Original Paper

Strengthening seismically vulnerable reinforced concrete flat plate-column connections by installing wing walls

H. M. Golam Samdani,1 Susumu Takahashi2 Rokhyun Yoon1 and Yasushi Sanada1

1Graduate School of Engineering, Osaka University, Osaka, Japan 2Faculty of Engineering, Daido University, Nagoya, Japan

Correspondence
Susumu Takahashi, Faculty of Engineering, Daido University, Nagoya, Japan.
Email: susumu-t@daido-it.ac.jp

Funding information
JST/JICA SATREPS-TSUIB project “Technical Development to Upgrade Structural Integrity of Buildings in Densely Populated Urban Areas and its Strategic Implementation towards Resilient Cities in Bangladesh (TSUIB)”.

Received March 15, 2021; Revised May 9, 2021; Accepted May 18, 2021
doi: 10.1002/2475-8876.12232

Abstract
This study emphasizes a new strengthening scheme for seismically vulnerable flat plate-column connections in existing structures. An experimental investigation on a series of three half-scaled interior plate-column connections was conducted to explore the effectiveness of the proposed seismic strengthening method using postinstalled RC wing walls and to analyze the punching shear capacity of the connections. This study focuses on typical existing flat plate structures constructed using low-strength concrete in Bangladesh. Two of the three specimens had wing walls employed in two principal directions, that is, along the loading direction and orthogonal direction, while another specimen was tested without wing walls and served as a control specimen for comparisons between the cases with and without strengthening. Static cyclic vertical displacements with increasing amplitude were applied to the ends of the slabs to represent interstory drift under seismic action. Both strengthened specimens showed higher resistance to punching shear failure. Thus, the test results experimentally verified that installing RC wing walls is a feasible method to upgrade flat plate-column connections, including those constructed using low-strength concrete, to prevent collapse under seismic action.

Keywords
brick aggregate concrete, developing country, flat plate structure, punching shear failure, seismic retrofit

1. Introduction

Reinforced concrete (RC) flat plate structural system is well known because of its various advantages. Thus, the application of such a system has become widespread among engineers and designers owing to its practicality. However, there is a lack of resistance to lateral loads in this structural system, resulting in the collapse of existing buildings during earthquakes. Therefore, buildings in developed countries, such as the United States, New Zealand, and Japan, should be well designed and managed with high/moderate seismicity. However, many flat plate structures exist in developing countries, even in high/moderate seismic areas, where it has become a vital issue to ensure the safety and integrity of flat plate structures because of their insufficient seismic resistance. However, flat plate structures are vulnerable owing to the punching phenomenon they experience during an earthquake. Punching shear failure is a failure pattern caused by extreme stresses on the cross-section of the plate close to the plate-column joint. These stresses are created while transferring lateral forces, vertical forces, and bending moments between the plate and column. This brittle failure sometimes leads to overall structural failure, as shown in Figure 1A, which demonstrates an example of progressive collapse due to punching shear failure.1

Geographically, Bangladesh is one of the most earthquake-prone zones because it is located at the juncture of several active tectonic plate boundaries, and many geologically active fault lines run across the country. However, a limited number of devastating earthquakes have occurred in Bangladesh over the past 100 years. The history of Bangladesh shows that 1762, 1885, and 1897 earthquakes in Bangladesh caused widespread damage throughout the country.2 The recurrence of similar earthquakes from any of these active seismic sources may cause significant damage to large population centers in Bangladesh.3,4 In fact,
frequent earthquakes have recently occurred in nearby regions (e.g., the Nepal earthquake in 2015 and the Myanmar earthquake in 2016). Additionally, the availability of natural stone aggregates is limited in Bangladesh; thus, brick aggregate concrete, which available abundantly, was used to construct the majority of the existing RC buildings in Bangladesh, resulting in a lower construction cost. Such usage of brick aggregate concrete has resulted in a large number of buildings being constructed with low-strength concrete. Moreover, the majority of existing RC buildings, especially those in the older parts of Dhaka city, were constructed under inappropriate construction management before and after the inception of the Bangladesh National Building Code (BNBC). Figure 1B provides an example of a flat plate structure failure in Bangladesh, which reflects collapse due to inappropriate design and construction management. Therefore, many existing RC buildings that commonly include a flat plate system in Bangladesh are at risk of damage/collapse during future earthquakes. Hence, seismic strengthening of existing vulnerable RC buildings is currently required to mitigate earthquake disasters.

To reduce the potential risks of earthquake damage to existing buildings with insufficient seismic performance, many strengthening methods have already been developed in earthquake-prone countries. Based on previous schemes to increase the punching shear capacity of flat plate-column connections, there are two alternative ways to increase the punching shear strength. One possible approach is to increase the shear strength of the plate. This can be implemented by using vertical shear reinforcement in the form of shear studs around the critical sections of the plate, bolts to act as shear reinforcement, fiber-reinforced polymer (FRP) stirrups, carbon fiber-reinforced polymer (CFRP) stirrups, steel bars grouted in holes drilled at a 45° inclination, and postinstalled combined reinforcement with nuts, washers, and bars. Enhancing the perimeter of the column is another approach used to increase the punching shear capacity. The enlargement of the top of a concrete column located directly below the slab, that is, a drop panel, can be a popular method for enhancing the column perimeter. The column perimeter can also be increased by attaching steel collars containing bolts or by employing the combined action of steel plates connected to steel bolts.

Many alternative strengthening methods for flat plate systems have been presented in the literature. Nevertheless, modern retrofitting techniques are often difficult to implement in developing countries because of the complexity of work, material supply, and low cost of human resources. Additionally, minimal to no guidance has been provided for strengthening flat plate structures constructed using low-strength concrete, which needs to perform not only to ensure the performance of flat plate-column connections but also to increase the lateral resistance of the entire structure. To satisfy the aforementioned requirements, this study proposes a practical strengthening scheme of installing RC wing walls beside an existing column. Furthermore, the behavior of RC flat plate-column connections constructed using low-strength concrete applied to the proposed strengthening method was investigated. Previously, this wing wall installation scheme was applied to substandard beam-column joints, and an improvement in the performance of the connections under seismic loading conditions was verified. As mentioned in the literature, the application of RC wing walls in developing countries is relatively easy and cost-effective. Figure 2 shows a typical flat plate-column connection strengthened using RC wing walls. The enlargement of the critical section of a concrete column with wing walls is likely to be accepted as an alternative method in terms of enhancing the column perimeter. Furthermore, this method increases the lateral resistance of the column. In this study, the effects of wing wall strengthening on the punching behavior of plates under static cyclic loading were investigated. The results of this study confirm that the proposed approach for RC wing wall installation is a promising strengthening method to increase the punching shear capacity of flat plate-column connections. Furthermore, this approach contributes to the practical upgrades of seismic performance of existing flat plate structures, including those constructed using low-strength concrete.

2. Significance

Many existing RC flat plate structures require seismic strengthening. Therefore, an experimental program was conducted for
flat plate structures constructed using low-strength concrete to investigate the effectiveness of the proposed strengthening method using RC wing walls. Punching shear failure is a very brittle failure mode for flat plate structures. Hence, it is necessary to enhance the punching shear capacity to prevent brittle failure, especially for flat plate structures constructed using low-strength concrete in developing countries, such as Bangladesh. Consequently, the proposed wing wall installation approach can be adopted as a promising method to upgrade existing RC flat plate structures, including those constructed using low-strength concrete. The experimental results showed that the strength of flat plate-column connections strengthened using wing walls improved significantly with an increase in punching shear strength. However, the existing components exhibited extremely poor conditions with a concrete strength equivalent to or less than 10 N/mm².

3. Target Building and Flat Plate Joint

This study focuses on an interior flat plate-column connection representing the subassembly of a typical prototype structure, as shown in Figure 3A. The span and thickness of the flat plate were 6 m between the two columns and 150 mm, respectively. The target building is a typical flat plate structure constructed using low-strength concrete in Bangladesh, which is more vulnerable to seismic action than similar structures in other Asian countries because of the use of brick chips as coarse aggregates. Brick aggregates were applied to the test specimens to simulate low-strength concrete. Figure 3B presents the result of the compressive strength test performed on a drilled concrete core obtained from an existing building in Dhaka city. As similar low-strength concrete with a compressive strength lower than 10 N/mm² is common, a concrete strength of approximately 6–7 N/mm² was used to design the test specimens, as shown in Figure 3B.

4. Experimental Program

4.1 Specimen details

The experimental research was conducted on a series of three half-scaled specimens. Specimen named FP was not strengthened and served as a control specimen for comparison with other strengthened specimens. Additionally, two specimens, FP-W1 and FP-W2, which had RC wing walls, were constructed to investigate the effectiveness of the strengthening method of installing a wing wall.

4.1.1 Control specimen FP

Figure 4 shows the details of the specimen. The plate dimensions were 1500 mm × 1100 mm × 75 mm in length × width × thickness and based on a model at half scale. Two layers of deformed bars with a nominal diameter of 6 mm were arranged for the plate at a spacing of 75 mm in the longitudinal and transverse directions. The column was located at the centroid of the plate with a cross-section of 175 mm × 175 mm. In total, eight 16 mm deformed bars were arranged as the main reinforcement, and 6 mm deformed bars with 135° hooks were provided at a spacing of 50 mm for shear reinforcement in the column. However, no column shear reinforcement was provided to the plate-column connections.
A column bottom stub with dimensions of 400 mm × 400 mm × 300 mm in width × depth × height, respectively, was constructed with the existing structure to ensure perfect fixing with the loading frame.

Two steel plates having a thickness of 6 mm and width of 80 mm (SS400 steel plate: according to the Japanese Industrial Standard (JIS)) were embedded on the top and bottom surfaces on both sides of the plate, as shown in Figures 4 and 5, to increase the flexural strength of the plate, resulting in prior punching shear failure. Ten studs with a diameter of 8 mm were installed on the steel plates for better integration between the steel plates and concrete, as shown in Figure 6. As a result, the designed ultimate flexural strength of the plate was sufficiently higher than the designed punching shear strength.

4.1.2 Strengthened specimens: FP-W1 and FP-W2

The process used to construct the existing parts was the same as that used to construct the control specimen FP, except for the construction of the base stub, which was enlarged to anchor the postinstalled wing walls. After curing the existing concrete parts, RC wing walls were installed beside the column along the loading direction for specimen FP-W1. However, the direction of the postinstalled walls was orthogonal for the specimen FP-W2. The thickness of the wing walls was 100 mm, which is half (specimen scale) of 200 mm, considering the minimum thickness required in the Japanese design guidelines for retrofitting RC buildings. The length of the wing walls was set to 175 mm, which is equivalent to the column depth. Four D6 deformed vertical rebars and two D6@175 mm horizontal rebars were installed in the wing walls. The aforementioned details of the wing walls were identical for both specimens, and the directions of installation were orthogonal. The details of both specimens are shown in Figure 7. Practical construction of the wing walls into the existing buildings can be referred to the Japanese design guidelines.

4.2 Mechanical properties of the materials

The concrete mixture was designed based on a target concrete compressive strength of 6.47 N/mm², which was derived from the compressive strength of low-strength concrete commonly found in Bangladesh (Figure 3B). Portland cement with mountain sand acting as fine aggregate and brick chips acting as coarse aggregate with a maximum aggregate size of 15 mm was used to simulate the low-strength concrete characteristics used in Bangladesh for the existing parts of the specimens. The mix ratio of cement:sand:brick chip of 1:2:4 was designed. However, 30% of the cement was replaced by CaCO₃ to decrease the concrete strength. The ratio of water to the sum of cement and CaCO₃ was 60%. Table 1 lists the concrete

Figure 5. Placement of steel plates

Figure 6. Installation of studs

Figure 7. Details of the strengthened specimens: units are in mm
mixture by weight for the existing parts of the specimens. The concrete used for the existing parts was mixed in a mixing machine. However, a normal ready-mixed concrete with a design strength of 30 N/mm² with stone chips acting as coarse aggregates with a maximum size of 15 mm was used for the wing walls and bottom stubs. The average properties of concrete determined from the cylindrical compression tests are listed in Table 2. The steel grades used in this study were SD295 (deformed rebar with a nominal yield stress of 295 N/mm²) for the D6 reinforcement and SD345 (deformed rebar with a nominal yield stress of 345 N/mm²) for the D16 reinforcement. The average material properties of the reinforcements determined from the tensile tests are listed in Table 3.

### 4.3 Loading and measurement methods

A schematic view of the experimental setup of the flat plate specimen is shown in Figure 8. The specimens were tested using a static loading system in the concrete structure laboratory at Daido University. The plate and column were supported by a bottom stub bolted to the base beam of the loading frame. Various continuous measurement systems, including load cells, linear variable differential transducers (LVDTs), and strain gauges, were implemented. The loading system, which was supported by the top beam of the loading frame, consisted of two vertical jacks with a capacity of 500 kN each. The jacks were manually operated so that the displacements δ₁ and δ₂ shown in Figure 9 were almost equal at each step of the loading process during the experiment. The loading of the specimen was given using two vertical hydraulic jacks in the following manner: the right hydraulic jack pushed the specimen while the left hydraulic jack pulled simultaneously during the positive loading direction, and, the right hydraulic jack pulled the specimen while the left hydraulic jack pushed simultaneously during the negative loading direction. Four CDP-100 (D1, D2, D15, and D16) LVDTs were installed underneath the four corners of the plate for deflection measurements, as shown in Figure 9. Loading was controlled by the

### Table 1. Concrete mixture in kg/m³

| W/(Cement+CaCO₃) | Water | Cement | CaCO₃ | Sand | Brick |
|-----------------|-------|--------|-------|------|-------|
| 60%             | 212   | 248    | 88    | 596  | 959   |

### Table 2. Material properties of concrete

| Specimen type | Compressive strength (N/mm²) | Elastic modulus (N/mm²) | Strain at compressive strength (µ) |
|---------------|------------------------------|-------------------------|----------------------------------|
| FP            | 6.93                         | 8,450                   | 1,829                            |
| FP-W1         | 7.16                         | 8,672                   | 1,694                            |
| FP-W2         | 7.20                         | 8,741                   | 1,586                            |
| Wing wall     | 37.04                        | 30,929                  | 1,956                            |

### Table 3. Averaged properties of the reinforcement

| Specimen no. | Reinforcement type | Yield stress (N/mm²) | Elastic modulus (kN/mm²) | Tensile strength (kN/mm²) |
|--------------|--------------------|---------------------|--------------------------|--------------------------|
| FP           | D6 (SD295A)        | 346                 | 185                      | 517                      |
|              | D16 (SD345)        | 432                 | 245                      | 566                      |
| FP-W1 and FP-| D6 (SD295A)        | 381                 | 201                      | 505                      |
| W2           | D16 (SD345)        | 369                 | 198                      | 536                      |

Vertical reversed cyclic loads with increasing amplitude were applied to the plate ends, which represented the bending moment distribution near the plate and column junction under earthquake loading (refer to Figure 3A). The hydraulic jacks were manually operated so that the displacements δ₁ and δ₂ shown in Figure 9 were almost equal at each step of the loading process during the experiment. The loading of the specimen was given using two vertical hydraulic jacks in the following manner: the right hydraulic jack pushed the specimen while the left hydraulic jack pulled simultaneously during the positive loading direction, and, the right hydraulic jack pulled the specimen while the left hydraulic jack pushed simultaneously during the negative loading direction. Four CDP-100 (D1, D2, D15, and D16) LVDTs were installed underneath the four corners of the plate for deflection measurements, as shown in Figure 9. Loading was controlled by the
drift angle \( R \), which was defined by Equation (1) (also refer to Figure 9):

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

The symbols used in this equation are defined in the LVDT setup shown in Figure 9, where \( \delta_1 \) and \( \delta_2 \) are the drift angle in the right and left sides of the plate, respectively; \( l \) is the distance between the loading points on both sides of the plate; and \( D_1, D_2, D_{15}, D_{16} \) are the displacements at designated points. The loading protocol with one cycle for each incremental amplitude was applied to the specimen, as illustrated in Figure 10.

5. Experimental Results

The nodal moment versus \( R \) relationships of the specimens are explained in this section. The nodal moments applied to the specimens were calculated as the summation of bending moments obtained by multiplying the absolute vertical forces applied to the slab by both hydraulic jacks by 600 mm and the distance from the application point to the center of the column, as shown in Figure 9.

5.1 Failure process

5.1.1 Specimen FP

Specimen FP was tested without strengthening and served as the control specimen. The relationship between the nodal moment and drift ratio of this specimen is shown in Figure 11A. An initial flexural crack occurred at the corner of the column at \( R = -0.33\% \) rad. Subsequently, flexural cracks appeared and radially extended from the corners of the column with an angle of 45° relative to the loading direction at an \( R \) value of +0.5% rad, as shown in Figure 12. The maximum strength in the positive direction was 7.3 kN m during the cycle to +2.35% rad with an increase in the diagonal cracks. The maximum strength in the negative direction was 6.1 kN m during the cycle to 2.0% rad. Thereafter, the strength started to decrease as the concrete surface lifted due to punching shear failure. The punching shear failure was fully visible at an \( R \) value of 3.33% rad during positive loading. The ultimate damage to the top surface of the slab is shown in Figure 11B. The detailed failure process and crack patterns at different drift levels are shown in Figure 12, where the damage on the top surface of the slab at punching shear failure is marked by a thick black line. The top surface of the concrete peeled up to 175 mm from the edge of the column in the loading direction. The failure mode of this specimen was classified as brittle punching shear failure. The maximum strength of 7.3 kN m was smaller than the calculated punching shear strength of 9.9 kN m according to AIJ standards (defined as \( nM_0 \) in Equation 5).

5.1.2 Specimen FP-W1

Specimen FP-W1, which was strengthened with postinstalled wing walls installed along the loading direction of the column, showed higher stiffness and strength, and lower deformation capacity. The relationship between the nodal moment and drift ratio \( R \) of the specimen is shown in Figure 13A. An initial flexural crack occurred at the corner of the wing wall at an \( R \) value of 0.167% rad in the positive cycle. Afterward, the cracks radially extended approximately 45° relative to the loading direction from the corners of the wing wall at an \( R \) value of 0.5% rad. The maximum strength was 19.9 kN m during positive and negative loading in the cycle at 1.33% rad. Subsequently, the strength decreased as the concrete surface lifted due to punching shear failure. The ultimate damage to the specimen is shown in Figure 13B. The detailed failure process and crack patterns for different cycles are shown in Figure 14, where the ultimate damage due to punching failure on the top surface of the slab is marked by thick black lines. Punching shear failure was observed at the top and bottom surfaces of the slab. It was observed that the top concrete surface peeled up by approximately 160 mm
from the edge of the wing wall in the loading direction. Additionally, brittle punching shear failure was observed in this specimen, as described in Section 4.1. The maximum strength was 19.9 kN m, which is close to the calculated punching shear strength of 20.5 kN m according to the AIJ standards\(^2\) (\(M_0\) in Equation 5).

### 5.1.3 Specimen FP-W2

Specimen FP-W2, which was strengthened with postinstalled wing walls along the orthogonal loading direction of the column, showed higher strength but did not exhibit higher initial stiffness than the control specimen. The relationship between the nodal moment and drift ratio for this specimen is shown in Figure 15A. An initial flexural crack occurred at the corner of the wing wall at an \(R\) value of +0.5% rad. Then, the cracks extended approximately 45° relative to the loading direction from the corners of the column during the negative cycle at an \(R\) value of 0.67% rad. The maximum strength was 14.7 kN m during positive loading in the cycle at 4.82% rad. The maximum strength in the negative loading direction was 14.1 kN m at 4.74% rad, but the strength did not decrease significantly as \(R\) increased to 6% rad. Subsequently, the bottom surface of the concrete peeled up slightly from the edge of the column in the loading direction at approximately 200 mm. In this specimen, punching shear failure was observed on the bottom surface of the slab, as shown in Figure 15B. Additionally, from the strain gauges, it was observed that the plate rebar started to yield at an \(R\) value of 4.2%, indicating that flexural yielding occurred...
in the plate before punching damage was observed (described later in Section 5.2). Consequently, the failure mode of this specimen was categorized as ductile flexural failure, followed by punching shear failure. The detailed failure process and crack patterns at different cycles are shown in Figure 16. The maximum strength of 14.7 kN m was smaller than the calculated punching shear strength of 20.2 kN m according to the AIJ standard

$$M_{0}$$

(5).

5.2 Strain distribution of the slab reinforcement

The strains were obtained from strain gauges installed on the top of the reinforcing bars of the slab along the loading direction. Although many strain gauges were installed in the slab reinforcement, the major strain responses during positive loading are discussed in this section. The reinforcements were highlighted based on two categories, where A (blue lines) was defined as the exterior side in which the reinforcements were located 150 mm from the side surface of the column, and B (green lines) was defined as the interior side located along the side surface of the column, as shown in Figures 17, 18, and 19 for specimens FP, FP-W1, and FP-W2, respectively. By comparing the strains of the reinforcing bars along A and B, it was observed that the strains on the interior side increased with increasing drift angle for all specimens. The strain responses were much higher in the critical sections shown in Figures 17, 18, and 19 for all the specimens. These strain responses demonstrated that the plate not only deformed in bending but also in torsion around the column. For specimens FP and FP-W1, no strain gauges yielded up to the peak load, as shown in Figures 17 and 18, thereby showing that punching shear failure occurred before flexural yielding of the plate, as described in Section 4.1. For specimen FP-W2, one strain gauge yielded before the specimen reached the maximum strength owing to the flexural behavior of the plate near the column surface at an $$R$$ value of 4.2% rad, as shown in Figure 19. Later, other strain gauges also started to yield with an increase in the drift ratio. The strain responses of FP-W2 were different from those of FP and FP-W1. This indicates that the failure process of FP-W2 was different from that of the other two specimens, as described in Section 5.1.

6. Strength Estimation for the Specimens

The bending moment diagrams of the specimens under static seismic loads are shown in Figures 20 and 21. The ultimate strength of all specimens was evaluated during the design stage. The ultimate strength of specimen $$M_{u}$$ represents the joint nodal moment and failure mode, which were evaluated using Equation (2):

$$M_{u} = \min[\mu M_{y}, M_{0}, cM_{u}],$$

(2)

where $$\mu M_{y}$$ is the nodal moment at the plate-column connection when the plate yielded in flexure (N mm), $$M_{0}$$ is the punching shear capacity at the critical section of the column (N mm), $$cM_{u}$$ is the nodal moment at the joint when the column (FP, FP-W2) or column with wing walls (FP-W1) yielded (N mm).
Figure 17. Comparison of strain distribution transitions under different cycles for specimen FP

Figure 18. Comparison of strain distribution transitions under different cycles for specimen FP-W1

Figure 19. Comparison of strain distribution transitions under different cycles for specimen FP-W2
The capacity of the flexural strength of the plate \( M_f \) was obtained by flexural section analysis, where the following assumptions were made:

a) The strain distribution in the cross-section is linear.

b) The stress–strain relationship of the reinforcement is assumed to be a bilinear model in which the yield stress and elastic modulus shown in Table 3 were used.

c) The stress–strain relationship of concrete was assumed by using the model proposed by Hognestad et al.\(^{28}\) where the strain at the compressive stress was 2\%.

The conversion of \( M_f \) into the nodal moment of the joint \( M_s \) is defined by Equation (3) for specimens FP and FP-W2 and Equation (4) for specimen FP-W1 (Figures 20 and 21):

\[
\begin{align*}
M_s &= 2M_f \frac{L_s/2}{L_s/2 - D_c/2} \\
M_s &= 2M_f \frac{L_s/2}{L_s/2 - D_c/2 - l_w}
\end{align*}
\]  

The designed punching shear capacities of the specimens were calculated based on the AIJ standard.\(^{27}\) The punching shear strength \( M_0 \) was calculated according to Equations (5), (6), (7), and (8) from the AIJ standard for RC structures. However, low-strength concrete was beyond the scope of these equations. Thus, this study investigates the applicability of these equations to specimens constructed using low-strength concrete. According to the AIJ standard, as shown in Figure 22, the moment resistance due to bending, shearing, and twisting of concrete in the critical cross-section around the column are summed in Equation (5):

\[
M_0 = M_f + M_s + M_t,
\]  

\[
M_f = 0.9a_0\sigma_c d\left(\frac{c_2 + d}{x_1} + 0.9a_0\sigma_c d\frac{c_2 + d}{x_b}\right),
\]  

\[
M_S = r_u(c_2 + d)(c_1 + d),
\]  

\[
M_t = r_u\left(\frac{c_1 + d}{3} - \frac{d}{3}\right)^2,
\]  

where \( a_0 \) and \( a_{0b} \) are the cross-sectional areas of the top of the plate and bottom rebar (mm\(^2\)), respectively; \( x_1 \) and \( x_b \) are the spacings of the top and bottom rebar (mm), respectively; \( \sigma_c \) is the yield stress of the rebar (N/mm\(^2\)); \( d \) is the effective depth of the plate (mm); \( c_1 \) is the column depth in the loading direction (mm); \( c_2 \) is the column width in the direction orthogonal to the loading direction (mm); \( r_u \) is the direct shear strength of concrete (N/mm\(^2\)), where \( r_u = 0.335\sqrt{\sigma_B}; \sigma_B \) is the compressive strength of concrete (N/mm\(^2\)); and \( \tau_u \) is the torsional shear strength of concrete (N/mm\(^2\)), where \( \tau_u = 6\tau_y \).  

The critical sections were assumed to be at a distance of \( d/2 \) from the column shown in Figure 23.

In addition to the normal application of Equations (5) to (8), some modifications were applied in this study to consider low-strength concrete. Miyauchi et al.\(^{29}\) proposed a formula for direct shear strength \( \tau_u \) to briefly estimate the compressive strength of low-strength concrete as shown in Equation (9), when the concrete strength was equal to or less than 13.5 N/mm\(^2\), based on their experimental investigation. Therefore, the components of shearing resistance \( M_s \) and twisting resistance \( M_t \) of the plate were reevaluated to compute punching shear strength \( M_{0b} \) considering the application of low-strength concrete. Furthermore, the bending resistance of the plate \( M_f \) was reevaluated using section analysis in the critical zone in the same manner that \( M_s \) was evaluated.

\[
\tau_y = 0.38\left(\sqrt{13.5/13.5}\right)\sigma_B \quad \text{(where } \sigma_B \leq 13.5 \text{ N/mm}^2)\]  

The flexural strength of the column and column with wing walls \( M_f \) were obtained by flexural section analysis in the same manner as \( M_s \).

According to the aforementioned calculations, the expected failure mode was estimated to be punching failure for all specimens. However, specimen FP-W2 experienced prior flexural yielding, as described in Section 5.1.3. The details of the calculated results and experimental strengths are presented in Table 4.

7. Discussion of the Proposed Strengthening Scheme

The performance comparisons of the strengthened specimens and control specimen are shown in Figure 24 using backbone curves. Punching shear failure occurred in the control specimen FP at a drift ratio of 2.35%. The strengthened specimen FP-W1 exhibited a significant increase in strength by 2.7 times, showing higher stiffness than the control specimen while punching shear failure occurred at a drift ratio of 1.35%. However, the strengthened specimen FP-W2 exhibited a two-fold increase in strength. Therefore, it can be concluded that
both strengthened specimens showed the effectiveness of the wing wall for improving the punching shear resistance of the flat plate-column connection.

The experimental strengths were compared with the design calculations, as listed in Table 4. The experimental maximum strengths of all the specimens were lower than the design

Table 4. Calculated ultimate strengths compared to the ultimate strengths obtained from the experimental data

| Specimen no. | Experimental strengths, $M_{exp}$ (kN m) | Flexural strength of the slab, $M_y$ (kN m) | Calculated strength by AIJ $M_0$ (kN m) [$M_{exp}/M_0$] | Calculated strength considering low-strength concrete $M_0l$ (kN m) [$M_{exp}/M_0l$] | Experimental failure mode |
|-------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|----------------------------|
| FP          | 7.3                             | 15.8                           | 9.85 [0.74]                     | 8.3 [0.88]                      | Punching                   |
| FP-W1       | 19.9                            | 26.1                           | 20.5 [0.97]                     | 17.2 [1.15]                     | Punching                   |
| FP-W2       | 14.7                            | 17.2                           | 20.2 [0.85]                     | 18.0 [0.85]                     | Flexure followed by punching |

Figure 22. Punching shear mechanism due to bending, shearing, and twisting

Figure 23. Assumed critical section

Figure 24. Performance comparisons of the strengthened specimens with that of the control specimen

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strengths according to the AIJ standards. In particular, the maximum punching shear capacity of FP was 26% lower than the calculated value. This was probably caused by the utilization of low-strength concrete in the specimens. Therefore, the previous section presented two modifications: the application of sectional bending analysis and the direct shear strength equation proposed by Miyuchi et al. (2009). Consequently, the estimate of the punching shear strength for specimen FP was improved by approximately 10%. Moreover, specimen FP-W1 strengthened with wing walls in the in-plane direction was conservative when considering low-strength conditions. However, for FP-W2 with wing walls in the out-of-plane direction, the estimate was 15% higher than the experimental value, which was likely caused by the flexural yielding of the flat plate, as described in the experimental results. In this study, it was confirmed that the estimate of the punching shear strength was acceptable for two flat plate specimens strengthened with and without using wing walls in the in-plane direction. However, further research, including finite element modeling and analysis, is necessary, particularly to validate these equations in terms of the failure mechanism of flat plate structures constructed using low-strength concrete.

8. Conclusions

In this investigation, a new strengthening scheme was proposed for the seismic retrofitting of an RC flat plate structure. A series of experiments were conducted involving three half-scaled interior flat plate-column connections with two specimens strengthened with RC wing walls to investigate the effectiveness of the proposed scheme and to evaluate the seismic behavior and performance. Based on the results obtained from the experiments, the following conclusions can be drawn:

In the experiments, specimens FP and FP-W1 showed punching shear failure as intended in their design, whereas FP-W2 showed plate rebar yielding followed by punching. Flexural cracks were prolonged in the 45° direction from the column/wing wall surface, whereas punching cracks extended radially from the column/wing wall surface. Finally, the surface concrete peeled due to punching shear failure.

The RC wing wall was effective in strengthening the flat plate structure. By applying in-plane direction strengthening for FP-W1, the maximum strength was 2.7 times larger than that of the control specimen FP. In the case of out-of-plane direction strengthening for FP-W2, the maximum strength was two times larger than that of the control specimen. Additionally, the former strengthening scheme contributed to stiffening the flat plate structure, but the latter scheme provided no additional stiffness.

The aforementioned increase in the resistance to punching shear failure by RC wing walls was attributed to the enlargement of the punching shear failure surface, which was considered in the punching shear strength estimations.

The punching shear strength estimated based on the AIJ standards overestimated the maximum capacity of the flat plate specimen using low-strength concrete. Thus, consideration of the low-strength concrete conditions improved the estimation but provided an overestimation of approximately 10%. However, the maximum capacity of the specimen strengthened with wing walls in the in-plane direction (FP-W1) was conservatively estimated by considering the low-strength conditions, whereas that of the specimen with wing walls in the out-of-plane direction (FP-W2) did not reach the estimated value because of prior flexural yielding of the flat plate.

As mentioned above, the proposed retrofitting scheme using RC wing walls was experimentally verified to be effective in upgrading flat plate-column connections constructed using low-strength concrete. However, further investigations are recommended to evaluate the punching shear strength. In particular, finite element analysis may clarify the punching shear failure mechanism in flat plate structures with low-strength concrete. The present experimental data will be an excellent source for calibrating future studies using finite element analysis.

Acknowledgments

This research was supported by the JST/JICA SATREPS-TSUBI project “Technical Development to Upgrade Structural Integrity of Buildings in Densely Populated Urban Areas and Its Strategic Implementation towards Resilient Cities in Bangladesh (TSUBI)” (principal investigators: Prof. Yoshiaki Nakano, U. Tokyo, and Mr. Mohammad Shamim Akhter, HBRI, Bangladesh (https://www.jst.go.jp/global/english/kadai/h2712_bangladesh.html)). The authors thank Prof. I. Anam and Dr. S. Suzuki for their assistance with the experiments. The authors gratefully acknowledge the support for this research.

Disclosure

The authors have no conflicts of interest.

Data Availability Statement

The data that support the findings of this study are available on request from the corresponding author. The data are not publicly available due to privacy or ethical restrictions.

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**How to cite this article:** Samdani HMG, Takahashi S, Yoon R, Sanada Y. Strengthening seismically vulnerable reinforced concrete flat plate-column connections by installing wing walls. *Jpn Archit Rev*. 2021;4:442–454. https://doi.org/10.1002/2475-8876.12232