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A PRACTICAL METHOD FOR STRENGTHENING/REPAIRING POORLY DETAILED RC BEAM-COLUMN JOINTS IN DEVELOPING COUNTRIES: INSTALLING WING WALLS TO EXISTING COLUMNS

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Abstract

This study develops a practical method for strengthening or repairing poorly detailed reinforced concrete (RC) beam-column joints which contain no transverse reinforcement, by installing RC wing walls beside existing columns. Such substandard beam-column joints were often observed in RC buildings after recent major earthquakes attacked developing countries. Three 1:2.5 scale partial frame specimens were prepared representing an exterior 1.5-story moment resisting frame with beam-column joint of a two-story RC building damaged in the 2013 Bohol earthquake, Philippines. One of them was for the benchmark specimen. Another was strengthened by the proposed strengthening method. The third one was repaired with the same details as the strengthened specimen after moderately damaged by a pre-loading simulating the seismic damage. Reversed cyclic loading tests were conducted. Consequently, the benchmark specimen suffered severe damage to the joint region with buckling of column longitudinal rebars; the strengthened specimen showed a ductile behavior with beam yielding; and for the repaired specimen, the beam yielded and the maximum strength was recorded approximately as high as that of the strengthened specimen, and the damage in joint region was more severe/slighter compared to the strengthened/benchmark one. The test results indicated that the developed method is practical for strengthening poorly detailed beam-column joints or repairing moderately damaged ones.

Keywords: substandard beam-column joints; retrofitting; post-earthquake repairing

1. Introduction

A substantial stock of reinforced concrete (RC) buildings that contain no horizontal reinforcement in beam-column joint regions still exist worldwide. These buildings were designed according to older design codes or constructed under rough construction management, especially in developing countries. Earthquake experiences [1 etc.] and test results [2 etc.] have revealed the potential risks of the seismically poorly detailed joints and urgency for developing practicable strengthening methods for them. However, the limited upgrading schemes reported in past studies [3, 4] are not necessarily practical in developing countries because of the need for advanced materials and the complexity of the construction process. In consideration of the economic situation and technical level of developing countries, installing RC wing walls beside existing columns as shown in Fig. 1 seems to be a practical way of upgrading these seismically substandard buildings. In this study, loading tests were conducted with three specimens, one for the benchmark specimen and the other two for the retrofitted or repaired specimen, to examine the retrofitting/repairing effectiveness of the method.

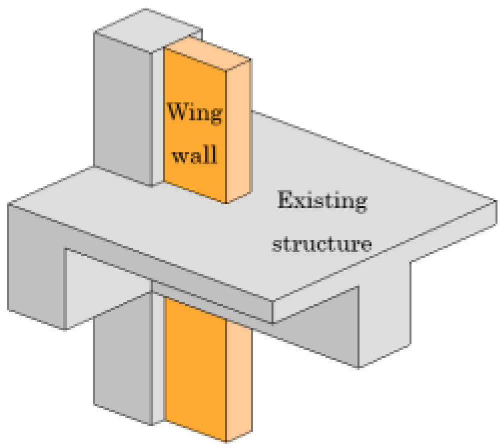


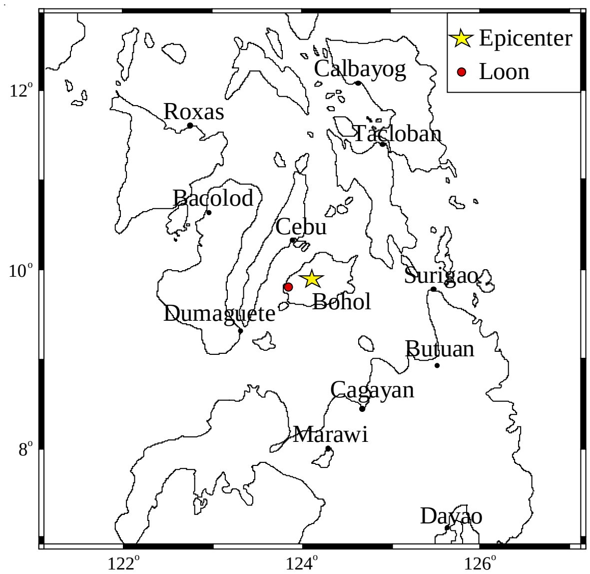
Fig. 1 – Proposed retrofitting/repairing method by installing RC wing walls

2. Target building and joint

On October 15, 2013, Bohol Island, located in the central region of the Philippines, experienced an earthquake. The location of epicenter is shown in Fig. 2, with the details of the earthquake reported by the USGS (U.S. Geological Survey).

According to a post-earthquake investigation by Narafu et al. [5], a number of RC buildings suffered typical damage in this earthquake. The present study focused on a typical RC building (a public market) in Loon. The location of Loon was also shown in Fig. 2. The two-story RC building was built within the past 10 years but was severely damaged by the earthquake. Figure 3 shows the overview of the building. The drawings of the building were provided by the Department of Public Works & Highways (DPWH) and are presented in Fig. 4 and Table 1, which give the first story plan and the details of the columns and beams, respectively. The averaged concrete strength estimated by hammer tests was 39.3 N/mm^2.

The damage to columns in the first story was evaluated in the span direction as shown in Fig. 4, according to the Japanese guidelines for post-earthquake damage evaluation [6]. The Roman numerals in the figure indicate damage classes of columns, while those in parentheses indicate the beam-column joint damage classes. For the resultant residual seismic capacities of the building, refer to the authors' past report [7]. The damage indexes in Fig. 4 reveal that severe damage was observed mainly in the beam-column joints in the first story.



Details of the earthquake

Time: 08:12, 15 October 2013 (local time)

Location: 9.88°N, 124.12°E

Depth: 19.0 km

Fig. 2 – Details of the 2013 Bohol earthquake, Philippines and the location of the target building

Fig. 3 – Overview of the public market building

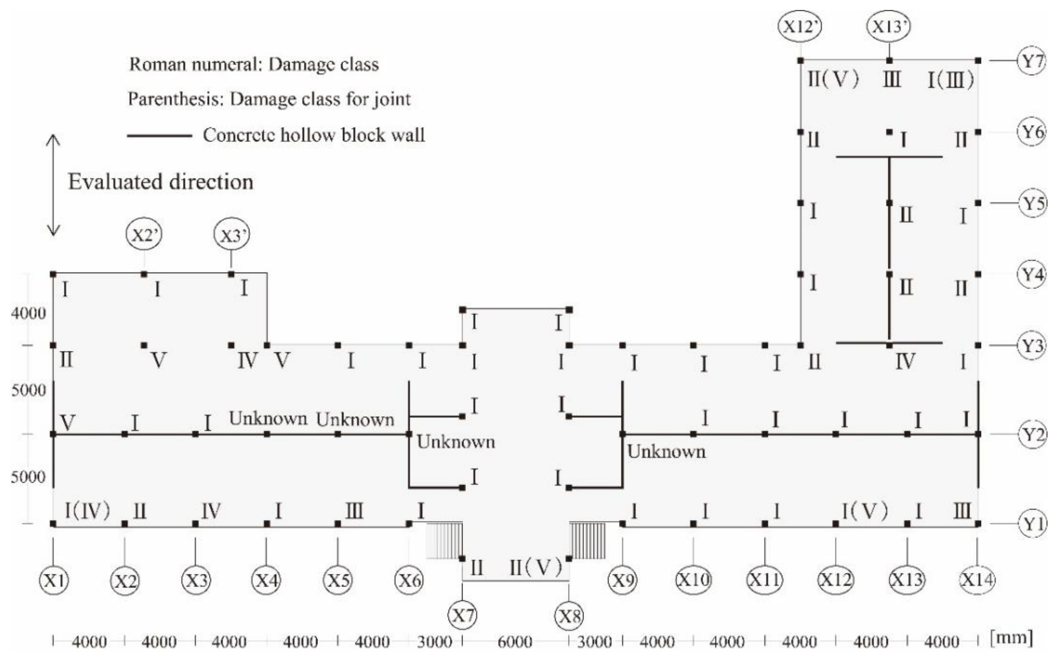


Fig. 4 – First story plan

Table 1 – List of typical columns and beams

Member	Section (mm)	Longitudinal reinforcement	Transverse reinforcement*3
Column 1*1	350×350	4-D20, 8-D16 (p_g = 2.34%)	D10@100 (p_w = 0.44%)
Column 2*1	350×350	12-D16 (p_g = 1.97%)	D10@100 (p_w = 0.44%)
Beam 1*2	250×400	Top: 4-D16 (p_t = 0.92%) Bottom: 2- D16 (p_t = 0.46%)	D10@50 (p_w = 1.26%)
Beam 2*2	250×400	Top: 6- D16 (p_t = 1.38%) Bottom: 3- D16 (p_t = 0.69%)	D10@50 (p_w = 1.26%)
Beam 3*2	250×400	Top: 6-D20 (p_t = 2.15%) Bottom: 3-D20 (p_t = 1.08%)	D10@100 (p_w = 0.63%)

Note: D10, D16 and D20 mean deformed bars with nominal diameters of 10, 16 and 20 mm, respectively; p_g is the ratio of the area of the longitudinal reinforcements to the column cross-section; p_t is the ratio of the area of the tensile longitudinal reinforcements to the beam cross-section; and p_w is the shear reinforcement ratio.
*1 Most of the interior/exterior columns are Column 1/Column 2.
*2 Most of the interior/exterior beams in the longitudinal direction are Beam 1/Beam 2, and most of the beams in the span direction are Beam 3.
*3 Data are values at the member ends.

Figure 5 shows typical damage to the beam-column joint at X12-Y1 in Fig. 4, whose structural details are shown in Fig. 6. No hoop was provided in the joint region despite the design requirements of the Philippine building code [8]; hence, severe damage was observed in the core concrete, and the actual frame performance was lower than that expected for the design assuming beam yielding. A lack of hoops in the joint panel is commonly observed in earthquake-damaged RC buildings in developing countries [1 etc.]. Therefore, the following experimental study focused on a partial exterior frame representing the above building to investigate the existing performance and confirm the validity of the retrofitting/repairing method by installing RC wing walls.

Fig. 5 – Beam-column joint at X12-Y1 with Class V damage

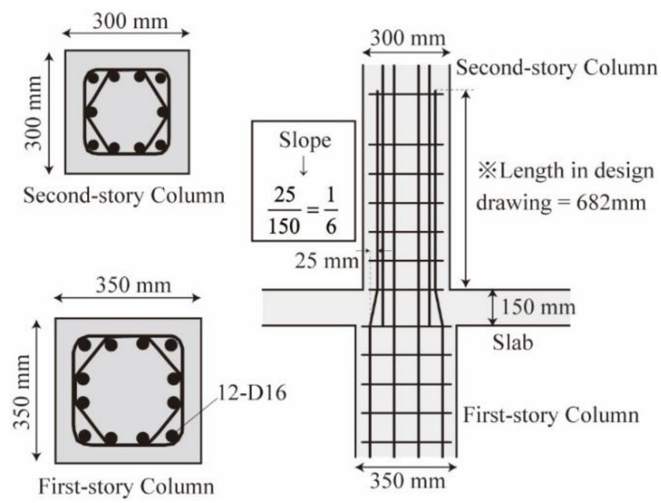


Fig. 6 – Structural details at the joint in Fig. 5

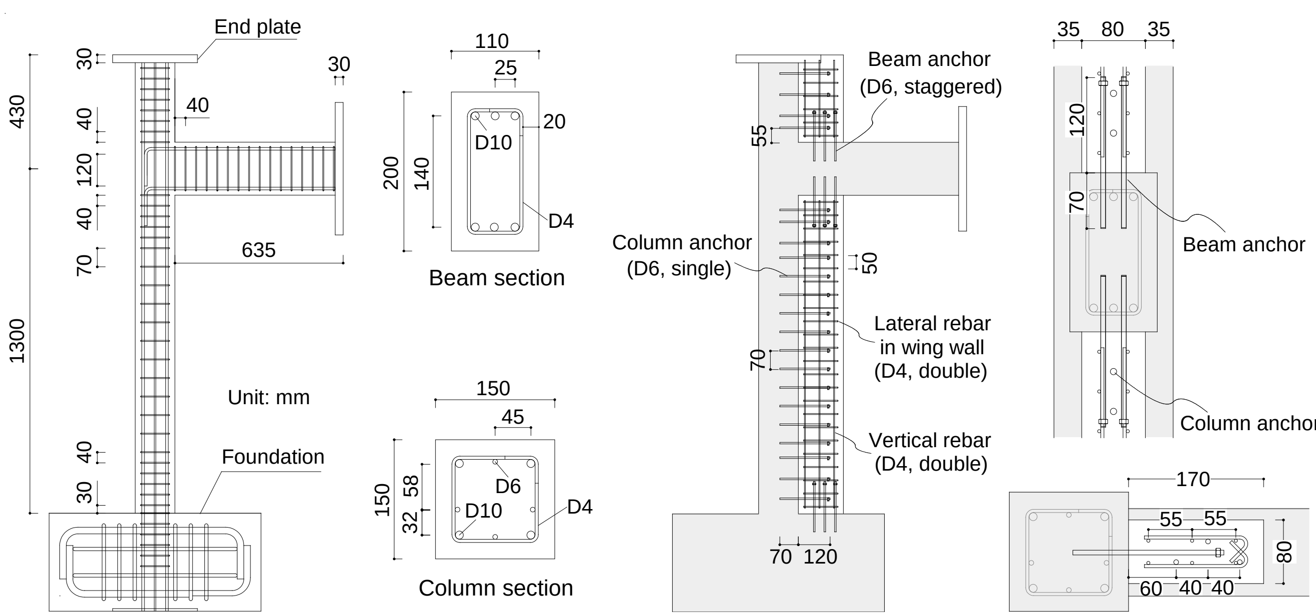
3. Test program

To evaluate the seismic performance of substandard beam-column joints without transverse reinforcement and develop practical retrofitting/repairing methods, experimentally study was conducted.

3.1 Specimens

Three 1/2.5-scale plane frame specimens with the same details were firstly prepared, approximating the original frame at X12-Y1 along the span direction in Fig. 4. They were modeled from the bottom of the first story column up to the inflection point (middle height) of the second story column and included the half span of the second floor beam. In designing the specimen, however, a reduction of the cross-sectional area of the second-story column and horizontal longitudinal reinforcement in the orthogonal beam were not considered. Figure 7(a) shows the dimensions and reinforcement details. Table 2 shows the structural parameters. As shown in Fig. 7(a), no transverse reinforcement was provided in the joint, representing the actual joint in Fig. 6. Each longitudinal beam reinforcement was embedded downward into the exterior column with a 90-degree bend, and 90-degree end hooks were applied to transverse reinforcements for the beam and columns.

One of the three specimens was a benchmark specimen named J3, another was retrofitted by installing wing walls named J3-WI, the other was repaired after moderately damaged by a pre-loading which was named as J3A/J3A-WI before/after repairing.



(a) Original frame (J3, J3A) (b) Retrofitted/repaired frame (J3-WI, J3A-WI)

Fig. 7 – Dimensions and reinforcement details of the specimens

Table 2 – Structural parameters of the original frames

	Section (mm)	Longitudinal Reinforcement	Shear Reinforcement
Column*1	150×150	4-D10, 4-D6 (p_g = 1.83%)	Middle: D4@70 mm (p_w = 0.27%) End: D4@40 mm (p_w = 0.47%)
Beam*1	110×200	Top: 3-D10 (p_t = 1.14%) Bottom: 3-D10 (p_t = 1.14%)	D4@40 mm (p_w = 0.64%)

Note: D4, and D6 mean deformed bars with nominal diameters of 4, and 6 mm, respectively.
*1 Scaled column and beam represent Column 2 and Beam 3 in Table 1, respectively.

The retrofitted specimen J3-WI and the repaired specimen J3A-WI possess the same details and were designed as follows: The wing walls were designed so that the width was 170 mm, equal to the summation of the depth of the existing column (150 mm) and the minimum thickness of the concrete cover (20 mm). According to the Japanese guidelines for retrofitting RC buildings [9], the thickness of the wing wall should not be less than 200 mm for strengthening; hence, the thickness of the wing wall was 80 mm for the 1/2.5-scale specimen. Beam anchors connecting the wing walls and the existing beam 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. 7(b). The column anchors were designed to resist shear on the boundaries between the column and wall. The embedding depths for the anchors into the existing frame and the wing walls were 70 mm (approximately $12d_a$, d_a: diameter of anchor) and 120 mm ($20d_a$), respectively. D4 reinforcing bars were provided for the wing walls with double layers. ϕ6 spirals were installed at the boundaries between the wing walls and the existing frame to prevent the splitting failure of the concrete (omitted from Fig. 7(b)). Tables 4 and 5 give the mechanical properties of the concrete and the reinforcing bars, respectively.

Table 4 – Test results for cylindrical concrete (N/mm^2)

		Compressive strength	Tensile strength	Young's Modulus
Existing frame	J3, J3A, J3-WI	35.9	2.84	2.27×10^4
	J3A-WI	35.2	1.61	3.02×10^4
Wing wall	J3-WI	26.1	2.18	2.27×10^4
	J3A-WI	30.4	1.52	3.10×10^4

Table 5 – Properties of reinforcements (N/mm^2)

Type	Usages	Yield stress	Tensile strength	Young's Modulus
D10	Columns; Beams	373	530	1.70×10^5
D6	Columns; Anchors in J3-WI	387	513	1.86×10^5
	Anchors in J3A-WI	380	496	1.88×10^5
D4	Column hoops	359	525	1.80×10^5
	Beam stirrups; J3-WI walls	365	529	1.76×10^5
	J3A-WI walls	308	477	1.80×10^5

3.2. Test Setup

Figure 8 shows the test setup. The actuators were connected to the top of the specimen. The beam end was supported by a roller. The specimens were subjected to horizontal reversed cyclic loads at the height of the middle of the second story column h under a constant axial load of 23.3 kN which was approximately equivalent to the column axial load ratio of 0.05. Figure 8 also shows the horizontal loading program, which was displacement-controlled with a column drift ratio $R = \delta/h$, where δ is the lateral displacement at the column tip. However, to simulate a moderately seismic damaged structure, J3A which was used for the existing frame of the repaired specimen, loading was stopped after the cycle to R = 1.5% rad., as shown in the figure. Then it was uninstalled from the loading facilities and repaired by installing wing walls.. Figure 9 presents a strain gauge arrangement. In addition, the crack widths on the specimens were measured at every peak and residual drifts during the cyclic loading.

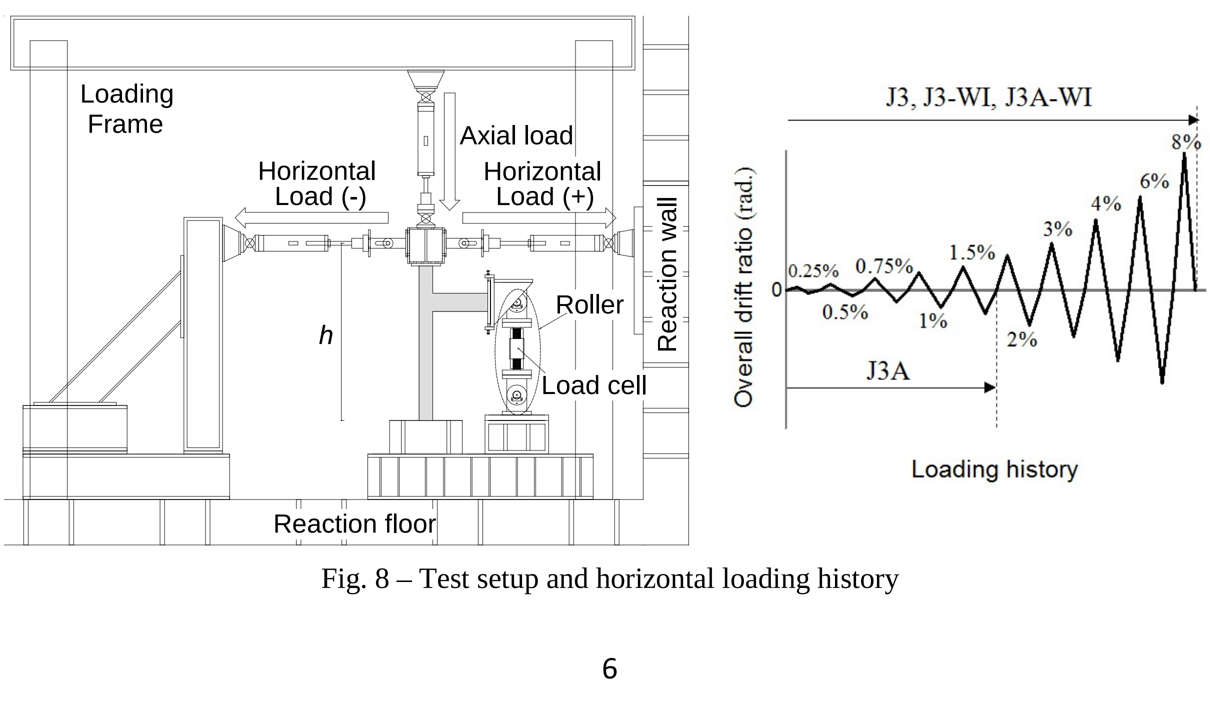


Fig. 8 – Test setup and horizontal loading history

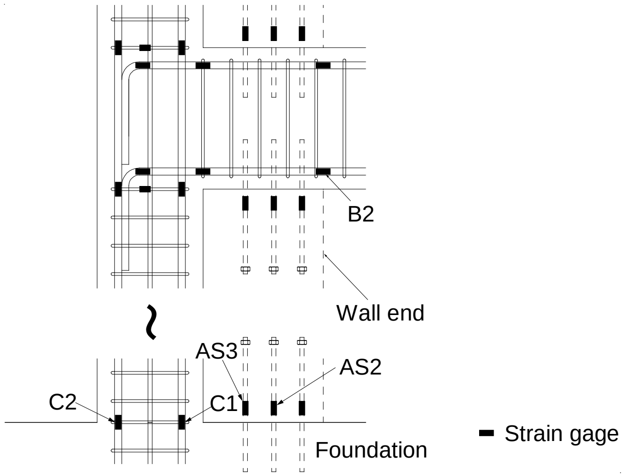


Fig. 9 – Strain gauge arrangement

4. Test Results

Figure 10 summarizes the horizontal load-drift relationships of the specimens, including major events observed during the tests. However, for the benchmark specimen J3, the hysteresis curve was not appropriately recorded during R = 2.5~3.8% rad. in the cycle to R = 3% rad. due to a technical error; therefore, it is shown by a dotted line in the figure. To clearly contrast the seismic performances of the specimens, the skeleton curves of the hysteresis loops in Fig. 10 are shown in Fig. 11. The behavior of every specimen is described in the following.

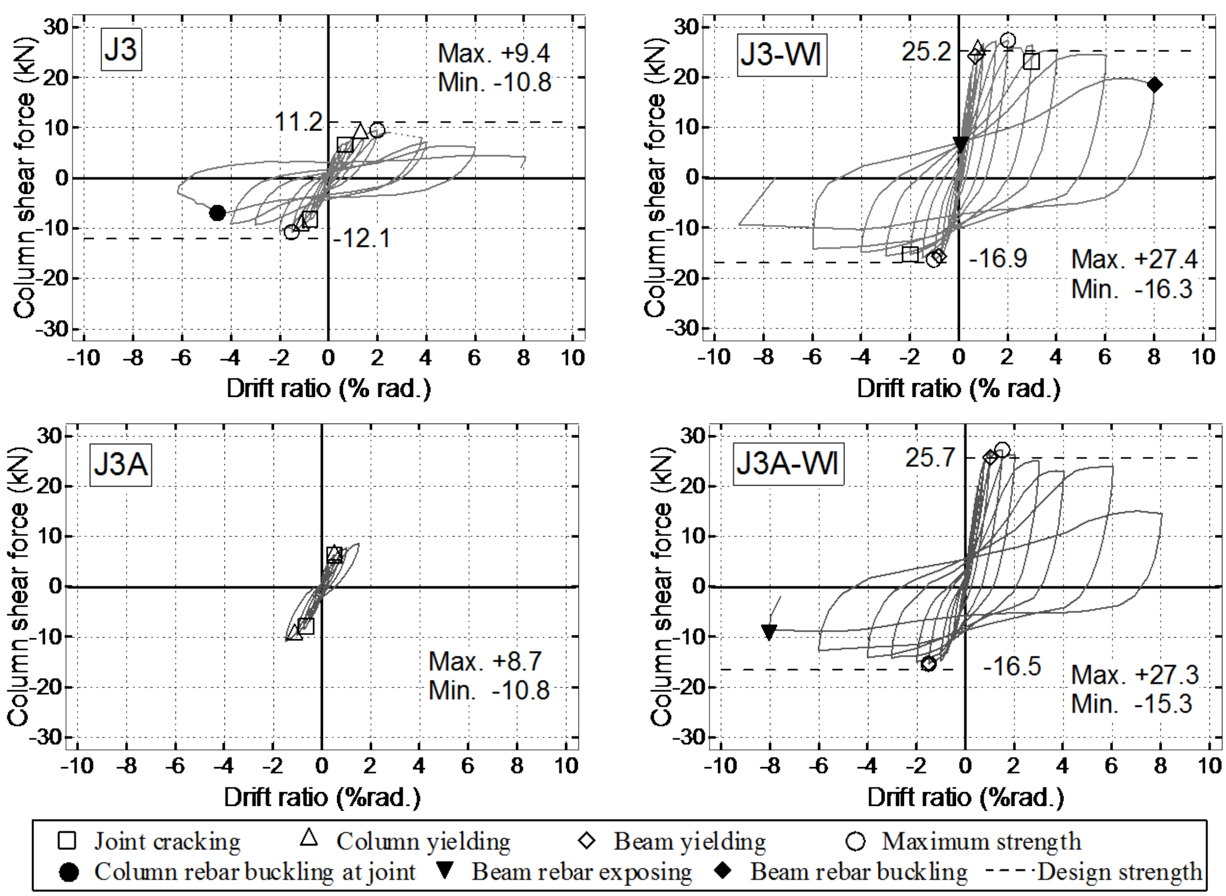


Fig. 10 – Horizontal load-drift relationships

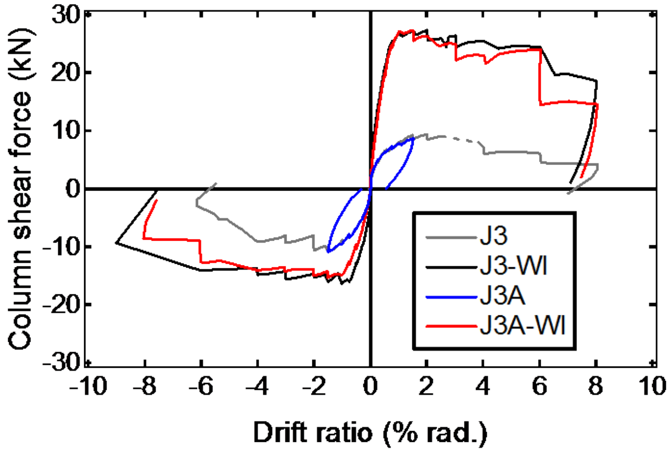
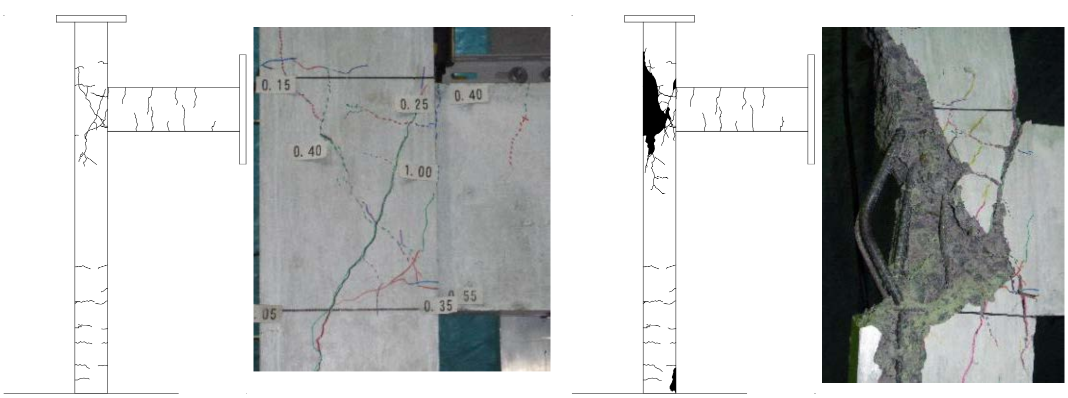


Fig. 11 – Skeleton curves of the hysteresis loops in Fig. 10

- Benchmark specimen J3

An initial flexural crack was observed at the end of the beam in the first cycle to R = 0.25% rad., after which flexural cracks appeared at both ends of the first story column and the bottom of the second story column. Diagonal cracks were observed to the joint panel during the cycle to R = 0.75% rad., which resulted in obvious stiffness reductions. The longitudinal column reinforcements yielded at the bottom of the first story column (C1 and C2 in Fig. 9) during the cycle to R = 1.5% rad., where a negative maximum strength of -10.8 kN was recorded at the peak drift. At a drift of +2% rad., a positive maximum strength of 9.4 kN was recorded (Fig. 12(a)). Subsequently, obvious strength deteriorations were observed in both loading directions. During the cycle to R = 6% rad., a substantial amount of concrete on the exterior surface of the column spalled off with buckling of the column longitudinal bars, as shown in Fig. 12(b). Concrete crushing occurred at the bottom of the first story column in the same loading cycle. Story collapse was observed to the second story in the final loading to R = 8% rad., as shown in Fig. 12(b). Consequently, severe damage was concentrated in the joint panel, which corresponded to the actual damage observed to the earthquake-damaged building, as shown in Fig. 5, while the applied axial load was still supported.



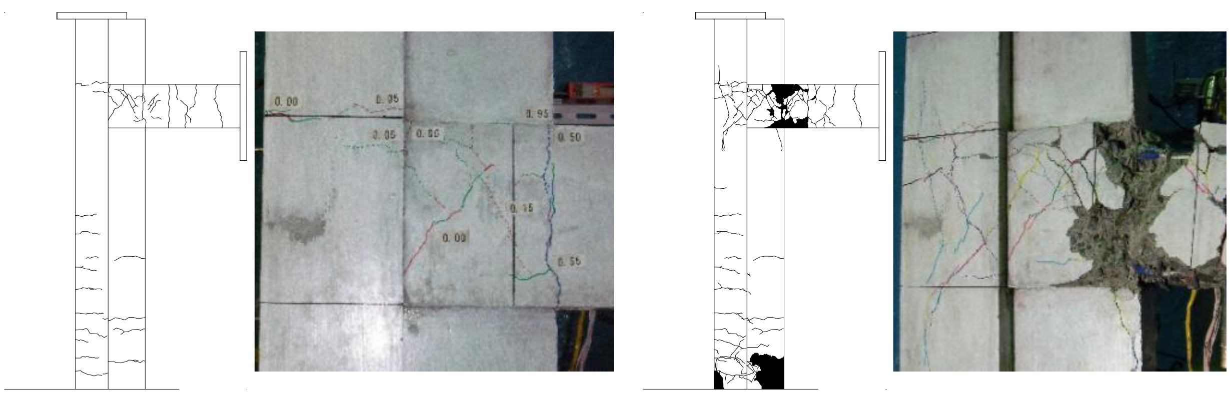
(a) After the cycle to +2% rad. (b) After test

Fig. 12 – Crack patterns and photos focused on the joint for J3

- Retrofitted specimen J3-WI

During the cycle to R = 0.25% rad., initial flexural cracks appeared at the critical section of beam (the end of the wing wall) and the bottom of the first story column with wing wall. Next, the beam and the first story column yielded at the wing wall end (B2 in Fig. 9) and the column bottom (C2 in Fig. 9), respectively, during the positive loading to R = 0.75% rad. The anchors at the bottom of the wing wall (AS2 and AS3 in Fig. 9) began to yield during the negative loading. The negative maximum strength at the peak drift in the cycle to R = 1% rad.

(Fig. 13(a)) was -16.3 kN, which was approximately 1.5 times that of J3. On the other hand, the positive maximum strength at a drift of R = +2% rad. was 27.4 kN, approximately equivalent to 2.9 times that of J3. In the following cycle to R = 3% rad., diagonal cracks appeared at the existing joint, concrete crushing was then observed at the bottom of the first story wing wall. Spalling off of the beam concrete was observed at the wing wall end due to buckling of the beam longitudinal reinforcements in the final loading to R = 8% rad., as shown in Fig. 13(b). In the case of J3-WI, severe damage occurred at the beam-wing wall boundary and the bottom of the specimen. The specimen could resist the applied axial load throughout the test.



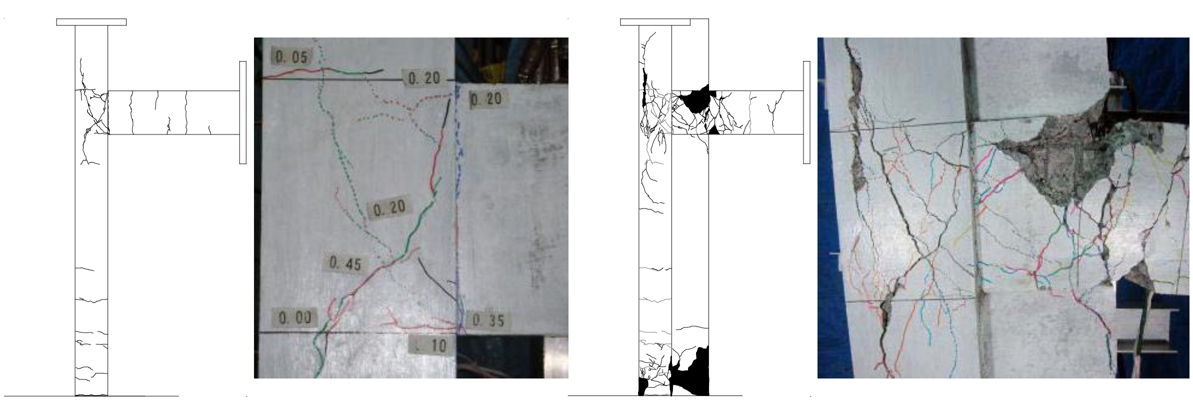
(a) After the cycle to 1% rad. (b) After test

Fig. 13 – Crack patterns and photos of the joint for J3-WI

- Repaired specimen J3A-WI

According to the test results of the benchmark specimen J3, after loading to the cycle of R = -1.5% rad., it had suffered moderate damage: the first-story column had yielded at the bottom and diagonal cracks have appeared in the joint region. Furthermore, the negative maximum strength was recorded. Consequently, the repairing objective, J3A, was pre-loaded to the cycle of R = -1.5% rad. to simulate the moderate damage. Its hysteresis loops and final crack patterns are shown in Fig. 10(c) and Fig. 14(a), respectively.

Subsequently, J3A was repaired by installing wing walls with the same structural details as the retrofitted specimen J3-WI (Fig. 7(b)). As shown in Figs. 10-14, it appeared similar performance to J3-WI. However, the damage to the joint region was more serious, which was considered as the joint has been moderately damaged before repaired. Moreover, strength deterioration of J3A-WI was a little more obvious than the retrofitted specimen.



(a) J3A (b) J3A-WI

Fig. 14 – Crack patterns after tests and photos of the joint for J3A and J3A-WI

5. Discussions

5.1 Change in failure mechanism

Damage to the benchmark specimen was observed at the joint and the bottom of the first story column, as shown in Fig. 12(b). Joint failure occurred before the beam yielding. Meanwhile, in the case of the retrofitted/repaired specimen J3-WI/J3A-WI, the beam (at wing wall end) and the bottom of the first story column with wing wall suffered severe damage, which revealed the beam yielding mechanism. This difference between the mechanisms of the specimens is analytically verified in the following section.

5.2 Improvement in strengths and ultimate strength evaluations

The maximum strengths of J3-WI/J3A-WI were obviously improved after retrofitted/repaired, reaching 2.9/2.9 and 1.5/1.4 times those of the benchmark specimen J3 in the positive and negative directions, respectively. These maximum strengths of the specimens are evaluated.

In the case of J3, the ultimate strength of each member was calculated according to the Japanese seismic evaluation methods [9, 10]: Eqs. (5.1) and (5.2) for the beam and column ultimate flexural strengths ${}_bM_u$ and ${}_cM_u$, respectively, and Eq. (5.3) for the joint shear strength V_{ju}. The column flexural strength was calculated considering a variable axial force at the occurrence of the ultimate failure mechanism. Because the shear strengths of the beam and column are considerably greater than their flexural strengths, they are not discussed here.

$$ {}_bM_u = 0.9 a_t \sigma_y D \tag{5.1} $$

$$ {}_cM_u = 0.8 a_t \sigma_y D + 0.5 ND \left(1 - \frac{N}{bDF_c}\right) \tag{5.2} $$

$$ V_{ju} = \kappa \cdot \phi \cdot F_j \cdot b_j \cdot D_j \tag{5.3} $$

Meanings of the symbols in the equations are not stated here and expected to refer to the reference literature for the limitation of space.

The calculated ultimate strengths are summarized in Table 6, in which conversions to an equivalent moment at the joint center M_j are shown together. The shear strength of the joint was converted to the equivalent joint moment M_j by Eq. (5.4).

$$ V_{ju} = T_b - V_c = \frac{M_j \cdot \dfrac{L_b/2 - d_c/2}{L_b/2}}{j} - \frac{M_j}{L_c} \tag{5.4a} $$

$$ M_j = \frac{V_{ju}}{\dfrac{L_b - d_c}{L_b \cdot j} - \dfrac{1}{L_c}} \tag{5.4b} $$

where T_b is total tensile force of the beam longitudinal bars; V_c is the column shear force; l_b is the length between the joint center and the beam inflection point, j is the distance between the compressive/tensile force couple on the beam critical section ($\approx 0.9d$); and h is the overall height of the specimen, as shown in Fig. 8.

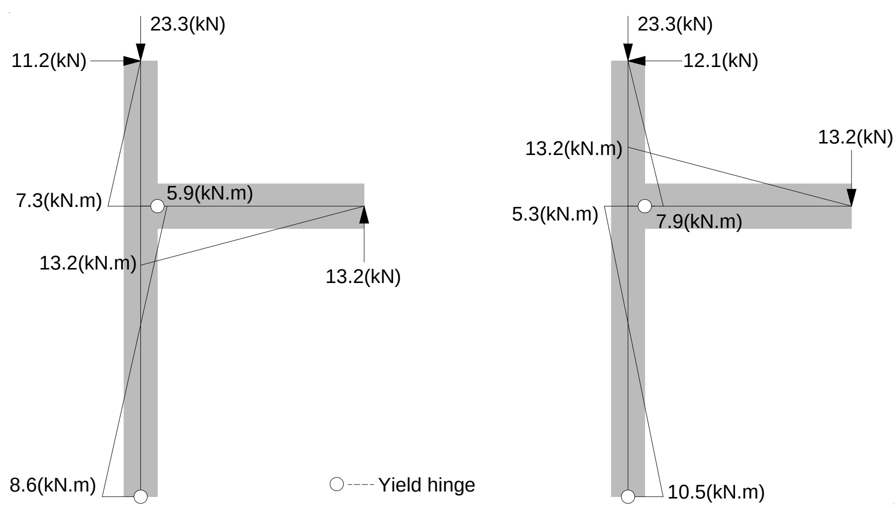
From Table 6, the beam strength (13.2kN.m for the equivalent joint moment) was lower than the column and joint strengths, which indicated that the beam yielded prior to the joint failure. These calculated results were not consistent with the experimental observations, which showed joint failure. Consequently, the Japanese seismic evaluation methods could not estimate the ultimate strength of the substandard joint without hoops in the

Philippines. Therefore, more precise performance evaluation is needed to screen seismically vulnerable buildings with such substandard joints in the Philippines and other developing countries.

Figure 15 presents the moment diagrams of J3 upon the ultimate failure mechanism based on Table 6. The maximum strengths of the specimens were estimated to be 11.2 kN and -12.1 kN under positive and negative loading, respectively, which were approximately 20% higher than the experimental results (9.4/-10.8 kN) due to the inappropriate joint strength estimation mentioned above.

Table 6 – Calculated ultimate strengths of J3

	Beam	Column		Joint	
		Positive loading	Negative loading	Positive loading	Negative loading
Ultimate strength of each member	12.2 kN.m	8.6 kN.m	10.5 kN.m	83.5 kN	83.5 kN
Equivalent joint moment at the ultimate strength	13.2 kN.m	20.3 kN.m	24.7 kN.m	15.4 kN.m	15.6 kN.m



(a) Positive loading (b) Negative loading

Fig. 15 – Moment diagrams upon the ultimate failure mechanism estimated for the benchmark specimen J3

The ultimate strengths of the retrofitted/repaired specimen J3-WI/J3A-WI was also evaluated in a similar manner as J3. However, the flexural strength of the column with wing wall was evaluated as its fully plastic moment based on the stress block concept by ACI318 [11]. The beam was assumed to yield at the location of wall end. The calculated strengths of J3-WI/J3A-WI were 25.2/25.7 kN and -16.9/-16.5 kN at the positive and negative loading, respectively. The calculations agreed well with the test results (27.4/27.3 kN, -16.3/-15.3 kN), verifying the beam yielding mechanism, as mentioned in the previously discussed test observations.

6 Conclusions

This study investigated the seismic performance of an exterior frame with beam-column joint that contained no transverse reinforcement in a typical RC building damaged in the 2013 Bohol, Philippines earthquake. A practical seismic retrofitting/repairing method of installing RC wing walls was proposed and examined by cyclic loading tests on the 1/2.5 scale models. The major findings are summarized as follows:

(1) A benchmark specimen representing the partial frame of the earthquake-damaged building was tested. Severe damage to the joint was reproduced in the test. The maximum strength of the specimen was less than the calculated ultimate strength, which was obtained under the beam yielding mechanism. This difference resulted from the overestimation of the joint strength by the Japanese seismic evaluation methods.

(2) Seismic strengthening by the proposed installation of RC wing walls effectively improved the seismic performance of the existing frame, resulting in a ductile beam yielding mechanism. The ultimate strength of the strengthened frame evaluated by the Japanese seismic evaluation methods agreed well with the experimental results. The proposed strengthening method could serve as a practical upgrade for the seismic performance of RC buildings with substandard joints.

(3) A moderately damaged specimen was repaired by installing wing walls with the same structural details as the retrofitted specimen. Compared to the retrofitted specimen, although damage to the joint region was more severe and strength deterioration was more obvious, similar maximum strengths and a beam yielding mechanism was observed. It's confirmed that the methods by installing RC wing walls are also effective in repairing seismic damaged buildings.

Acknowledgments

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