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Abstract

This paper describes the effects of nonstructural brick walls on the seismic performance of reinforced concrete (R/C) buildings. Experimental and analytical studies were conducted on two buildings: one of which collapsed and the other suffered moderate damage due to the 2007 Sumatra, Indonesia earthquakes. A brick wall was extracted from the moderately damaged building and transported to Japan from Indonesia to experimentally evaluate its seismic performance. Two R/C one-bay frame specimens were constructed, and the imported wall was installed in one of the specimens. Comparing the seismic performance of specimens with and without the brick wall through quasi-static cyclic loading tests, wall contributions were quantitatively evaluated. Moreover, the seismic performance of the earthquake-damaged Indonesian buildings was evaluated numerically considering the findings of the tests. The analyses revealed a possible reason for the collapse of one of the buildings due to the earthquakes.

Keywords: masonry infill; numerical analysis; reinforced concrete; seismic loading test; 2007 Sumatra, Indonesia earthquakes

1. Introduction

West Sumatra, Indonesia is located close to a major earthquake fault line where destructive earthquakes have occurred in recent years. Padang city, which is the capital of West Sumatra province, suffered moderate/serious damage due to the 2007¹⁾/2009²⁾ earthquakes. In particular, a large number of reinforced concrete (R/C) buildings totally collapsed in the latest case. The authors conducted a post-earthquake field investigation in Padang city after the 2007 event to collect data on typical structural details and damage¹⁾. The investigation focused on two 3-story R/C buildings, standing side by side: one totally collapsed and the other was moderately damaged, as shown in Photo 1. Although the buildings were structurally similar, one of them suffered severe damage. In fact, the authors' previous analytical study¹⁾ indicated that the base shear coefficients of both R/C moment-resisting frames were about 0.17 in the E-W direction, to which one of the buildings toppled, when assuming the story collapse mechanism. However, the amount of nonstructural brick walls was significantly larger in the surviving building.

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Therefore, a series of structural tests was conducted to investigate the effects of nonstructural brick walls, which are not considered in the seismic design, on the actual performance of damaged buildings. A brick wall was extracted from the moderately damaged building and transported to Toyohashi University of Technology, Japan from Indonesia. After installing the imported wall in a R/C one-bay frame specimen, its performance was evaluated through laboratory tests. Moreover, based on the test results, numerical analyses of both buildings were carried out to investigate the reason why one of the buildings collapsed due to the earthquakes.



Photo 1. Collapsed and Moderately Damaged Buildings Due to the 2007 Sumatra, Indonesia Earthquakes (Source: Padang Ekspres Newspaper)

2. Collapsed and Moderately Damaged Buildings

The collapsed building was a 3-story R/C building used as a car showroom, which was constructed in 1980. Photo 2 shows the building before its collapse. The ground plan and the first story column details are also shown in Fig.1. Nonstructural brick walls were used for exterior and partition walls. The building fell to the west during the earthquake, as shown in Photo 1.



Photo 2. Collapsed Building before the 2007 Earthquakes

The surviving building was structurally similar to the collapsed one, which was also a 3-story R/C building with nonstructural brick walls. Photo 3 and Fig.2. show the building after the earthquake and the ground plan with column details, respectively. Damage to columns was investigated in detail by the authors to categorize the damage grade. As a result, the grade was classified as "moderate" for the first story as reported in the authors' previous paper¹⁾.



Photo 3. Surviving Building after the 2007 Earthquakes

3. Test Specimens

3.1 Preparation of Brick Wall

To experimentally clarify the structural contributions of nonstructural brick walls to the seismic performance of R/C buildings, a brick wall was extracted as a test specimen from the surviving building, as shown in Photo 4(a). The size of the brick wall was about 1500 x 1500 mm. Then, it was transported to Toyohashi University of Technology, as shown in Photo 4(b).

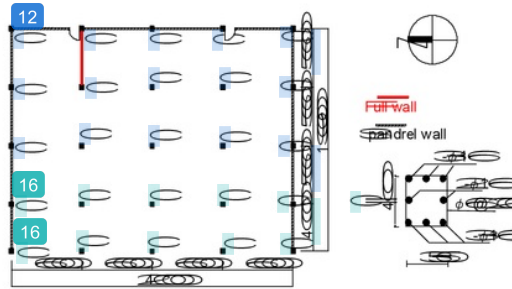


Fig.1. Ground Plan with the First Story Column Details of the Collapsed Building

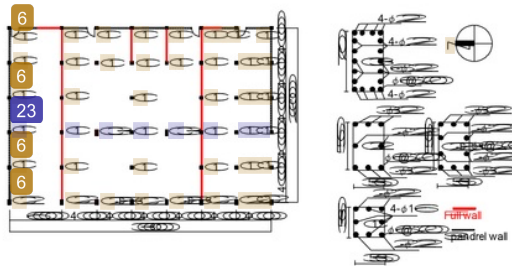


Fig.2. Ground Plan with the First Story Column Details of the Surviving Building



(a) Extracting Wall. (b) Transporting Wall.
Photo 4. Preparation of Brick Wall Specimen

3.2 R/C Frame

Two 1/2.5 scale R/C one-bay frame specimens were prepared: one bare frame (BF) and the other with the imported brick wall (IF), as described in the following.

The R/C frames represent the first story of the surviving building shown in Photo 3 and Fig.2. The cross-sectional dimensions of the columns were 140 x 140 mm, with 4-φ9 longitudinal rebars and 2-φ4@100 transverse hoops. The clear height of columns was 1000 mm. Figure 3 shows the configuration and bar arrangements of the BF specimen.

3.3 Installation of Brick Wall

The imported brick wall was installed in one of the R/C frame specimens, as shown in Photo 5(a), after it was cut to dimensions of 1420 mm in width x 960 mm in height, as shown in Photo 5(b). In this specimen, however, the wall thickness of 140 mm was not reduced because of technical difficulties related to scale reduction. Mortar was produced with a volume ratio of cement : sand : water = 1 : 4 : 0.5, and was applied between the main frame and inserted wall as

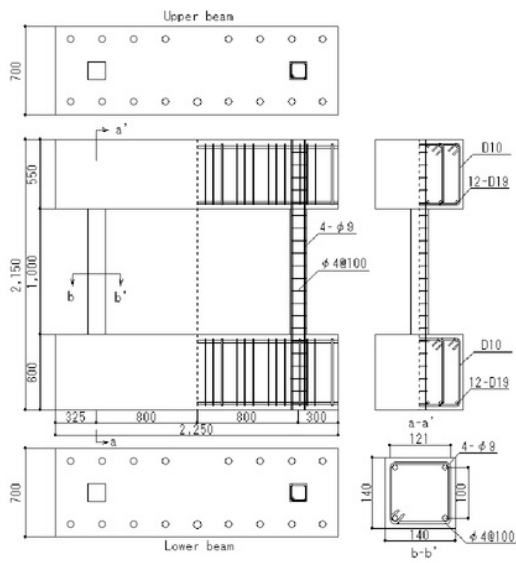


Fig.3. Detailed Drawing of BF Specimen

a joint material. The thickness of the mortar joint was 20 mm, as shown in Fig.4. Initial slight damage was observed in the infilled wall, as shown in Photo 6.

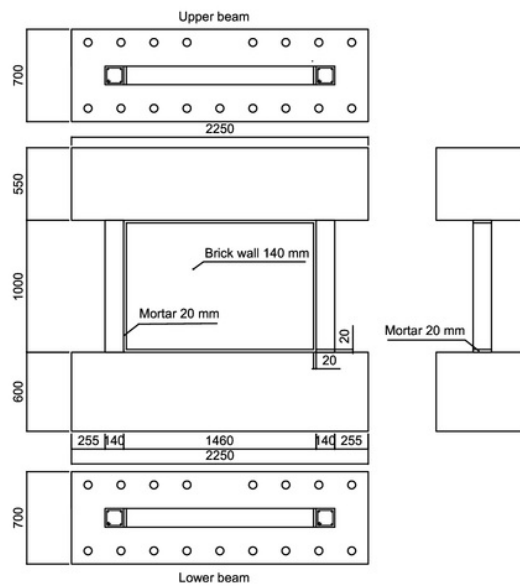


Fig.4. Detailed Drawing of IF Specimen



(a) Installing Wall. (b) Cutting off Wall.
Photo 5. Installation of Brick Wall



Note: Dashed lines on the wall surface represent initial damage.

Photo 6. Initial Damage to Infilled Wall

3.4 Material Properties and Column Performance

The mechanical properties of concrete, mortar, and reinforcements used for the specimens are shown in Table 1.

The columns of both specimens were estimated to yield in flexure prior to shear failure. Column performance was evaluated based on the Japanese standard³⁾ (refer to Appendix A), which is summarized in Table 2.

Table 1. Material Properties

Concrete			
Specimen	Material age Day	Compressive strength N/mm ²	Tensile strength N/mm ²
BF	44	19.6	1.89
IF	37	20.6	1.96
Mortar			
Specimen	Material age Day	Compressive strength N/mm ²	Tensile strength N/mm ²
IF	42	10.0	3.33
Reinforcing bar			
Bar number	Yield strength N/mm ²	Tensile strength N/mm ²	Young's modulus N/mm ²
φ 9	355	440	2.02 × 10 ⁵
φ 4	583	631	2.14 × 10 ⁵

Table 2. Column Performance

Specimen	Flexural strength	Shear strength
	kN	kN
BF	39.8	59.5
IF	40.1	59.9

4. Experimental Methods

4.1 Loading Method

The specimens were tested at the testing facility of the Toyohashi University of Technology. A schematic representation of the experimental set-up is shown in Fig.5. One horizontal hydraulic jack (2000 kN) and two vertical ones were equipped for the loading system.

The specimens were subjected to a constant vertical load of 183.4 kN ($\approx 0.24 \times$ column sectional area \times compressive strength of concrete) based on

the calculated weight of the surviving building. Then, reversed cyclic lateral loads were applied to the specimens. Drift angle R (rad.), ratio of lateral displacement to column height, was used to control incremental loading. Lateral loading program was initial cycle to $R = 1/800$ followed by two cycles to $R = 1/400, 1/200, 1/100, 1/50, 1/25,$ and $1/12.5$. When the specimens failed, however, loading was stopped. Figure 6 shows the lateral loading history. The shear span to depth ratio ($= h_w/l_w$ in Fig. 5.) of the specimens was maintained at 0.75 throughout the tests so that lateral loads were applied at an assumed second floor height of 1200 mm.

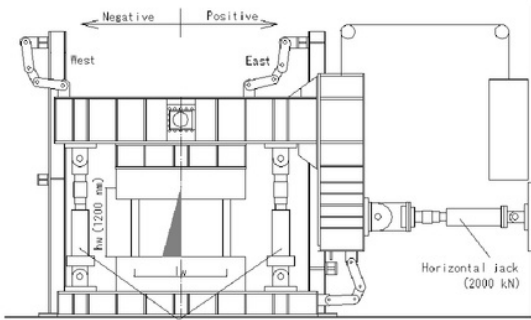


Fig. 5. Schematic View of Test Set-up

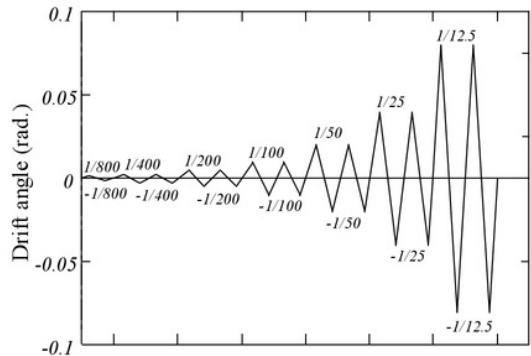


Fig. 6. Lateral Loading History

4.2 Measurement

The horizontal, vertical, and diagonal relative displacements of the specimens were measured with transducers, as illustrated in Fig. 7.(a). Strains of longitudinal rebars and hoops were also measured using strain gauges, as illustrated in Fig. 7.(b). Initiated cracks, crack propagation, and major crack widths were observed at every peak and residual drift, to identify the failure mechanism of specimens.

5. Experimental Results

5.1 Failure Process and Mechanism

Table 3. lists the failure processes of BF and IF specimens. In the case of BF, plastic hinges were

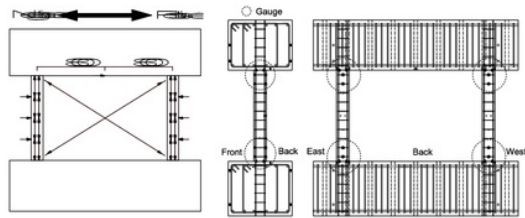


Fig. 7. Measurement

formed at the top and bottom ends of both columns around a 1.0% drift ratio. Although high ductility performance was exhibited until the loading cycles reached $R = 1/25$, the specimen finally lost its axial resistance during the cycle to $R = 1/12.5$. On the other hand, the IF exhibited a different failure mechanism, with shear failures of columns and infill. This specimen failed axially during the first cycle to $R = 1/25$. Fig. 8. compares the final crack patterns of both specimens observed after the tests.

5.2 Lateral Force-Drift Ratio Relationship

Fig. 9. compares the lateral force vs. drift ratio, R relationships between the specimens. The maximum lateral strengths of 36.8 kN and 174.0 kN were observed at 2.0% and 0.5% drift ratios for BF and IF, respectively. The deformation capacities, which were defined as a deformation where post-peak strength dropped to 80% of the peak strength, were 2.8% and 1.6% for BF and IF, respectively.

After installing the nonstructural brick infill, strength increased to 4.7 times, but ductility decreased to about half.

5.3 Effects of Nonstructural Brick Wall

The brick wall installed in a R/C frame significantly increased the lateral strength of the overall frame. The incremental strength due to the wall was briefly evaluated at the maximum strength in Fig. 9.(b) by Eq. (1). However, this equation considered only the difference between overall strengths neglecting interactions between the columns and wall. As a result, the averaged shear strength of the brick wall was 0.73 N/mm^2 , which was applied to an analytical study in the following.

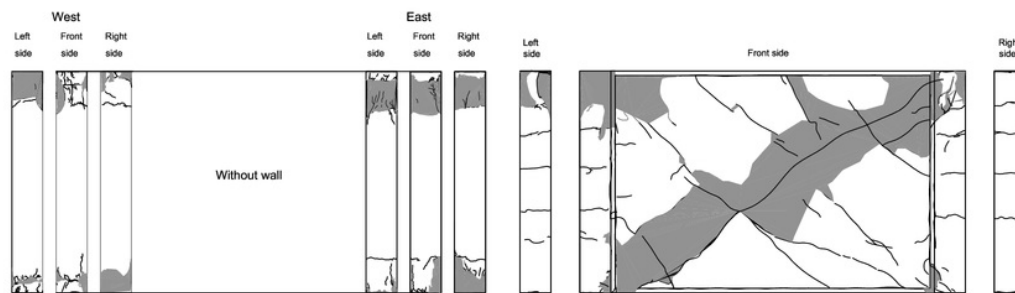
$$\tau = \frac{Q_{IF} - Q_{BF}}{A_w} \quad (1)$$

where, τ : averaged shear strength of brick wall, Q_{IF} : maximum lateral strength of the IF specimen, Q_{BF} : lateral force of BF specimen at the drift where Q_{IF} was recorded, A_w : cross-sectional area of brick wall.

On the other hand, the ductility of the R/C frame increased upon installing the brick wall. This was because an inclined compression strut was formed in the panel when it was subjected to shear deformation by the surrounding frame, as shown in Fig. 10. As a consequence, a high punching shear acted on the

Table 3. Failure Processes

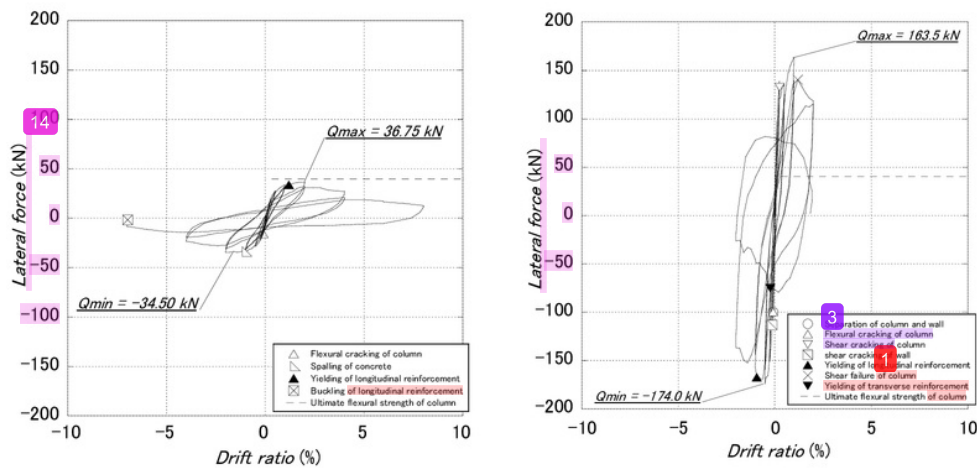
Cycle (rad.)	BF specimen		IF specimen	
	Columns		Columns	Brick wall
Initial crack	None.		None.	As shown in Photo 6.
1/800	Initial flexural crack at the top of the compressive column.		Initial flexural crack at the top of the tensile column.	Separation cracks around the wall. Initial shear crack.
1/400	Flexural cracks at the top and bottom of both columns.		Flexural cracks at the top and middle of the tensile column.	Shear crack development.
1/200	Crack propagation in both columns.		Shear cracks at the top of the tensile column. Flexural cracks at the bottom of the compressive column.	Shear crack propagation.
1/100	Initial crushing of concrete at the bottom of the compressive column.		Shear cracks at the bottom of the compressive column. Initial yielding of longitudinal rebar.	Peeling off of plaster.
1/50	Concrete crush at the top and bottom of the compressive column. Initial yielding of longitudinal rebar.		Shear failure at the top of the tensile column. Buckling of longitudinal rebars. Spalling of cover concrete at the bottom of the compressive column. Initial yielding of hoop. Degradation of lateral strength.	Spalling of plaster.
1/25	Spalling of cover concrete. Degradation of lateral strength.		Loss of axial resistance.	Remarkable damage.
1/12.5	Buckling of longitudinal rebars in the tensile column. Loss of axial resistance.			



(a) BF Specimen (R = 1/12.5 rad.)

(b) IF Specimen (R = 1/25 rad.)

Fig.8. Final Crack Patterns



(a) BF Specimen.

(b) IF Specimen.

Fig.9. Lateral Force-Drift Ratio Relationships

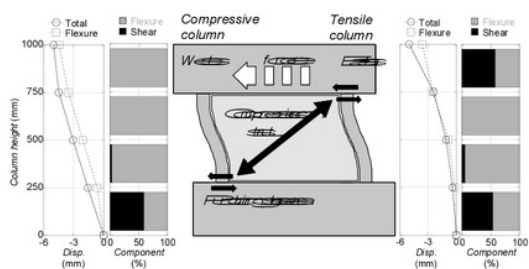


Fig. 10. Punching Shear Action on Columns

bottom/top of compressive/tensile column. In this figure, the distributions of lateral displacements and deformation components along the column height were also shown at the maximum strength. It was found that the shear deformation was clearly dominant at the bottom/top of compressive/tensile column.

6. Seismic Performance Evaluation of Earthquake-Damaged Buildings

6.1 Analytical Methods

Pushover analyses were conducted focusing on the seismic performance of collapsed and moderately damaged buildings, in particular the performance in E-W direction where more severe damage was observed in the authors' post-earthquake investigation¹⁾. Two 3-dimensional frame analyses were performed for each building—one with and one without infill walls, to clarify the contributions of nonstructural brick infills to the seismic performance of two earthquake-damaged buildings.

The structural details are summarized in Fig. 11. and Table 4. for the collapsed building and in Fig. 12. and Table 5. for the surviving one, respectively. As shown in Table 4., however, member details of the collapsed building excluding the first story column could not be collected before the building was demolished. Therefore, column details in the second and third stories were assumed to be identical to those in the first story, and beam details were referred to a typical beam in the other building (G1 in Table 5.).

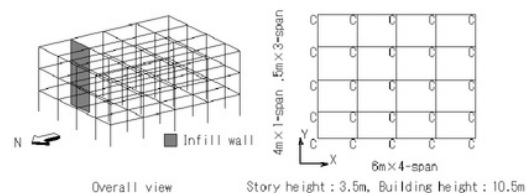


Fig. 11. Structural Brief of the Collapsed Building

Table 4. Column List of the Collapsed Building

Story	Column	C
1	B×D	350×400
	Main rebar	8-φ 16
	Hoop	2-φ 6@200

Unit: mm

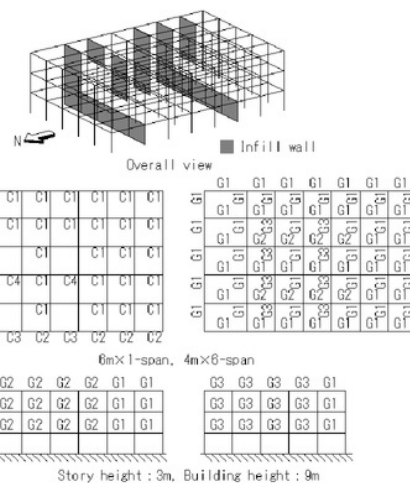


Fig. 12. Structural Brief of the Surviving Building

Table 5. Member List of the Surviving Building

Story	Column	C1	C2	C3	C4
1	B×D	350×450	350×550	350×700	
	Main rebar	4-φ 22 4-φ 16	8-φ 22	10-φ 22	18-φ 22
	Hoop		2-φ 6@200		
2	B×D	350×350	350×550	350×700	
	Main rebar	4-φ 22 4-φ 16		4-φ 22 6-φ 16	4-φ 22 14-φ 16
	Hoop		2-φ 6@200		
3	B×D	350×350	350×550	350×700	
	Main rebar		8-φ 16		18-φ 16
	Hoop		2-φ 6@200		
Floor	Beam	G1	G2	G3	
2	B×D	350×550	250×420	350×720	
	Main rebar	4-φ 16 4-φ 12	10-φ 16 2-φ 12	10-φ 12	
	Stirrup	2-φ 6@100 (Middle: 2-φ 6@150)			
3	B×D	350×450	250×420	300×600	
	Main rebar	4-φ 16 4-φ 12	10-φ 16 2-φ 12	6-φ 22	
	Stirrup	2-φ 6@100 (Middle: 2-φ 6@150)			
R	B×D	300×450	250×420	300×550	
	Main rebar	4-φ 16 4-φ 12	4-φ 22 4-φ 12	4-φ 22 4-φ 12	
	Stirrup	2-φ 6@100 (Middle: 2-φ 6@150)			

Unit: mm

The following numerical models were applied to the analyzed buildings:

1. Each column was replaced by a line element representing nonlinear (trilinear) flexural behavior and linear shear and axial behavior. However, flexural strengths were evaluated considering shear strengths: in the case of $Q_{su} < Q_{mu}$ from Appendix A, Q_{mu} was replaced by Q_{su} .
2. Each beam was replaced by a line element representing linear flexural, shear, and axial behavior.
3. Beam-column joints were assumed to be rigid.
4. Spandrel walls were considered as rigid zones for columns.

5. Fixed base and rigid floors were assumed.

Moreover, in the cases of analyses considering infill walls, their behavior was represented by bilinear elements with lateral strengths equivalent to the experimental result, as shown in Fig.13. Lateral strength of each infill was evaluated by multiplying its cross-sectional area by the averaged shear strength of 0.73 N/mm².

Incremental lateral loads were applied in the western direction (refer to Figs.11. and 12.), to which the collapsed building had toppled as shown in Photo 1. Table 6. gives the applied distributions of lateral loads along the building height, which were calculated on the basis of the Japanese building design code⁴⁾.

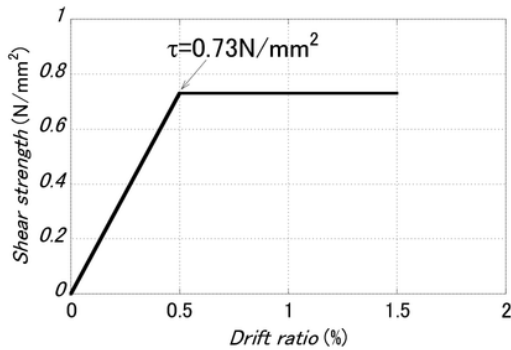


Fig.13. Modeling of Infill Performance

Table 6. Distributions of Lateral Loads along the Building Height

Story	Collapsed building	Surviving building
3	1.588	1.534
2	1.236	1.214
1	1.000	1.000

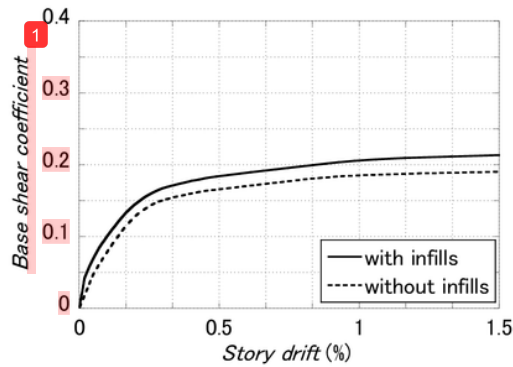
6.2 Analytical Results

Fig.14. compares the base shear coefficient vs. the first story drift relationships of both buildings with and without infill walls. The difference between the lateral strengths of the buildings was less than 20% when neglecting infills. With infills, however, the strength of the surviving building exceeded that of the collapsed one by more than 50% at a 1.5% drift level approximately equivalent to the deformation capacity of the IF specimen (Refer to Section 5.2). This result means that nonstructural brick infill walls should be considered in a seismic performance evaluation of this kind of structure. In particular, the case of the surviving building, they seemed to contribute significantly to reducing structural damage under the earthquakes, which is a possible reason for the difference of damage grade observed between two similar buildings.

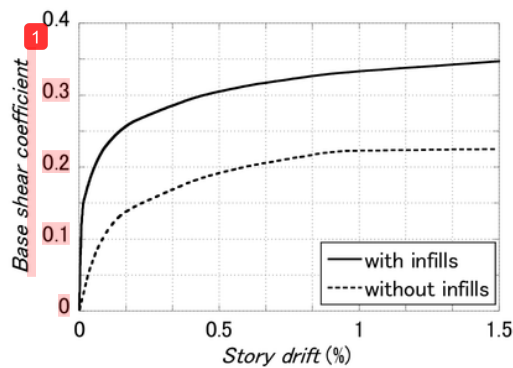
7. Conclusions

Structural contributions of nonstructural brick walls to the seismic performance of R/C buildings were investigated based on two Indonesian earthquake-damaged buildings: one of which collapsed and the other suffered moderate damage. Two R/C one-bay frame specimens were constructed representing the first story of the surviving building. Moreover, a brick wall was extracted from the building, transported to Japan from Indonesia, and installed in one of the specimens. These specimens were prepared to experimentally clarify the effects of brick infills on the seismic performance of the earthquake-damaged buildings. Major findings from experimental and analytical investigations are summarized as follows.

1. Seismic loading tests on the specimens were carried out to quantitatively obtain the structural contributions of the brick infill to the seismic performance of the R/C frame. As a result, brick infill was found to significantly increase the strength of the overall frame by the compression strut mechanism. The averaged shear strength was 0.73 N/mm² in this case.



(a) Collapsed Building.



(b) Surviving Building.

Fig.14. Comparison of Performance Curves

2. Pushover analyses were also conducted for both of the earthquake-damaged buildings based on the test results. It was found that nonstructural brick infills significantly increased the strength of the surviving building, which was more than 1.5 times that of the collapsed one at the ultimate state. This is a possible reason why different damage grades were observed between two similar buildings.

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Acknowledgments

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Appendix A

The performance of the R/C columns was evaluated on the basis of Reference 3) in this study. Shear force at flexural strength and shear strength were calculated by Eqs. (2) and (3), respectively.

$$Q_{mu} = \frac{2M_u}{h_0} \quad (2)$$

$$M_u = 0.8 \cdot a_t \cdot \sigma_y \cdot D + 0.5 \cdot N \cdot D \left(1 - \frac{N}{b \cdot D \cdot F_c} \right)$$

$$Q_{su} = \left[\frac{0.053 \cdot p_t^{0.25} \cdot (18 + F_c)}{M/Q \cdot d + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy} + 0.1 \cdot \sigma_o} \right] \cdot b \cdot j \quad (3)$$

$$p_t = \frac{a_t}{b \cdot D}, \quad p_w = \frac{a_w}{b \cdot s}, \quad \sigma_o = \frac{N}{b \cdot D}$$

where, Q_{mu} : shear force at flexural strength, Q_{su} : shear strength of column, M_u : flexural strength of column, h_0 : clear height of column, a_t : total cross-sectional area of tensile reinforcing bars, σ_y : yield stress of longitudinal reinforcement, D : column depth, N : axial force, b : column width, F_c : compressive strength of concrete, p_t : tensile reinforcement ratio, M/Q : shear span length (default value is $h_0/2$), d : effective depth of column, p_w : shear reinforcement ratio, σ_{wy} : yield stress of shear reinforcement, σ_o : axial stress in column, j : distance between tensile and compression forces (default value is $0.8D$), a_w : cross-sectional area of shear reinforcing bars, s : spacing of hoops.

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