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Structural Behavior of Brick Wall Specimens Reinforced on the Surface with RC Walls under Horizontal Loading

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Abstract

This study aims to develop and propose a seismic retrofitting method for unreinforced brick masonry for improving the seismic safety of the 1st and 3rd headquarters buildings of Kyushu University. Five brick wall specimens representing parts of the walls that were adjacent to the openings in the headquarters buildings were prepared on a 3/4th scale of the actual structure and they were reinforced on either one or sides of their surfaces with reinforced concrete (RC) walls. Horizontal loading experiment was conducted to confirm the reinforcing effect. Results indicated that the maximum load of the specimens (CS01, CS02) reinforced on one side with RC walls was 6.1 to 6.2 times higher than that of the unreinforced specimen. Similarly, the maximum load of the specimens (CD01, CD02) reinforced on both sides with RC walls was 12.6 to 14.2 times higher than that of the unreinforced specimen. The specimen CS02 reached its maximum load at a smaller deformation angle compared to the specimen CS01 because of the effect of twisting. Additionally, the authors also derived and verified a horizontal strength evaluation formula for brick wall specimens reinforced with RC walls. The values calculated using horizontal strength evaluation formulas were close to the experimental values.

1. Introduction

Owing to the heavyweight and brittleness of masonry buildings, they can easily collapse during earthquake, which causes various casualties and property losses (Bhattacharya et al. 2014). Damage to unreinforced masonry (URM) buildings in earthquakes comprises both out-of-plane failures and in-plane failures of the walls (Liu and Crewe 2020). Owing to the weak out-of-plane behavior of such walls or inadequate diaphragms or diaphragm-wall connections, URM buildings mostly fail when loaded in-plane (Mahmood and Ingham 2011). The out-of-plane failure could be prevented by additional structural elements. However, the overall seismic performance of URM buildings depends on the capacity of in-plane walls to transfer the lateral loads safely to foundations, providing the post-earthquake stability, which is required to avoid collapse of the entire structure (Lourenço 1996). Shear, sliding, rocking and toe crushing failures are common in in-plane mechanisms. Shear and sliding failures are commonly observed in URM structures (Mustafaraj and Yardim 2018). Various strengthening techniques have been used to increase the tensile and shear strength and improve the structural performance of URM structures under seismic loading (Mustafaraj and Yardim 2018, 2019; Kalali and Kabir 2012). Filling cracks and voids by grouting, stitching of large cracks and weak areas with metallic or brick elements, external or internal post-tensioning with steel ties, shotcrete jacking, ferro-cement and center core are various conventional reinforcement techniques for brick masonry (Mustafaraj and Yardim 2018, 2019; Kalali and Kabir 2012). The ferro-cement jacking technique is commonly used in concrete and masonry structures (Winokur and Rosenthal 1982).

In recent years, fiber-reinforced plaster and textile-reinforced mortar have been used for retrofitting of masonry buildings. Wang et al. (2006) reported that masonry walls retrofitted with glass-fiber-reinforced plaster increased the load-carrying capacity of masonry walls subjected to in-plane shear loading. Mishra et al. (2019, 2020b) and Endo et al. (2020) reported that timber reinforcement and steel reinforcement increased the shear strength of URM specimens by 198% and 278%, respectively, compared with the unreinforced specimens in Nepalese masonry buildings.

Reinforced concrete (RC) jacking is a retrofitting technique that involves the application of single- or double-sided RC walls or coatings (Churilov and Dumova 2012). This technique is suitable for stone and brick masonry. Alcocer et al. (1996), Penazzi et al. (2001) and ElGawady et al. (2004) reported that RC
jacketing improves the lateral capacity of brick masonry. Mishra et al. (2020a) reported that the shear strengths of double-sided steel and double-sided RC reinforcement were approximately 1.56 and 1.92 times higher, respectively, than those of the unreinforced specimens. The RC reinforcement exhibited higher strength and rigidity than the steel plate reinforcement. The initial stiffness of the RC double-sided reinforcement was approximately 1.70 times higher than that of the double-sided steel reinforcement (Mishra et al. 2020a).

Surface reinforcement method was employed in this study. The inside faces of external walls and most of the inner walls of the 1st and 3rd headquarters buildings of the Kyushu University were plastered with cement mortar. The original appearance of buildings did not change when the masonry walls were reinforced from the surface on both the inside faces of external walls and most of the inner wall. Additionally, the thickness of brick walls for both buildings was larger compared to those of the beams and slabs when they were connected to the reinforcing part with anchors. Therefore, damage could occur and the expected strength of the anchor could not be exhibited. It is suitable to reinforce walls than beams or slabs. Moreover, it is effective to reinforce the visible parts of each floor from the inside surface without changing the beams, slabs and walls of the floors. Furthermore, surface reinforcement is economical and easy to construct.

The mechanical properties of the unreinforced brick masonry obtained from the actual structures of the 1st and 3rd headquarters buildings of Kyushu University were investigated to obtain basic data to confirm the seismic resistance of the buildings (Koshi et al. 2014a, 2014b). Furthermore, a reinforcement method that could maintain the appearance and reduce the cost of the 1st and 3rd headquarters buildings of Kyushu University was investigated. Surface reinforcement with RC walls was selected for this study because it has higher strength and rigidity than steel plate reinforcement (Mishra et al. 2020a). Brick wall specimens were prepared and reinforced from the surface on one side or two sides with RC walls and a horizontal loading experiment was carried out to confirm the reinforcing effect. In addition, the authors derived and verified an evaluation formula for the horizontal strength of brick wall specimens reinforced with RC walls.

2. Materials and Methods

2.1 Experimental Program

The inner walls of the 1st headquarters building of the University are shown in Fig. 1(a). Figure 1(b) shows the cross section of the wall of the actual structure. As shown in Fig. 2, the specimen has an inverse T-shaped configuration. The upper part of the specimen represents a side of the opening and the longer lower part the spandrel wall. The upper part of the specimen is hereinafter re-
ferred to as the upper wall and the longer lower part is hereinafter referred to as the spandrel wall. The bending moment is higher at the upper walls of the 1st headquarters building of the Kyushu University, which is nearly double that at the spandrel wall.

A horizontal loading experiment was performed to investigate the effectiveness of the retrofitting method from the surface with RC walls on one or two sides. The experiment was conducted at the Laboratory of Structural Engineering, Department of Architecture, Kyushu University, Japan.

2.2 Mechanical properties of bricks, mortar and reinforced concrete

The mechanical properties of the brick and mortar used for the brick walls are summarized in Table 1. The bricks used were non-perforated fired bricks with dimensions of 216×108×63 mm, with an average compressive strength of 22.6 N/mm², similar to that of the bricks used in the 1st and 3rd headquarters buildings of the Kyushu University (Koshi et al. 2014a, 2014b). The mortar composition ratio (cement, sand and water) for brick wall was 1:3.25:0.614. The composition was determined through estimation and was performed using two trial mixes. For the RC walls, the ordered concrete had a nominal strength of 42 N/mm², slump flow of 60 cm and maximum size of coarse aggregate of 13 cm. The concrete for casting the RC walls was of high fluidity. An air-entraining superplasticizer (5.5 kg/m³) was also added. The water cement ratio was 60%. Ordinary Portland cement was used in this study. The compressive strength of concrete for all walls was higher than 60 N/mm². It is possible that additives used in concrete with less water content might have led to such high strength of concrete.

The compressive strengths of mortars and concrete were determined using compression tests on 3-cylinder specimens for each wall. The mechanical properties of the reinforced concrete used for the RC wall are summarized in Table 2. The difference in the reinforcement and the loading methods for the five specimens are summarized in Table 2. The mechanical properties of the reinforcing bars were determined using tensile tests and are summarized in Table 3.

2.3 Description of test specimens

Owing to experimental limitations, all test specimens were scaled down to 3/4th of the original wall. That is, the full scale of the original wall was significantly large for the experiment and it was difficult to conduct experiments on a full scale. The bricks were not scaled down owing to the limited time period of this project and insufficient budget. Five brick wall specimens, which represented the walls around the openings in the actual structure, were prepared. Among them, one was unreinforced masonry specimen (specimen UR01), two were reinforced by an RC wall on one side (specimens CS01 and CS02) and two were reinforced by RC walls on both sides (specimens CD01 and CD02). All the RC walls had a thickness of 110 mm. The minimum thickness required for the RC walls was 110 mm because of the rebars detailing, anchor layouts and minimum concrete cover. The brick walls were set on the RC stub and the reinforce-

| Table 1 Mechanical properties of brick, mortar and reinforced concrete. |
|-------------------|-----------------|-----------------|-----------------|-----------------|
| Specimen | Compressive strength (N/mm²) | Young’s modulus (N/mm²) | Compressive strength (N/mm²) | Young’s modulus (N/mm²) |
| UR01 | 22.6 | 0.52×10⁹ | 43.3 | 2.34×10⁹ |
| CS01 | 22.6 | 0.52×10⁹ | 45.1 | 2.56×10⁹ |
| CD01 | 22.6 | 0.52×10⁹ | 43.3 | 2.34×10⁹ |
| CS02 | 22.6 | 0.52×10⁹ | 43.4 | 2.41×10⁹ |
| CD02 | 22.6 | 0.52×10⁹ | 43.4 | 2.41×10⁹ |

| Table 2 Reinforcement and loading method of specimens. |
|-------------------|----------------|-----------------|-----------------|
| Specimen | No of anchors | Anchor rod ratio (%) | Twist restraint |
| UR01 | No | 0 | Yes |
| CS01 | One side | 100 | 0.56 | Yes |
| CS02 | One side | 100 | 0.56 | No |
| CD01 | Both sides | 100 | 0.56 | Yes |
| CD02 | Both sides | 60 | 0.33 | Yes |

| Table 3 Mechanical properties of reinforcing bars. |
|-------------------|----------------|-----------------|-----------------|
| Standard | Young’s modulus (N/mm²) | Yield strength (N/mm²) | Tensile strength (N/mm²) | Ultimate elongation (%) |
| D10(SD345) | 1.84×10⁵ | 382 | 594 | 19.2 |
| D16(SD345) | 1.85×10⁵ | 403 | 585 | 17.9 |
| D16(SD345)* | 1.84×10⁵ | 401 | 576 | 16.9 |

*Used for specimen CD02
ment part was cut off and not embedded in the RC stub [Fig. 5(d)].

2.4 Arrangement of rebars and anchors

The URM walls were constructed in the laboratory. The drilling of holes and fixing of anchors on the brick wall specimens were also carried in the laboratory and are shown in Fig. 3. In the specimens CS01 and CS02, the anchors were embedded and placed at a depth of 295 mm and through the brick wall in the specimens CD01 and CD02. However, it was necessary to shorten the embedding length slightly because five places were accidentally

![Fig. 3 Drilling of holes and fixing of anchors on wall specimens.](image-url)
penetrated when drilling the holes of the specimen CS02 [Fig. 3(c)]. The laying of reinforcement and casting of RC walls are shown in Fig. 4. The rebars were placed on the surface of the brick walls and concrete was casted. Hence, the RC walls were cast in the same laboratory.

Figure 5 shows the shapes of the rebar arrangements, anchor positions and strain gauge positions of the five specimens, with Fig. 5(a) showing the front view of the UR01 specimen and Fig. 5(b) showing the reinforcement diagram of the specimens CS01 and CD01. Figure 5(c) shows the reinforcement of the specimen CD02 along with the strain gauges attached on rebars, while Fig. 5(d) shows the cross section of the specimens CS01/CS02 (left side) and CD01/CD02 (right side). D13 bars of SD345 type were used for the anchor rods and epoxy resin was used as an adhesive to fix the anchors. D16 bars of SD345 type were used as main rebars at the ends of the upper walls and the spandrel walls. D10 bars of SD345 type were used as the intermediate rebars in the vertical direction at the center. D10 bars of SD345 type with 90° hooks were used as shear reinforcement bars. As the maximum forces are at the sides of the upper walls, where significant displacement could occur, higher diameter rebars were used as flexural reinforcement. One hundred anchors were placed in each of the specimens CS01, CS02 and CD01. However, only sixty anchors were placed in the specimen CD02. The anchor bar ratio is summarized in Table 3. An index showing the anchor amount is defined as the ratio of the cross-sectional area of the anchor bars to the area of the brick wall. Mishra et
al. (2020a) investigated the behavior of reinforced masonry through the diagonal compression test where the anchor was embedded with a length of one brick edge.

During the splitting in all masonry specimens, the cracks were observed at the tip of the embedded anchor as shown in Fig. 6. Therefore, in this study, the masonry wall was reinforced by penetrating the anchor rods through the brick masonry.

2.5 Description of test procedures

Figure 7 shows the loading setup. The shear span to depth ratio of the specimens was 1.20. The arrangement of the twist-restraining device is shown in Fig. 8. An axial force of 54 kN, which was equivalent to the axial force generated on the second floor of the 1st headquarters building of Kyushu University, was applied (compressive stress: 0.21 N/mm²) on the upper surface of the brick masonry wall (excluding RC wall) and was main-
tain at a constant level throughout the experiment. The RC wall was connected to the URM brick wall by anchors, but the axial load was not transferred to the RC wall after strengthening. This is because the RC wall is to be added after years of construction of the existing walls. Generally, the slab loads transfer the force directly to the URM brick wall. If additional loads are added after attaching the RC walls, the loads are transferred to the attached RC walls. In this experiment, the additional load was ignored. An easy reinforcement method was considered in this experiment, wherein only the walls around the openings were strengthened. It is to be noted that, the RC walls are not connected to the RC stub with anchor and has no support as shown in Fig. 5(d).

The positions of the relative displacement meter and strain gauge on the tensile rebars are shown in Figs. 9(a)
and 9(b), respectively. Strain gauges were attached to the flexural reinforcement rebars at the lower part of the upper wall, where significant displacement could occur. The loading program and loading method are shown in Figs. 10 and 11, respectively. For the horizontal force, the deformation angles generated at the height from the top surface of the stub to the loading point were ±0.01, 0.025, 0.05, 0.075, 0.1, 0.2, 0.3, 0.4, 0.5, 0.75, 1.0, 1.5 and 2.0×10⁻² rad. Three altered loading cycles were carried out for each deformation angle (up to 1.5×10⁻² rad. for the specimen CS01 and up to 1.0×10⁻² rad. for the specimen CS02). Specimen CS02 was loaded without
restraining the twist in the out-of-plane direction to confirm the performance when affected by twisting. For the other specimens, loading was performed with the twist restrained in the out-of-plane direction. As shown in Fig. 11, the horizontal load was positive when the compressive force was applied from the jack and negative when the tensile force was applied. The deformation angle was positive when the stub moved to the positive owing to the compressive force of the jack and the deformation angle was negative when the stub was displaced to the negative side by the tensile force of the jack, as shown in Fig. 11.

Table 4 summarizes the axial forces before and after when the RC wall was added to the 1st headquarters building of Kyushu University. The axial compressive stress summarized in Table 4 was obtained from the seismic diagnosis of the building at the maximum axial force position of the walls. The value 0.569 MPa was obtained by dividing the axial force (275 kN) on the cross-sectional area of the brick wall at the maximum axial force position. Higher axial loads were applied to the retrofitted specimens to determine whether the bearing capacity was safe when the retrofitted walls were added and the weight was increased. The “minimum value of the compressive strength” in Table 4 that was obtained from the experimental results on masonry prisms taken from some walls of the building (Specimen OPC 111; compressive strength of 6.35 N/mm²) (Yamaguchi et al. 2014). As the foundation of the 1st headquarters building of Kyushu University is composed of similar brick walls whose cross-sectional area is gradually increased along the depth (Fig. 1) it is expected that the compressive strength is equal to or higher than that of the wall; therefore, the bearing capacity would be sufficient for long-term axial forces.

3. Results and discussion

3.1 Experimental observations
This section summarizes the experimental observations of the behavior, crack pattern and failure mode of the test walls subjected to horizontal cyclic loading. The relationship in terms of the horizontal load-deformation angles of each specimen is shown in Fig. 12. The marks in the graphs show when the main rebars in the first,
second and third position from the outside yielded. The states of the specimens after completion of the test are shown in Fig. 13.

3.1.1 Specimen UR01
The maximum load of the unreinforced specimen UR01 was 31.8 kN (at +0.041×10⁻² rad.) [Fig. 12(a)]. On the positive loading direction of the loading cycle +0.025×10⁻² rad., bending cracks occurred in the horizontal joint on the lower surface of the first brick layer of the upper wall. On the negative direction of the same loading cycle, the bending cracks occurred in the horizontal joint on the upper surface of the first brick layer. On the positive loading direction of the loading cycle +0.050×10⁻² rad., the bending cracks occurred in the horizontal joint on the upper surface of the first brick layer [Fig. 13(a)]. In the subsequent larger deformation cycles, the second and higher layers of the wall rotated rigidly around its bottom.

3.1.2 Specimen CS01
The maximum load of specimen CS01 was +196 kN (at +0.71×10⁻² rad.), which was 6.2 times higher than that of the specimen UR01 [Fig. 12(b)]. On the negative loading direction of the loading cycle -0.050×10⁻² rad., bending cracks occurred on the lower surface of the first brick layer on the unreinforced surface (front) of the upper wall. On the positive loading direction of the loading cycle...
cycle $+0.30 \times 10^{-2}$ rad., step-like cracks occurred on the surface of the masonry [Fig. 13(b), left side]. In addition, bending cracks occurred on the reinforcing surface (back surface) at the bottom of the upper wall. After a cycle of $\pm 0.75$ to $1.0 \times 10^{-2}$ rad., bending cracks were developed on the reinforcing surface at the bottom of the upper wall [Fig. 13(b), right side] and diagonal cracks were observed on the reinforcing surface. At $\pm 1.5 \times 10^{-2}$ rad., cycle, crushing in the concrete at the bottom of the upper wall was observed.

3.1.3 Specimen CS02
The maximum load of specimen CS02 was $+194$ kN (at $+0.51 \times 10^{-2}$ rad.), which was 6.1 times higher than that of specimen UR01 [Fig. 12(c)]. On the positive loading direction of the loading cycle $+0.10 \times 10^{-2}$ rad., the bending cracks occurred on the lower surface of the first brick layer on the unreinforced surface (front) at the bottom of the upper wall. On the positive loading direction of the loading cycle $+0.30 \times 10^{-2}$ rad., cracks were observed diagonally on the surface of the masonry [Fig. 13(c), left side]. On the positive loading direction of the loading cycle of $+0.20 \times 10^{-2}$ rad., bending cracks occurred on the reinforcing surface (back surface) at the bottom of the upper wall; furthermore, cracks progressed after the cycle of $\pm 0.50 \times 10^{-2}$ rad. [Fig. 13(c), right side].

3.1.4 Specimen CD01
The maximum load of the specimen CD01 was $+453$ kN (at $+0.88 \times 10^{-2}$ rad.), which was 14.2 times higher than that of specimen UR01 [Fig. 12(d)]. On the positive loading direction of the loading cycle $+0.20 \times 10^{-2}$ rad., bending cracks first occurred on the reinforcing surface (back surface) at the bottom of the upper wall. After that, cracks occurred on other side and progressed in the subsequent cycles [Fig. 13(d)]. In addition, crushing in the concrete was observed at the bottom of the upper wall after a cycle of $+0.75 \times 10^{-2}$ rad. The concrete cover at the toe of the upper wall started to fall off and cracks were observed. However, the spandrel did not crack. This could be because the bending moment in the spandrel wall was nearly half that of the opening wall.

3.1.5 Specimen CD02
The maximum load of specimen CD02 was $+402$ kN (at $+0.77 \times 10^{-2}$ rad.), which was 12.6 times higher than that of specimen UR01 [Fig. 12(e)]. On the positive loading direction of the loading $+0.10 \times 10^{-2}$ rad., bending cracks occurred on the reinforcing surface (front) at the bottom of the upper wall and the cracks continued to expand thereafter. Additionally, after a cycle of $+0.75 \times 10^{-2}$ rad., the concrete was crushed at the bottom of the upper wall as shown in Fig. 13(e).

Fig. 13 Cracks and failure mode of specimens.
3.1.6 Magnitude of the maximum horizontal load for each deformation angle

Figure 14 shows the magnitude of the maximum horizontal load for each deformation angle of each specimen. The load of the four reinforced specimens increased even after +0.041×10^{-2} rad., a level at which the maximum load of the unreinforced test specimen UR01 was measured. The main bars of the walls began to yield after the loading cycle ±0.20×10^{-2} rad. and the specimens exhibited a large deformation ability until the maximum load was reached.

3.2 Effect of twist restrain

Comparison between the load-deformation curves of the specimen CS01 (with restraining device) and specimen CS02 (without restraining device) is shown in Figure 15. The figure shows that the maximum loads of the specimens were similar, but that of the specimen CS02 was reached at a smaller deformation angle than that of the specimen CS01. This was attributed to the effect of twisting.

Figure 16 shows a conceptual diagram of the twist of the upper part of the wall. Figure 17(a) shows the relationship between the twist-overall deformation angles of the upper part of the specimens CS01 and CS02. Figure 17(b) shows the relationship between the twist-overall deformation angles of the upper part of the specimens CD01 and CD02.

The twist of the upper part of the single-sided reinforced specimen was significant. The upper twists of both specimens were almost similar up to deformation angle of ±0.40×10^{-2} rad. The twist of the upper part of specimen CS02 increased gradually from deformation angle of ±0.50×10^{-2} rad. The upper twist of specimen CS01 was approximately 2.2×10^{-2} rad. when the deformation angle of the specimen was ±1.0×10^{-2} rad. and that of specimen CS02 was approximately 5×10^{-2} rad. Notably, the restraining system was slightly loose, which caused a twist in the restrained specimen (Figure 8). However, the twist in the restrained specimen was

\[ \gamma = \frac{a - b}{t} \]

\( a \) : Relative displacement of the upper back
\( b \) : Relative displacement of the upper front
\( t \) : Distance between displacement meters at the top of the upper wall
\( \gamma \) : Twist at the top of the upper wall
2.2×10^{-2} \text{ rad.} \text{ at the end of the experiment, whereas it was } 5×10^{-2} \text{ rad.} \text{ at the middle of the experiment}

3.3 Effect of anchor's number
Comparison between the specimens CD01 (100 anchors) and CD02 (60 anchors) is shown in Fig. 18. Even though the specimen CD02 had a maximum strength approximately 11% lower than that of the specimen CD01, it did not show any strength degradation in the deformation region ±1.50×10^{-2} \text{ rad.}

3.4 Shear deformation angle
The shear deformation angle was obtained by calculating the shear deformation from the diagonal relative displacement meters installed on the upper part of the wall. Figures 19 and 20 show the relationship between the shear deformation angle and the overall deformation angle of the specimens CS02 and CD02. It can be observed that the shear deformation angle of both specimens increases with the increasing deformation angle. Additionally, the relationship horizontal load-deformation angle in Fig. 12 indicates that the horizontal load decreases with the shear failure.

4. Analytical study
4.1 Calculation method of horizontal strength of wall specimens
During the horizontal loading experiment of the wall specimen UR01, it was bent and experienced fracture at the lower joint of the upper wall, as shown in Fig. 13(a).

$$\frac{Q_w}{2} = \frac{N + w}{L} \cdot \sin \phi$$

where,

- $N$: Axial force acting on the upper surface of the brick masonry,
$w$: Self-weight of the upper wall,
$L$: Length of the upper wall, and
$h$: Height from the bottom of the upper wall to the point of application.

In this study, the horizontal strength equation of the shown in seismic diagnostic criteria (SDC) for existing RC buildings in the standard of The Japan Building Disaster Prevention Association (MLIT 2017) were applied with necessary modifications.

The horizontal strength ($Q_y$) at the time of bending yield of the brick walls reinforced with RC walls was calculated according to the Eq. (2), based on the SDC. The value was obtained by dividing the moment of bending yield ($M_y$) of the wall by the height, $h$. The moment $M_y$ was obtained by accumulating the moments owing to tensile rebar (1st term), intermediate rebars (2nd term) and axial force, as well as the self-weight of the brick wall (3rd term). Figure 22 shows the position of the symbols used in Eqs. (2), (3), (4), (6) and (7).

$$Q_y = M_y / h$$  

$$M_y = \left[ \sum (u a_i \cdot \sigma_i) \right] \cdot l' + 0.5 \left[ \sum (u a_i \cdot \sigma_{op}) \right] \cdot l' + 0.5(N + w) l'$$  

where,
$u a_i$: The cross-sectional area of the vertical tensile rebars within the range of 110 mm from the end on

Fig. 19 Shear deformation angle - overall deformation angle relationship for specimen CS02.

Fig. 20 Shear deformation angle - overall deformation angle relationship for specimen CD02.

Fig. 21 Position of symbols in Eq. (1).

Fig. 22 Position of symbols in Eqs. (2), (3), (4), (6) and (7).
the tensile side according to the SDC standard,  

\[ \sigma_y, \sigma_{wy} : \text{Standard yield strength (345 N/mm}^2) \text{ of tensile rebars, counted 1.1 times when calculating the ultimate bending moment,} \]

\[ l' : 0.9 \text{ times the length } L \text{ of the upper wall,} \]

\[ a_w, a_{wy} : \text{Cross-sectional area of intermediate rebars, and} \]

\[ \sigma_{sy} : \text{Standard yield strength of intermediate rebars, counted 1.1 times (380 N/mm}^2) \text{ when calculating the ultimate bending moment.} \]

The ultimate bending strength of the brick wall reinforced with RC walls \((w, Q_w)\) was calculated according to Eq. (4) by dividing the ultimate bending moment of the wall by \( h \).

\[ wQ_w = wM_w / h \tag{4} \]

where,

\[ wM_w : \text{Ultimate bending moment of wall, obtained by counting 1.1 times the standard yield strength of intermediate rebars } \sigma_y \text{ and } \sigma_{wy} \text{ in Eq. (3).} \]

The ultimate shear strength \( wQsu \) of the wall was calculated using Eq. (5). As the original equation does not consider openings in the wall, it was slightly modified as presented in Eq. (5). Notably, this equation is only for RC wall retrofit.

\[ wQsu = \begin{cases} \frac{0.053p_w^{0.23}(F_c + 180)M}{(Q/l)} + 0.85v_{wy}\sigma_{wy} + 0.12v_{yw}\sigma_{yw} & \text{be, je} \end{cases} \tag{5} \]

where,

\[ p_w : \text{Equivalent tensile rebar ratio } (=100a_t/(be \cdot l)) \text{ (\%)} \]

\[ a_t : \text{Total cross-sectional area of the main rebar of the tension side,} \]

\[ be : \text{RC wall thickness,} \]

\[ l : \text{Overall length of the wall,} \]

\[ P_w : \text{Equivalent lateral rebar ratio } (=ah/(be \cdot s)) \]

\[ a_s : \text{Cross-sectional area of one rebar,} \]

\[ S : \text{Pitch of horizontal rebars,} \]

\[ F_c : \text{Compressive strength of concrete,} \]

\[ \sigma_{yw} : \text{Yield strength of the lateral rebar,} \]

\[ je : \text{Stress center distance } \text{i.e., } 0.8l , \]

\[ M/(Q/l) : \text{Shear span ratio, where } 1 \geq M/(Q/l) \leq 3, \text{ and} \]

\[ \sigma_{suw} : \text{Axial stress } \sigma_{suw} \leq 8 \text{ N/mm}^2, \text{ which was taken as zero in the current case as there was no axial stress on the RC wall.} \]

Additionally, for comparison, the yield strength obtained by tensile test of the rebars and summarized in Table 2 was used for calculating the ultimate strength according to SDC standard, as shown in Eq. (6).

\[ Q_{suw} = Q_{suw}/h \tag{6} \]

\[ Q_{suw} = \left( \sum_{i=1}^{n} a_{wi} \sigma_{yi} \right) \cdot l' + 0.5 \sum_{i=1}^{n} a_{wi} \cdot v_{iw} \sigma_{wy} / l' \tag{7} \]

where,

\[ m, \sigma_{yi} : \text{Yield strength of tensile rebar (Laboratory test result, Table-2), and} \]

\[ m, \sigma_{wy} : \text{Yield strength of intermediate rebar (Laboratory test result, Table-2).} \]

### 4.2 Considerations for the horizontal strength of wall specimens

Table 5 summarizes the experimental results for wall specimens, while Table 6 summarizes a comparison between the ultimate shear strength and ultimate bending strength. The calculation results indicate that the bending strength was low and, therefore, critical (Table 6). Table 7 summarizes a comparison between the calculated horizontal strength and the experimental value, while Table 8 summarizes the values used to calculate the horizontal strength.

The calculated yield strength at the rotation of the rigid body, which was determined using Eq. (1) was similar to the experimental value. The yield bending strength calculated using Eq. (2) was over the experimental value by 22% to 33%. Likewise, the calculated ultimate bending

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Table 5 Results of loading test of wall specimens.

| Specimen | [1] Horizontal load when the opening wall rotates rigidly (kN) | [2] Horizontal load during the first tensile yield of the main bar of the opening wall (kN) | Deformation angle at the time of the tensile yield (×10² rad.) | [3] Maximum load (kN) | Deformation angle at maximum load (×10² rad.) |
|----------|---------------------------------------------------------------|---------------------------------------------------------------|-------------------------------------------------|-----------------|---------------------------------------------|
| UR01     | Positive 27.5                                                | -                                                             | -                                               | 31.8            | 0.041                                        |
| Negative | -28.2                                                        | -                                                             | -                                               | -29.8           | -0.087                                       |
| CS01     | Positive 157                                                 | -                                                             | 0.23                                            | 196             | 0.714                                        |
| Negative | -160                                                         | -0.57                                                         | -                                               | -193            | -1.007                                       |
| CS02     | Positive 159                                                 | -                                                             | 0.27                                            | 194             | 0.509                                        |
| Negative | -154                                                         | -0.37                                                         | -                                               | -192            | -0.752                                       |
| CD01     | Positive 302                                                 | -                                                             | 0.18                                            | 453             | 0.879                                        |
| Negative | -277                                                         | -0.22                                                         | -                                               | -392            | -1.394                                       |
| CD02     | Positive 280                                                 | 0.13                                                          | 402                                             | 0.770           | -                                            |
| Negative | -288                                                         | -0.13                                                         | -                                               | -400            | -1.519                                       |
strength using Eq. (4) was similar to the experimental value; however the ultimate strength of the single-sided reinforced specimen was over the experimental value by 8 to 11%. Furthermore, the calculated value of the ultimate bending strength using Eq. (6) was similar to the calculated value using Eq. (4), although the ratio of the calculated value to the experimental value was large, because the stress of the rebar was high. Therefore, the authors concluded that the ultimate strength of the double-sided reinforcement could roughly be obtained by Eqs. (4) and (6).

Here, to investigate the difference between the calculated and experimental values, the actual moment during yielding and the ultimate moment due to the tensile rebars and intermediate rebars at the bottom of the upper wall were compared with the corresponding calculated values. The results are summarized in Tables 9, 10 and 11.

In addition, Figs. 23 and 24 show the strain distribution and the axial force distribution at the bottom of the upper wall. For the strain distribution, the value obtained from the strain gauge attached to the rebar at the bottom of the upper wall presented in Fig. 9(b); values for the part where the strain gauge was not attached were obtained via linear interpolation. During yielding, the strain at time [2] in Table 5 (i.e., during the first tensile yield of the main rebar at the bottom of the upper wall) was obtained and the strain at the maximum load [3] in Table 5 was determined for the ultimate. The axial force of the rebar was calculated using the yield strength for all the rebars whose strain exceeded the ultimate yield strain. The experimental moment was calculated as the moment around the center of gravity of the compressive force. The distances between the stress centers listed in Tables 9, 10 and 11 were calculated as the distances at which the sum of the moments of each rebar and the moment of the resultant force were equal and were expressed by \( l' \). For those whose neutral axis did not appear at the end owing to the residual strain, the position of the center of gravity of the compressive force, obtained from the distance between the stress centers assumed in the calculation formula, was used. The first term of the tensile rebar in Eq. (3) tends to underestimate the moment because of the actual tensile rebar during yielding; however, the distance between the stress centers was slightly overestimated (Table 9). Moreover, the stress levels of the rebars were underestimated. For the second term of the intermediate rebar in Eq. (3), the actual distance between the stress centers was larger than that in the calculation formula; however the moment owing to the actual in-

| Specimens | Ultimate shear strength \( \sigma_{Q_u} \) (kN) | \( \sigma_{Q_u}/\text{Exp. value [1] in Table 5} \) | Ultimate bending strength \( \sigma_{wQ_u} \) (kN) | \( \sigma_{wQ_u}/\text{Exp. value [2] in Table 5} \) | \( \sigma_{cQ_u} \) (N/mm²) | \( \sigma_{cQ_u}/\text{Exp. value [3] in Table 5} \) | \( \sigma_{cwQ_u} \) (N/mm²) | \( \sigma_{cwQ_u}/\text{Exp. value [3] in Table 5} \) |
|-----------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|
| CS01      | 254                             | 0.89                            | 212                             | 1.24                            | 1.08                            | 1.14                            |
| CS02      | 288                             | 0.87                            | 212                             | 1.22                            | 1.10                            | 1.16                            |
| CD01      | 508                             | 0.89                            | 402                             | 1.22                            | 0.89                            | 0.94                            |
| CD02      | 576                             | 0.89                            | 402                             | 1.33                            | 1.03                            | 1.08                            |

Note: Exp. value = Experimental value

| Specimens | \( N \) (kN) | \( w \) (kN) | \( L \) (mm) | \( h \) (mm) | \( \sigma_{\text{yy}} \) (N/mm²) | \( \sigma_{\text{yy}} \) (N/mm²) | \( \sigma_{\text{yy}} \) (N/mm²) | \( \sigma_{\text{yy}} \) (N/mm²) |
|-----------|-------------|-------------|-------------|-------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|
| UR01      | 54.0        | 4.7         | 798.0       | 956.0       | 199.0                           | 345.0                           | 403.0                           | 345.0                           | 382.0                           |
| CS01      | 54.0        | 4.7         | 798.0       | 956.0       | 199.0                           | 345.0                           | 403.0                           | 345.0                           | 382.0                           |
| CS02      | 54.0        | 4.7         | 798.0       | 956.0       | 199.0                           | 345.0                           | 403.0                           | 345.0                           | 382.0                           |
| CD01      | 54.0        | 4.7         | 798.0       | 956.0       | 199.0                           | 345.0                           | 401.0                           | 345.0                           | 382.0                           |
| CD02      | 54.0        | 4.7         | 798.0       | 956.0       | 199.0                           | 345.0                           | 401.0                           | 345.0                           | 382.0                           |
19% for the single-sided reinforcement. According to the cause of the intermediate rebar used for double-sided reinforcement, almost all the rebars up to the intermediate rebar section (second term) in Eq. (7). This tendency was similar to that indicated by Eq. (4). For the double-sided reinforcement, almost all the rebars up to the intermediate rebars yield even under the actual stress level of the intermediate rebar. The distance between the stress centers was slightly underestimated; for the intermediate rebars yield; hence, the calculation formula overestimates the stress of the intermediate rebar. Table 11, the ratio of the calculated values to the experimental values increased owing to the higher stress of the rebars in the tensile rebar section (first term) and the intermediate rebar section (second term) in Eq. (7). This tendency was similar to that indicated by Eq. (4). For the double-sided reinforcement, almost all the rebars up to the intermediate rebars yield even under the actual stress and the distance between the stress centers was similar to that in the calculation formula. Therefore, the actual situation could be reflected. However, for the single-sided reinforcement, only a part of the intermediate rebar yields; hence, the formula overestimates the stress of the intermediate rebar.

In conclusion, we established that the calculated value of the bending moment based on the tensile rebar was relatively similar to the bending strength. The effect of the intermediate rebar at the ultimate was greater than its effect on the yield and the formula overestimates the stress level of the intermediate rebar. The distance between the stress centers was slightly underestimated; for

### Table 9 Comparison between the calculated values for the moment of the upper wall during yield in Eq. (3) and the experimental values.

| Specimen | [4] Total moment by tensile rebars (kN-m) | [5] Total moment by intermediate rebars (kN-m) | [4'] Eq. (3) first term (kN-m) | [5'] Eq. (3) second term (kN-m) | Calculated value [4']/Exp. value [4] | Calculated value [5']/Exp. value [5] | Actual tensile rebar stress center-to-center distance (mm) | Actual distance between intermediate rebar stress centers (mm) | Average of the left term |
|----------|------------------------------------------|---------------------------------------------|---------------------------------|---------------------------------|----------------------------------|----------------------------------|---------------------------------|---------------------------------|-----------------------------|
| CS01     | Positive 55.3 45.3 49.2 116 0.89 2.57 0.96 | 0.62 | Negative 52.4 76.3 49.2 116 0.94 1.52 0.91 | 0.60 | 0.61 |
| CS02     | Positive 56.3 77.3 49.2 116 0.87 1.50 0.98 | 0.62 | Negative 55.5 68.0 49.2 116 0.89 1.71 0.97 | 0.62 | 0.63 |
| CD01     | Positive 105 93.3 98.4 232 0.94 2.49 0.96 | 0.59 | Negative 102 101.9 98.4 232 0.97 2.28 0.96 | 0.65 | 0.63 |
| CD02     | Positive 104 134 98.4 232 0.94 1.73 0.97 | 0.64 | Negative 109 120 98.4 232 0.91 1.93 0.96 | 0.62 | 0.62 |

Notes:
Exp. value = Experimental value
Eq. (3), First term: \(\sum (a_i \cdot \sigma_i)\)·
Eq. (3), Second term: \(0.5 \times \sum (a_i \cdot \sigma_i)\)·

### Table 10 Comparison between the calculated values of the moment of the upper wall at the ultimate point in Eq. (4) and the experimental values.

| Specimen | [6] Total moment by intermediate reinforcing rebars (kN-m) | [7] Calculated value [6']/Exp. value [6] | Actual tensile rebar stress center-to-center distance (mm) | Actual distance between intermediate rebar stress centers (mm) | Avg. of the left term |
|----------|----------------------------------------------------------|----------------------------------------|--------------------------------|---------------------------------|-----------------------------|
| CS01     | Positive 55.9 113 54.1 128 0.97 1.13 0.97 | 0.57 | Negative 57.4 111 54.1 128 0.94 1.15 0.96 | 0.69 | 0.61 |
| CS02     | Positive 56.4 114 54.1 128 0.96 1.12 0.98 | 0.60 | Negative 55.2 108 54.1 128 0.98 1.19 0.96 | 0.59 | 0.60 |
| CD01     | Positive 113 266 108 255 0.96 0.96 0.98 | 0.53 | Negative 115 282 108 255 0.94 0.91 1.00 | 0.52 | 0.55 |
| CD02     | Positive 114 257 108 255 0.95 0.99 1.00 | 0.60 | Negative 114 275 108 255 0.95 0.93 1.00 | 0.54 | 0.54 |

Notes:
Exp. value = Experimental value
Avg. = Average

*Since the position of the neutral axis is unknown, the position of the center of gravity of the compressive force is used instead.

1\(^{1}\) First term \(wMu: \sum (a_i \cdot \sigma_i)\)· [Refer to Eq. (4)].
2\(^{2}\) Second term \(wMu: 0.5 \times \sum (a_i \cdot \sigma_i)\)· [Refer Eq. (4)].
double-sided and single-sided reinforcements, the effects of the intermediate rebars at the ultimate differed slightly.

4.3 Proposed formula for calculating the horizontal strength of wall specimens

Based on the actual situation, following points were summarized:

(i) The calculation formula for tensile rebars was relatively consistent with the actual situation. Using the yield strength affords similar results.

(ii) The effect of the intermediate rebar was greater at the ultimate than at the yield.

(iii) The existing calculation formula overestimates the stress of an intermediate rebar.

(iv) The existing calculation formula slightly underestimates the distance between the stress centers of the intermediate rebars.

(v) For double-sided and single-sided reinforcements, the influences of the intermediate rebar at the ultimate end differ slightly.

Based on the above calculations, the authors propose a calculation formula that can reflect the actual conditions. The horizontal bearing force \( Q_{yx} \) during the bending yield of the brick wall reinforced with the proposed RC wall was calculated according to Eq. (8) by using Eq. (9) for the yield bending moment of the bearing wall.

\[
Q_{yx} = \frac{M_y}{h}
\]

\[
M_y = \sum (a_i \cdot \sigma_i) \cdot l' + 0.6 \left( \sum (a_i \cdot \alpha_i \cdot \sigma_{iy}) \right) \cdot l' + (N + w) \cdot L / 2
\]

where,

\( \alpha_i \): Reduction coefficient of the yield strength of the intermediate rebar in the \( i \)-th stage from the tension side when calculating the yield bending moment (based on the strain distribution, this is equal to 0.4 for the single-sided reinforcement and 0.3 for the double-sided

Fig. 23 Conceptual diagram of strain distribution at the lower part of the upper wall.
of intermediate rebars. The distance between the stress centers of intermediate rebar was considered as 0.6 \( l' \) instead of 0.5 \( l' \).

As the formula for the bending strength of the wall in the SDC involves obtaining the ultimate strength, it is possible that the stress center distance of 0.5, used in the formula, is determined based on the stress distribution at the ultimate state, where the intermediate rebars also yield. Therefore, we considered adopting a larger value when the rebars at the edge of the wall yield; thus, 0.6 was used for the distance in this study.

In addition, during the ultimate bending of the brick wall reinforced with the proposed RC wall was calculated according to Eq. (10) by using Eq. (11) for the ultimate bending moment of the bearing wall.

\[
\text{mean} \, Q_m = \frac{\text{mean} \, M_n}{h} \tag{10}
\]

\[
xw \, M_w = \left\{ \sum_{w} \left( a_w \cdot \sigma_w \right) \right\} \cdot l' + 0.5 \left\{ \sum_{w} \left( a_w \cdot \beta_w \cdot \sigma_w \right) \right\} \cdot l' + (N + w) \cdot L / 2 \tag{11}
\]

where,

\( \beta_i \): Reduction coefficient of the yield strength of the intermediate rebar in the \( i \)-th stage from the tension side when calculating the ultimate bending moment, as shown in Fig. 25. \( \beta \) is 0.7 for the single-sided reinforcement and 0.9 for double-sided reinforcement, based on the actual strain distribution. In the equation, the distance between the stress centers of the intermediate rebars (0.5 \( l' \) ) remained constant.

**Figure 25** shows the graph for obtaining the reduction coefficients \( \alpha \) and \( \beta \), where \( \alpha \) denotes the yield and \( \beta \) denotes the ultimate value. **Figure 25** shows the amount of tensile stress exerted by the intermediate rebar; in the figure, the vertical axis indicates that it has yielded and the horizontal axis represents the location of the rebar. Zero on the left end denotes the position of the tensile rebar at the end of the upper wall. The value of \( \beta \) depends on the RC reinforcement method, i.e., whether single-sided or double-sided reinforcement is employed. When determining the reduction coefficients \( \alpha \) and \( \beta \), the average value of the ratio of the actual stress to the yield strength of the intermediate rebar of each specimen was considered; the average value was employed to determine the reference value, as shown in **Fig. 25**, which shows the actual strain distribution and the strength expression rate of the intermediate rebars.

The coefficient of the stress center distance of the intermediate rebar was determined using the average value

![Fig. 24 Conceptual diagram of axial force distribution at the lower part of the upper wall.](image)

![Fig. 25 Graph for calculating reduction coefficients (\( \alpha \) and \( \beta \)).](image)
of the actual stress center distances listed in Tables 9 and 10 as a reference. Table 12 summarizes the comparison between the calculated values obtained using the proposed formula and the experimental values. Although the proposed formula underestimated the yield strength of the double-sided reinforcement specimen, it afforded values similar to the experimental values.

5. Conclusions

In this study, the authors carried out a loading experiment to investigate the reinforcing effect on URM walls. In addition, the authors derived and verified a horizontal strength evaluation formula of a brick wall specimen reinforced with RC wall. The following results and findings were obtained:

1) The results of the horizontal loading test indicated that the maximum load of specimens reinforced with RC walls from one side was 6.1 to 6.2 times higher than that of unreinforced specimens. Similarly, the maximum loads of the specimens reinforced with RC walls from both sides were 12.6 to 14.2 times higher than that of the unreinforced specimen.

2) The maximum loads of the two single-sided reinforced specimens were similar; however, CS02 exhibited a maximum load with a smaller deformation angle than CS01 owing to the effect of twisting.

3) The maximum horizontal strength of specimen CD02 was approximately 11% lower than that of specimen CD01. However, the strength was maintained without degradation to a large deformation level.

4) The term of the tensile rebar in the calculation formula based on the SDC standard was relatively consistent with the actual situation. Using the actual yield strength of the steel bars gave closer values to test results.

5) The effect of the intermediate rebar was more significant at the ultimate than at the yield.

6) The calculation formula based on the SDC standard overestimated the stress of the intermediate rebar and slightly underestimated the distance between the stress centers.

7) The strains generated in the intermediate rebars at the ultimate level were larger when both sides were reinforced than those of the intermediate rebars when one side was reinforced.

8) The values calculated using the proposed formula based on the actual strain distribution were highly similar to the experimental values.

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