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EXPERIMENTAL STUDY ON A NEW STRENGTHENING TECHNIQUE OF FLAT PLATE-COLUMN CONNECTIONS USING WING WALLS

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Abstract

Reinforced concrete (RC) flat-plate structural system is a structurally economic and feasible system because of its practicality. However, it has become a vital issue to ensure the safety of buildings in Bangladesh due to the earthquakes that happened recently in surrounding areas (e.g. Nepal earthquake 2015, and Myanmar earthquake 2016). This study focuses on the strengthening of flat plate-column connections in existing structures made with low strength concrete using brick aggregate concrete. Because of unexpected increase in applied loads, lack of considerations of seismic effects, and inappropriate management during design and/or constructions, a significant number of existing flat plate structures are currently required to be strengthened against punching shear failure. Therefore, seismic evaluation and retrofit are necessary. Experimental research on three half-scaled specimens was conducted mainly to investigate the effectiveness of the proposed retrofitting technique using RC wing walls and the punching shear capacity of the connections with low strength concrete. Two specimens FP-W1 and FP-W2 were retrofitted by RC wing walls along the column. Another specimen FP-1 was tested without wing walls and served as a control specimen to make a comparison between the cases with retrofitting. RC wing walls were attached along the loading direction of the column for specimen FP-W1and along the orthogonal loading direction for specimen FP-W2.

The specimens were subjected to cyclic loads at the slab ends, which represented bending moment distribution near the slab and column junction under earthquake loading. Cyclic loads with increasing amplitude were applied to the slabs. The results of the experimental program on the seismic upgrade of slab-column connections are presented in this paper. The nodal moment on the slab and its corresponding drift ratio, crack pattern, and deformations were compared among three specimens. The experimental ultimate strengths were compared to the design calculations according to the AIJ (Architectural Institute of Japan) standard punching shear equations. Punching shear failure occurred for the control specimen at a drift ratio of 2.35%. The strengthened specimen FP-W1 had a significant strength increase by 170% while punching shear failure occurred at a drift of 1.35%. On the other hand, the strengthened specimen FP-W2 had a significant deformation capacity increase by 100% with an increase of strength by 100% as well. The above results experimentally verified that installing wing walls was a feasible method of upgrading flat plate-column connections with low strength concrete under seismic loading conditions.

Keywords: Brick aggregate concrete; Flat plate; Punching shear failure; Static loading tests; Seismic retrofit.

1. Introduction

Nowadays a reinforced concrete (RC) flat plate structure is becoming very popular among many types of RC structures. The flat plate system is expansively adopted by engineers and researchers as it provides a flexible partition of space without the agency of beams and its practicality. It is a special kind of structural system where the plate loads directly transferred to the column without drop panel or column capital. But under seismic loads, transferring bending moments between the plate and adjacent column cause high stresses nearby the column faces and may lead to punching shear failure around the plate-column connections, showing an extreme brittle failure mode.

On the other hand, most of the existing RC buildings in Bangladesh including industrial, commercial, and residential buildings were constructed under inappropriate construction management before or even after the inception of Bangladesh National Building Code (BNBC 1993) [1]. Moreover, Natural stone aggregate is of limited availability and expensive in Bangladesh. Thus, brick aggregate concrete has been used significantly for most of the existing reinforced structures in Bangladesh due to its cheap rate, and abundant availability, which makes the concrete strength extremely low. Due to the increase of applied loads and deficiencies during design or construction, many existing flat structures in Bangladesh are currently required strengthening against punching shear failure for safety reasons [2,3].

Therefore, many substitute method has been developed to increase the punching shear capacity of flat plate-column connections. There are two main ways to increase the punching shear strength. One way is to increase the strength of critical section, for example, by using vertical shear reinforcement in the form of shear studs around the column critical sections [4,5], by using bolts to act as shear reinforcement around the column critical section [6], by using CFRP stirrups [7], by using steel bars grouted into 45-degree inclined drilled holes [8], by using post-installed shear reinforcement as the combined use of nut, washer, and bars [9]. The enhancing of column perimeter is another way to increase the punching shear capacity. The enlargement of the top of a concrete column located directly below the slab or drop panel can be a popular method in terms of enhancing the column perimeter [10]. Column perimeter also can be increased by attaching steel collars made by bolts [11], or employing a combine action of steel plates connected through bolts [12].

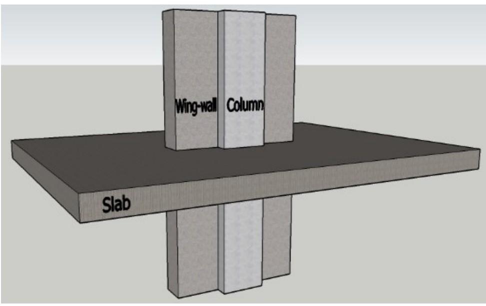


Fig. 1 – Flat plate column connections with wing wall

There are many retrofitting methods for the flat plate system are available from the literature review. But conventional retrofitting techniques are sometimes challenging due to the nature of occupancy, the importance of the structure, cost of the man and materials. For this purpose, this study proposes a new strengthening technique using RC wing walls and describes experimental investigations on the behavior of reinforced concrete flat plate-column connections. This strengthening method is to install small RC wall panels that may not be considered shear walls. Figure 1 shows typical flat plate-column connections strengthened with RC wing wall. The objective of this strengthening method is to increase the seismic performance of existing buildings by changing the existing independent columns to columns with wing walls for upgrading their strength [13]. The use of RC wing walls in Bangladesh will be relatively easy and economic. In the present study, the punching effects of strengthening by the wing walls on the plate under static cyclic loading were investigated. The outcome of the study ensures that the proposed strengthening technique can be a promising strengthening technique to increase the punching shear capacity with the increase of deformation capacity of flat plate-column connections under seismic loads.

2. Significance

An experimental program was conducted for flat plate structures made of low strength concrete to investigate the effectiveness of the proposed strengthening technique using RC wing walls. In the flat plate structure punching shear failure is the very brittle failure mode. Therefore, it is an important issue to enhance the punching shear strength, especially for the structure with low strength concrete in developing countries like Bangladesh. Consequently, the proposed strengthening method can be adopted as a promising method to upgrade an existing RC flat plate structure. The experimental results showed that the strengthened connections by wing walls were significantly improved with increasing of strength and/or deformation capacity compared to that of the existing connection. This method can be an effective solution for strengthening flat plate structure with moderate to high strength concrete flat plate structures as well as that with low strength concrete focused in the present study.

3. Experimental Program

Experimental research with a series of three half-scaled specimens was constructed. To investigate the improvement of punching shear capacity two specimens FP-W1 and FP-W2 retrofitted by RC wing walls were tested and compared to another specimen (Specimen FP-1) without wing walls which was served as a control specimen. RC wing walls were installed along the loading direction of the column for specimen FP-W1, while the direction was orthogonal for specimen FP-W2.

3.1 Specimen Details

The plate dimensions were 1500mm x 1100mm x 75mm in length x width x thickness, respectively. The double layer of D6@75 mm was used for the slab reinforcement in both longitudinal and transverse directions. The column was on the centroid of the slab with a cross-section of 175 mm square. 8-D16 bars were used as main reinforcement and 2-D6@50 mm was used for hoop reinforcement of the column. There was no shear reinforcement on the slab near the slab column connections. The dimensions of wing walls were 175 x 100 mm in length x thickness, which were equivalent to the column depth and the minimum thickness required in the Japanese guidelines of retrofit design, respectively [12]. 4-D6 (U shaped) vertical rebar and 2-D6@175 mm horizontal rebar were used in the wing walls. The specimen details are shown the Fig. 2.

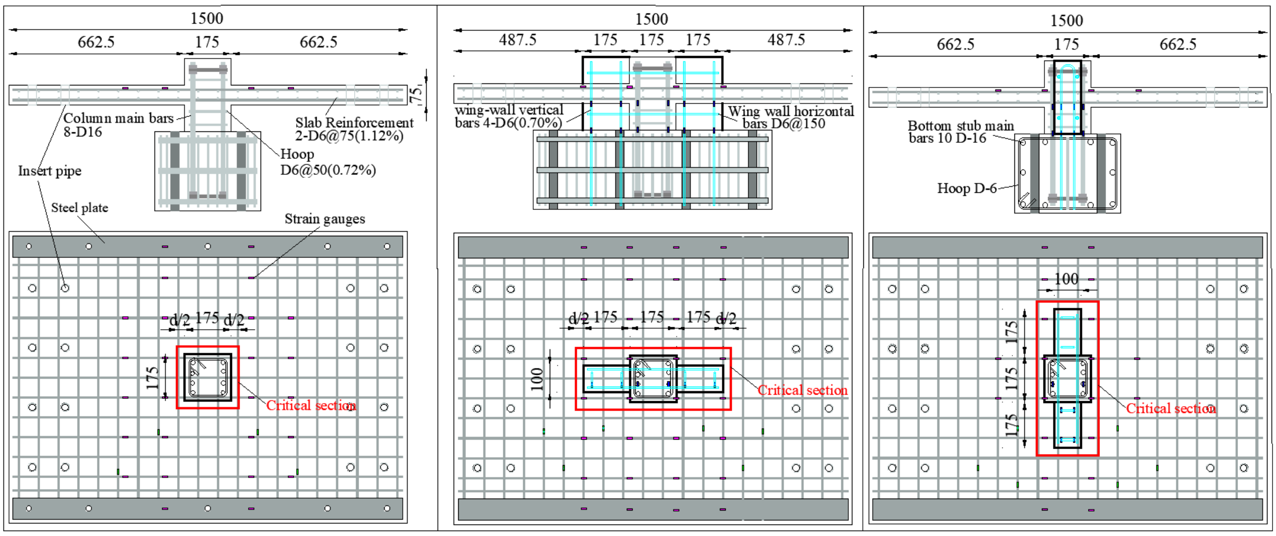


Fig. 2 – Details of the specimens: Units are in mm

Moreover, two steel plates (PL-6 x 80, SS400) were embedded in both sides of the slab, as shown in Fig. 3, mainly to realize the punching shear failure by increasing the slab flexural strength. Studs were installed on the steel plates for the better integration between steel plates and concrete. For this purpose, a total of 10 studs ($\Phi 8$) were installed for getting a better connection of the concrete with steel plates, shown in Fig. 4. As a result, the design ultimate flexural strength of the slab and column were sufficiently larger than the design punching shear strength.

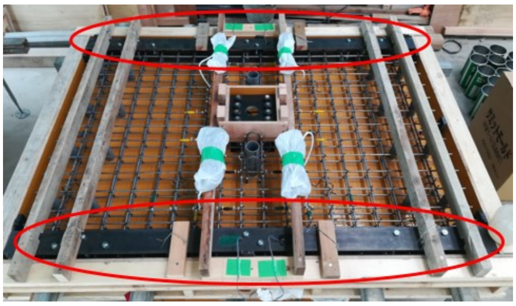


Fig. 3 – Placing of steel plates


Fig. 4 – Installation of studs

3.2 Material Properties

Table 1 – Concrete mixture in kg/m^3

W/(Cement+$CaCO_3$)	Water	Cement	$CaCO_3$	Sand	Brick
60%	212	248	88	596	959

Table 2 – Material properties of concrete

Specimen type	Compressive strength (N/mm^2)	Elastic modulus (N/mm^2)	Strain at compressive strength (μ)
FP-1	6.93	8,450	1,829
FP-W1	7.16	8,672	1,694
FP-W2	7.20	8,741	1,586
Wing walls	37.04	30,929	1,956

Table 3 – Averaged properties of the reinforcement

Specimen no.	Reinforcement type	Cross section of Bar (mm^2)	Yield stress (N/mm^2)	Elastic modulus (N/mm^2)	Tensile strength (N/mm^2)
FP-1	D6 (SD295A)	28.27	346	177000	517
	D16 (SD345)	201.06	432	262000	566
FP-W1 and FP-W2	D6 (SD295A)	28.27	381	191000	505
	D16 (SD345)	201.06	369	175000	536

The recommended concrete mixture was designed for target concrete compressive strength of 8 MPa which was low strength concrete in Bangladesh. For preparing the concrete mixture, Portland cement was used along with mountain sand as fine aggregate. For a better simulation of low strength concrete used in Bangladesh, brick chips were used as coarse aggregate with the maximum aggregate size of 15 mm for the existing components of the specimens. The concrete for the existing structures (slab and column) was mixed with a mixture machine. A cement and CaCO3: sand: brick chips volumetric ratio was designed as 1(0.7+0.3): 2: 4, where 30% of cement was replaced by $CaCO_3$ mainly to decrease the concrete strength. A ratio of water to the sum of cement and $CaCO_3$ was 60%. Table 1 shows a concrete mixture by weight for the existing structures with low strength concrete. A normal ready-mixed concrete of design strength 30 MPa with stone chips as coarse aggregate was used for the wing walls and bottom stubs. The average properties of the concrete from the compression tests are summarized in Table 2. And the averaged material properties of reinforcement are shown in Table 3.

3.3 Loading and Measurement Methods

A schematic view of the experimental setup of the flat plate specimen is shown in Fig. 5. The specimens were tested using a static loading system at Daido University Concrete Structure Lab. The slab and column were supported by a bottom stub on the bottom beam of the loading frame. The loading system consisted of two vertical jacks with 500 kN each, which were supported by the top beam. The jacks were connected to the channel at the end of the slab; thus, the loads were uniformly distributed to the slab ends through the steel channels (125 x 65 x 12 x 21). The load cell (200 kN capacity) in vertical jacks was used to measure the absolute applied shear force on the specimens. In this experiment, no axial load was applied to the column.

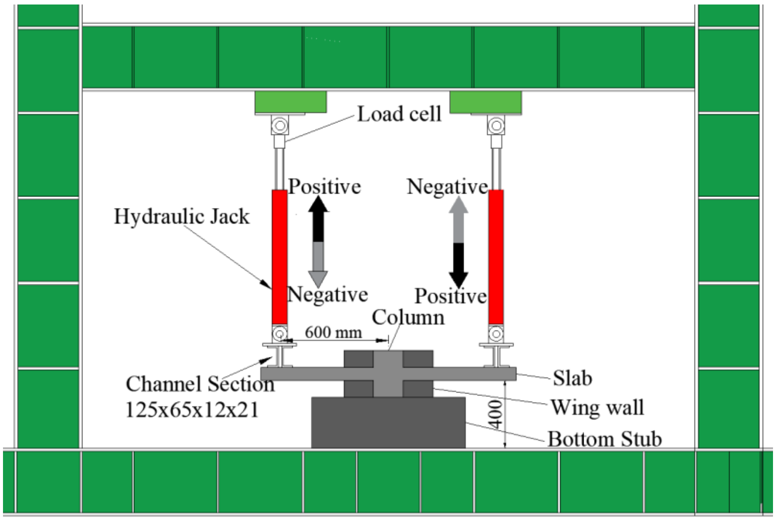


Fig. 5 – Schematic view of an experimental setup

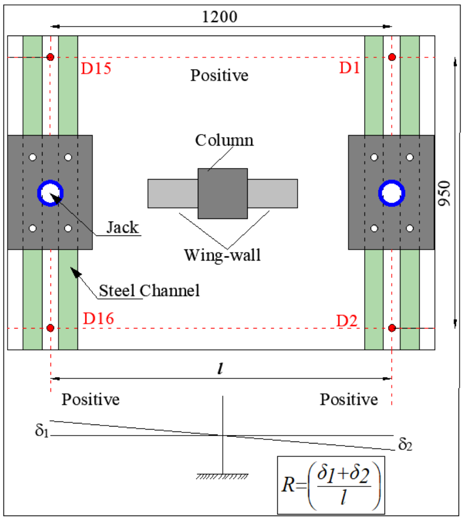


Fig. 6 – Position of LVDT and definition of R

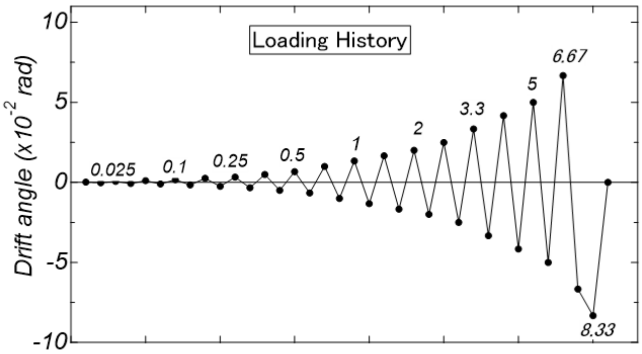


Fig. 7 – Loading history

The hydraulic jacks were manually controlled so that the displacements δ_1 and δ_2 shown in Fig. 6 were equal during the experiment. The cyclic load with increasing amplitude was applied to the slabs. Four CDP-100 LVDT's were installed at four corners of the slab shown in Fig. 6. The loading was controlled by the drift angle R which was defined by the following Eq. (1):

$$R=\left(\frac{\delta_1+\delta_2}{l}\right)=\left\{\frac{(D_1+D_2)/2+(D_{15}+D_{16})/2}{l}\right\} \quad (1)$$

where the symbols can be referred to Fig. 6 providing transducers setup. The loading cycle started from an initial cycle of R of 0.01675x10^{-2} rad, followed by one cycle of each in both positive and negative loading up to the final loading, as shown in Fig. 7.

4. Ultimate Strength Evaluation

The capacity of the flexural strength of the plate used by the following equation [13,14]. It was assumed that all steel components in tension side yielded and that the distance between compression and tension forces was 0.9 times of the effective depth of the plate.

$$My=0.9\sum a_{t1}\sigma_{y1}d_1+0.9a_{t2}\sigma_{y2}d_2 \quad (2)$$

where $\sum a_{t1}$ and $\sum a_{t2}$ are the cross-sectional area of the tensile reinforcing bars and steel plates, d_1 is the effective depth of reinforcing bars of 57 mm, and d_2 is that of steel plate of 72mm. Besides, σ_{y1} and σ_{y2} are the yield stress of the tensile reinforcing bars and steel plates.

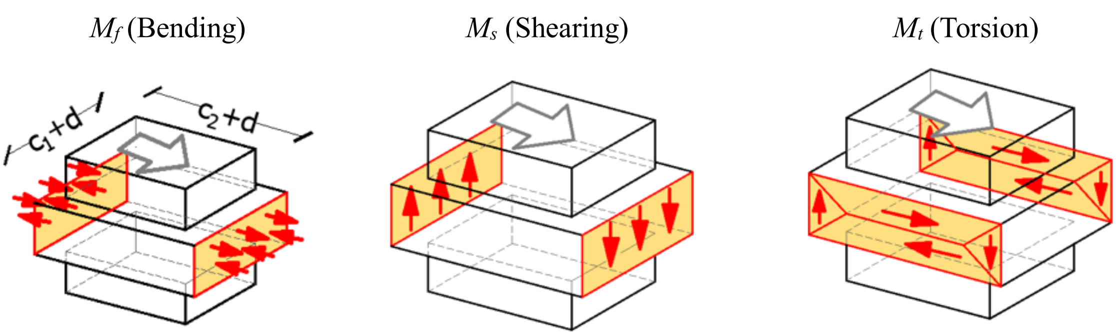


Fig. 8 – Punching shear mechanism due to bending, shearing, and twisting

The design punching shear capacity of the specimens was calculated based on the AIJ (Architectural Institute of Japan) standard [14]. To ensure the testing successfully, the specimens were designed to fail in punching shear failure rather than other structural failures. Therefore, the flexural strength of the plate had to be sufficiently greater than the punching shear strength. The punching shear strength was calculated according to the following Eqs. (3,4,5,6) by the AIJ standard for RC structure. According to this standard, as shown in Fig. 8, the moment resistances due to bending, shearing, and twisting of concrete in the critical cross-section around the column were simply summed up by the following Eq. (3).

$$M_o=M_f+M_s+M_t \quad (3)$$

$$M_f=0.9a_{0t}\sigma_y d\frac{c_2+d}{x_t}+0.9a_{0b}\sigma_y d\frac{c_2+d}{x_b} \quad (4)$$

$$M_s=\tau_u(c_2+d)d(c_1+d) \quad (5)$$

$$M_t=\tau_{tu}\frac{d^2}{2}\left\{(c_1+d)-\frac{d}{3}\right\}2 \quad (6)$$

where, a_{ot}, a_{ob} is the cross-sectional area of slab top reinforcement and bottom bar, respectively; x_t is top reinforcement spacing of slab; x_b is bottom reinforcement spacing of slab; σ_y yield stress of the slab reinforcement; d is the effective depth of the slab; c_1 is the column depth in the loading direction; c_2 is the column width in the direction orthogonal to the loading; τ_u is the direct shear strength of concrete, where, $\tau_u=0.335\sqrt{\sigma_B}$; σ_B is the compressive strength of concrete in MPa; τ_{tu} is the torsional shear strength of concrete, where $\tau_{tu}= 6\ \tau_u$ [13].

The flexural strength of the column was calculated by the following Eq. (7). And the ultimate flexural strength of the column with the wing wall was calculated by the Eq. (8) [13]. In this calculation, the wing wall reinforcement and intermediate reinforcement in the column with the multilayered arrangement of flexural reinforcement shall be considered in principle.

$$_cM_u=0.8\sum a_t\sigma_y d \tag{7}$$

where a_t is the cross-sectional area of tensile reinforcing bars; σ_y yield stress of the column tensile reinforcing bars; and d is the effective depth of the column.

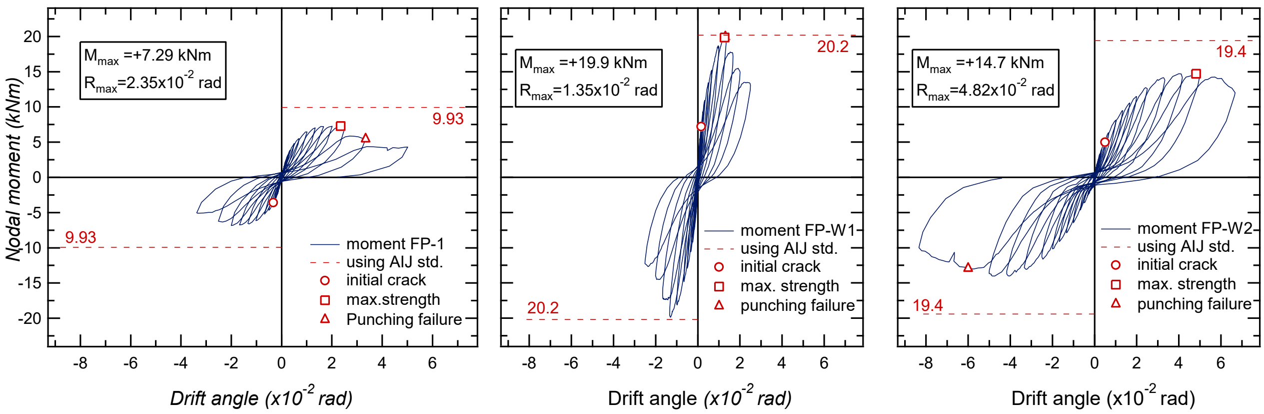
$$_{cw}M_u=a_{te}\sigma_y(b-0.5x_n) \tag{8}$$

where $a_{te}=a_t+a_t'(\sigma_y'/\sigma_y)$, b is the distance between center gravity of the tensile reinforcement and the extreme fiber of compressive zone, a_t and a_t' is the total tensile reinforcement areas of the column and wing wall, respectively; σ_y and σ_y' are the yield stress of the tensile bars of the column and wing wall, respectively; and $x_n=(a_{te}\sigma_y/0.85F_ct)$, F_c is the compressive strength of the concrete in MPa; t is the wall thickness.

According to the above equations the expected failure mode was estimated to be punching failure for all specimens. The details of calculated results and experimental strengths are tabulated in Table 4.

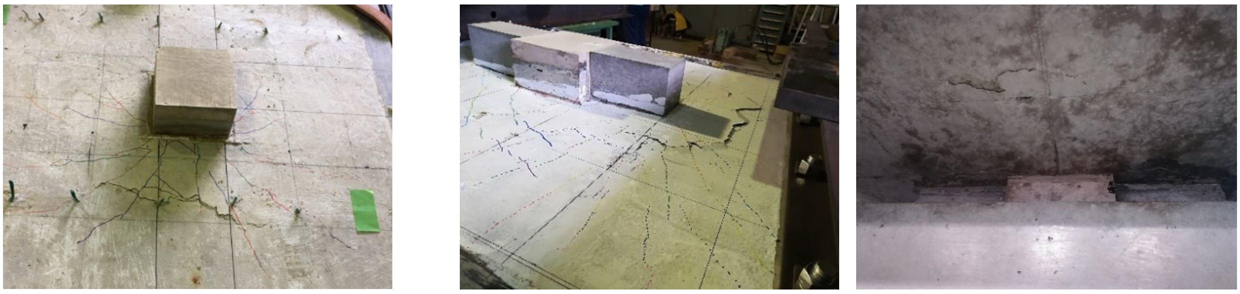
5. Experimental Results and Discussion

The nodal moment versus drift angle relationships of the specimens are explained in this section. The nodal moments applied to the specimens were calculated by multiplying the absolute vertical forces applied to the slab, by the distance from the application point to the center of column of 600 mm shown in Fig. 5. The plate shear force was the average value from both hydraulic jacks. Similarly; the drift was calculated as the ratio of the vertical displacement to the distance from the application point to the center of the column. The nodal moment vs drift ratio relationships are shown in Fig. 9.



(a) Specimen FP-1 (b) Specimen FP-W1 (c) Specimen FP-W2

Fig. 9 – Nodal moment vs drift ratio relationships

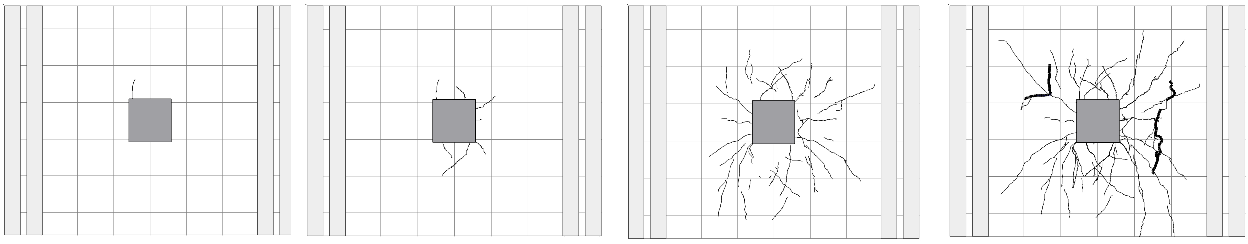


(a) Specimen FP-1 (b) Specimen FP-W1 (c) Specimen FP-W2

Fig. 10 – Damage after punching shear failure

5.1 Failure Process of Specimen FP-1

Specimen FP-1 was tested without retrofitting and served as a control specimen. An initial flexural crack occurred at the corner of the column at *R* of -0.33x10^{-2} rad. Subsequently, it was also observed that the flexural cracks were extended 45^{0} directions from the corner of the column at *R* of +0.5x10^{-2} rad. The maximum strength of 7.29 kN-m was observed in the cycle to +2.35x10^{-2} rad. Thereafter, the strength decreased with raising up of the surface concrete due to punching shear failure. Punching shear failure was fully visible at *R* of +3.33x10^{-2} rad. after attaining the maximum strength. The detailed failure process with crack patterns at different cycles are shown in Fig. 11. The punching shear failure was observed on the top surface of the slab, as shown in Fig. 10(a).

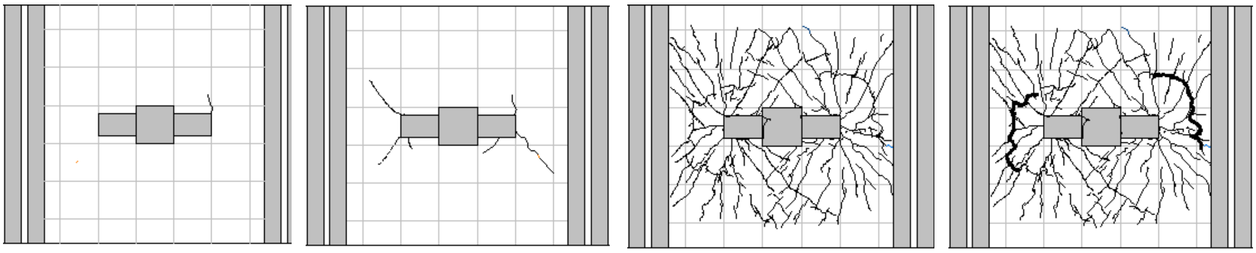


a) Initial cracks started at R -0.33% b) 45^{0} cracks extended at R 0.5% c) Maximum capacity at R 2.5% d) Punching shear failure at R 3.34%

Fig. 11 – Failure process of Specimen FP-1

5.2 Failure Process of Specimen FP-W1

Specimen FP-W1, which was strengthened by post-installed wing walls along the loading direction of the column, showed higher strength but did not experience higher deformation capacity. An initial flexural crack occurred at the corner of the wing wall at *R* of +0.167x10^{-2} rad. Afterward, the cracks were extended 45^{0} directions from the corners of the wing wall at *R* of +0.5x10^{-2} rad. The maximum strength of 19.9 kN-m, during both of the positive and negative loading, was observed in the cycle to 1.35x10^{-2} rad. Subsequently, the strength decreased with raising up of the surface concrete due to punching shear failure. Punching shear failure was fully visible in both positive and negative loading at *R* of 1.33x10^{-2} rad when the maximum strength was observed. The detailed failure process with crack patterns at different cycles are shown in Fig. 12. The punching shear failure was observed on the top and bottom surfaces of the slab. Figure 10(b) shows the damage of the punching failure on the top surface.

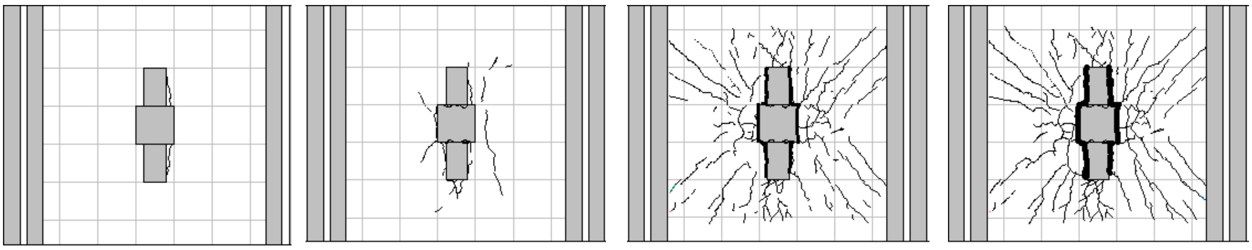


a) Initial cracks started at R +0.167%
b) 45^0 cracks extended at R 0.5%
c) Maximum capacity at R 1.33%
d) Punching shear failure at R 3.33%

Fig. 12 – Failure process of Specimen FP-W1

5.3 Failure Process of Specimen FP-W2

Specimen FP-W2, which was strengthened by post-installed wing walls along the orthogonal loading direction of the column, showed higher strength and deformation capacity than those of the control specimen. An initial flexural crack occurred at the corner of the wing wall at R of +0.5x10^{-2} rad. Then, the cracks were extended 45^0 directions from the corners of the column at R of -0.67x10^{-2} rad. The maximum strength of 14.7 kN-m, during the positive loading, was observed in the cycle to 1.35x10^{-2} rad. Subsequently, the strength decreased with raising up of the surface concrete due to punching shear failure. However, punching shear failure was fully visible in the negative loading after the maximum strength at R of -6.0x10^{-2} rad. The detailed failure process with crack patterns at different cycles are shown in Fig. 13. The punching shear failure was observed on the bottom surface of the slab, as shown in Fig. 10(c).

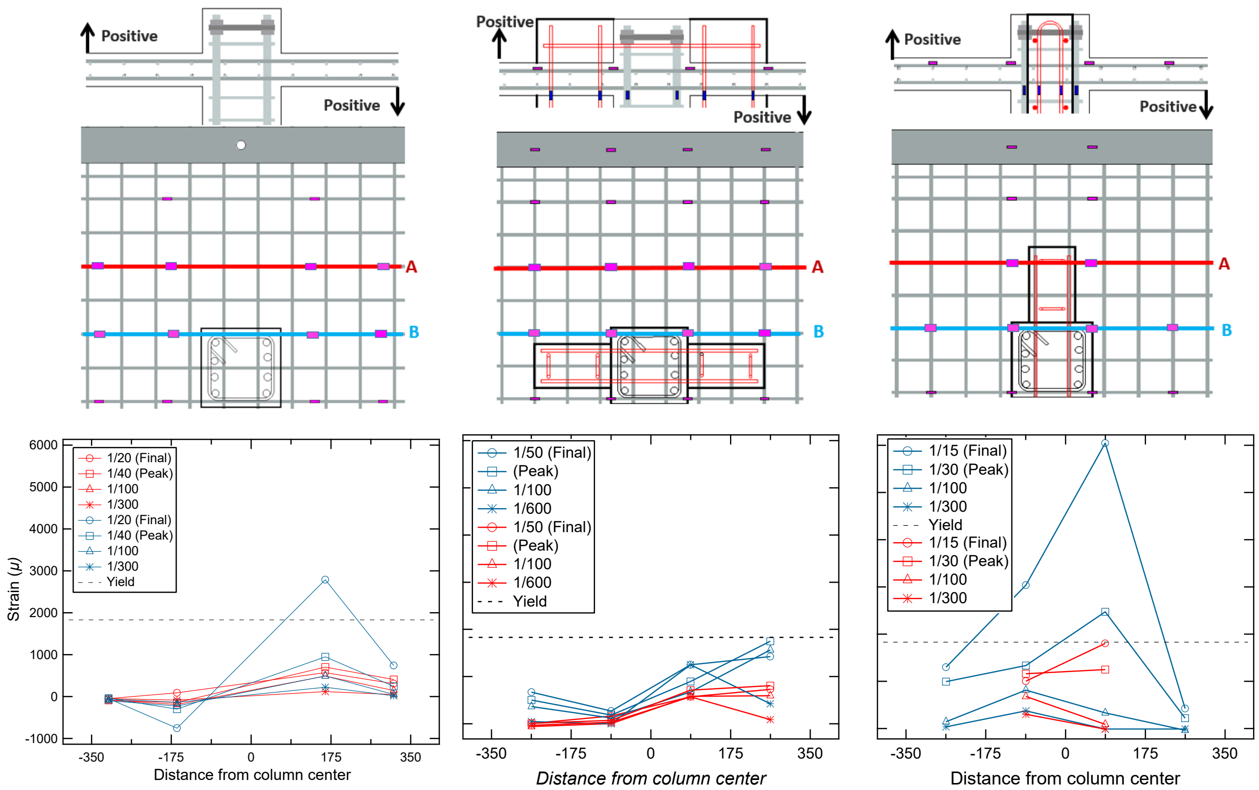


a) Initial cracks started at R +0.5%
b) 45^0 cracks extended at R -0.67%
c) Maximum capacity at R 5.0%
d) Punching shear failure at R 6.0%

Fig. 13 – Failure process of Specimen FP-W2

5.4 Strain Distribution of Slab Reinforcement

The strains were obtained from the strain gauges installed on the top reinforcing bars. The reinforcements were highlighted mainly in two categories, where A (red-lines) was defined as the exterior side in which the reinforcements located 150 mm distance from the edge of the column, as shown in Fig. 14. On the other hand, B (blue-lines) was defined as the interior side which located near the edge of the column, as shown in Fig. 14. Comparing the strains of reinforcing bars, A and B, those on the interior side became larger with increasing of the drift angle for all specimens. These strain responses indicated that the plate deformed not only in bending but also in torsion around the column. For specimen FP-1, it was observed that no reinforcement yielded up to the peak load, while one reinforcement yielded during the final loading, as shown in Fig. 14 (a). For specimen FP-W1, it was observed that no reinforcements yielded up to the final loading, as shown in Fig. 14 (b). For specimen FP-W2, it was observed that one reinforcement yielded at the maximum strength near the column surface, as shown in Fig. 14 (c). The transitions of the strain distribution are graphically illustrated in Fig. 14.



(a) Specimen FP-1 (b) Specimen FP-W1 (c) Specimen FP-W2

Fig. 14 – Comparison of transitions of strain distributions at different cycles

5.5 Experimental Discussion

The experimental strengths are compared to the design calculations, as shown in Table 4. According to the AIJ standard, the maximum capacity of FP-1 was 27% lower than the calculated result of the punching shear strength. The maximum capacity of FP-W1 was 2% lower than the calculated result of the punching shear strength whereas the maximum capacity of FP-W2 was 25% lower than the calculated result of the punching shear strength. In particular, the calculated results for punching strength provided by (assumed critical section shown in Fig.2) the AIJ standards overestimated the maximum capacities of the specimens, which might be caused by the application of low strength concrete.

Table 4 – Calculated and Experimental ultimate strengths

Specimen type	Flexural strength of the slab M_y (kN-m)	Punching shear strength of the slab M_0 (kN-m)	Flexural strength of the column $_cM_u$ (kN-m)	Flexural strength of column with wing wall $_{cw}M_u$ (kN-m)	Expected failure mode	Experimental strengths M_{exp} (kN-m)	Experimental failure mode
FP-1	32.16	9.93	39.81	-	Punching	7.3	Punching
FP-W1	48.94	20.2	39.81	110.9	Punching	19.9	Punching
FP-W2	32.16	19.4	39.81	-	Punching	14.7	Punching

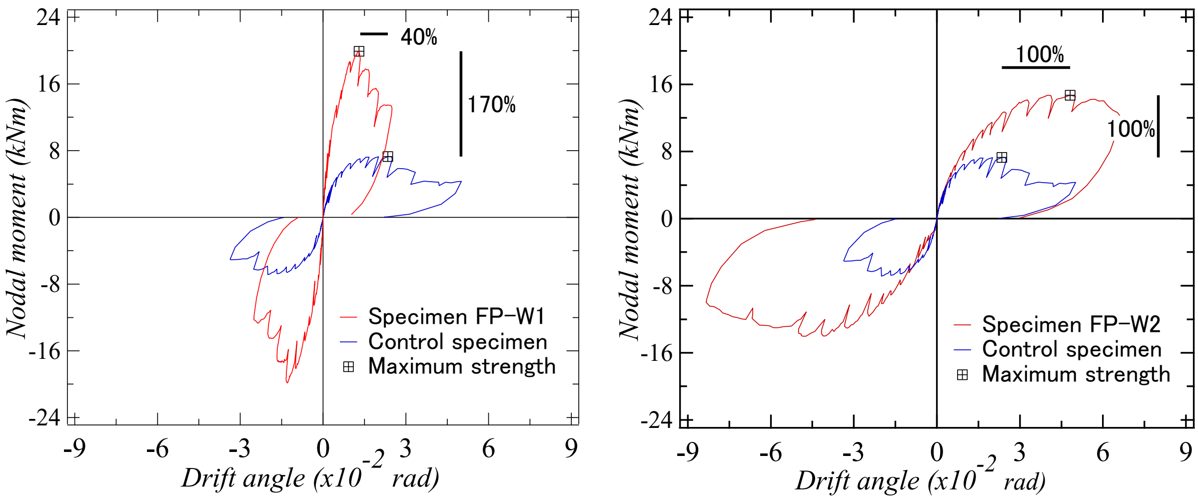


Fig. 15 – Comparison of the performance of strengthened specimens with the control specimen

The comparison of strengthened specimens with the control specimen is shown in Fig. 15 using the backbone curves. Punching shear failure occurred for the control specimen FP-1 at a drift ratio of 2.35%. The strengthened specimen FP-W1 had a significant strength increase by 2.7 times than that of control specimen while punching shear failure occurred at a drift of 1.35%. On the other hand, the strengthened specimen FP-W2 had a significant deformation capacity increase by twice with an increase of strength by twice as well. Therefore, it can be concluded that both strengthened specimens showed the effectiveness of the wing wall application to upgrade the punching shear resistance of the flat plane-column connection.

6. Conclusions

A series of the experiment of three half-scaled interior flat plate-column connections with two strengthened specimens by RC wing walls were conducted mainly to investigate the effectiveness of this technique as well as to evaluate the seismic behavior and performance. Based on the results obtained from the tests, the following conclusions can be drawn:

1) It was observed that flexural cracks were extended in 45° directions from the column/wing walls surface where punching cracks extended radially from the column/wing walls surface and the surface concrete peeled up due to punching shear failure.
2) RC wing wall was effective to strengthen the flat-plate structure. For the in-plane direction strengthening (FP-W1), the maximum strength achieved 2.7 times larger than that of the control specimen (for FP-1). For the out-of-plane direction strengthening (for FP-W2), the maximum strength was 2 times larger than that of the control specimen. Moreover, the ductility of specimen FP-W2 was larger than that of the specimen FP-W1.
3) Punching shear failure occurred in FP-1 after attaining the maximum strength. On the other hand, punching shear failure occurred in FP-W1 at the maximum strength, while that occurred after the maximum strength of FP-W2.
4) The punching shear strength estimated based on the AIJ standards overestimated the maximum capacity of the flat plate specimens using low strength concrete.
5) It was experimentally verified that installing RC wing-wall as a new concept of retrofitting technique of flat plate-column connections made with low strength concrete is a structurally feasible method to upgrade the connections performance under seismic loading.

7. Acknowledgments

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