 \text{ rad.} \), when the wing wall was pushed, shear cracks appeared at the wing wall, at an angle of about 45° to the column, as shown in Fig. 14. Finally, the column reinforcement became exposed and buckled. The maximum strengths varied significantly between positive and negative loading, when the wall was pulled or pushed. The damage was also asymmetric.

DISCUSSION

The skeleton curves of the hysteresis loops in Fig. 13, the strengths considering beam yielding and joint failure based on Eqs. 3 and 5, and the ratios of the maximum strengths of strengthened specimens to the control specimen are illustrated in Fig. 15.

![Fig. 14: Cone-Style Failure, and 45° Cracks at Wing Wall for J2-W1.](image1)

![Fig. 15: Skeletons, Calculated Strengths, and Strengthening Effect.](image2)

Failure of J2 was brittle, showing low strength and poor deformability. Its ultimate strength was obviously below the value calculated according to the current Japanese standards [10, 11] (±124, depending on the joint strength), revealing that joint strength of unreinforced joints cannot be exactly evaluated by current Japanese standards.

The failure mode of J2-W2 successfully changed from brittle joint failure to ductile beam yielding, showing high strength, exceeding the beam yield strength (±164, assuming yield at wall ends), and good deformability.

For J2-W1, when the wing wall was pushed, the strength was improved by 81%, about the same as J2-W2. When it was pulled, the strength was improved by 39% compared with J2, reaching the beam yield strength (±142, assuming yielding at column face) of the benchmark specimen, but less than the yield strength at the wing wall end (±164, \( M_{bu} \) in Fig. 6). The reason seemed to be the early development of cone failure of concrete shown in Fig. 14.

CONCLUSIONS

This paper proposed and verified a practical strengthening method using RC wing walls for the unreinforced exterior beam-column joints, aiming to improve their moment capacities. Three partial plain frame specimens were tested and their seismic performance especially the ultimate strengths were evaluated. The major findings were as follows:

1. In the case of the control specimen, brittle damage of the joint was observed. The maximum strength was less than the calculated design value.
2. For the specimen strengthened by installing wing walls to both upper and lower columns, the failure mode changed from brittle joint failure to ductile beam yielding. The strength and deformability were significantly improved. It was verified that the strengthening method effectively prevented a premature failure of the joint.

3. For the specimen strengthened on one side, the strengthening effect was greater when the wing wall was pushed than when that was pulled.

REFERENCES


