Seismic capacity and retrofit of existing brick masonry building

Seismic Capacity and Retrofit of Existing Brick Masonry Building

Résistance sismique et réparation d'un immeuble en maçonnerie

Sismische Tragfähigkeit und Nachrüstung eines bestehenden Ziegelmauerwerkgebäudes

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SUMMARY

This paper describes an experimental study on seismic capacity and retrofit of an existing masonry building constructed in 1914. The structural system of this building consists of brick walls, which contain steel elements inside the walls. Brick masonry wall models with or without steel elements are tested for evaluation of seismic capacity of this building. Models retrofitted with reinforced concrete or steel walls are tested. Reinforcing effects of steel elements and retrofit performance are discussed.

RÉSUMÉ

Le rapport porte sur l'étude expérimentale de la résistance sismique et de la réparation d'un immeuble en maçonnerie construit en 1914, dont le système structural consiste en murs de briques contenant des éléments métalliques. Des murs en maçonnerie en briques, avec et sans éléments métalliques, ont été testés en vue de l'évaluation de la résistance sismique du bâtiment. Des prototypes renforcés avec du béton armé ou des parois métalliques ont également été testés. Les effets du renforcement par des éléments métalliques ainsi que l'efficacité de la réparation sont discutés.

ZUSAMMENFASSUNG

In diesem Aufsatz wird eine experimentelle Studie über die seismische Widerstandsfähigkeit und Nachrüstung eines bestehenden, im Jahre 1914 gebauten Gebäudes aus Ziegelsteinmauerwerk beschrieben. Das Tragsystem dieses Gebäudes besteht aus Mauerwerkswänden mit eingelegten Stahlteilen. Wandmodelle mit und ohne Stahlelemente wurden daher geprüft, um die seismische Widerstandsfähigkeit des Gebäudes zu ermitteln. Ausserdem wurden auch mit Stahlbeton- bzw. Stahlwänden nachgerüstete Modelle untersucht. Die verstärkende Wirkung der Stahlelemente und die Wirksamkeit der Nachrüstung werden erörtert.

1. Introduction

Tokyo Station, located near the Imperial Palace, is the central station of Japan. The building of Marunouchi side of this station was constructed in 1914. Japanese people love this historical and Western-styled building because the building symbolizes rapid modernization of Meiji Era. The building is a brick masonry and steel structure, which is 400m long and 2 storied. In the original figure, it was 3 storied, however the top floor was demolished because of heavy damage during World War 2.

Recently, its owner, East Japan Railway Company is planning a redevelopment project of Marunouchi area, including renewal of this station. Considering symbolic existence of this building in Japan, it is strongly hoped to reserve the building in some ways. Therefore, it is needed to investigate the structural performance of this building, especially seismic capacity, and if necessary, to develop retrofitting techniques. For these purposes, the authors carried out the following tests and investigation.

(1) diagonal shear loading test of brick masonry walls;
contribution of steel elements to behavior of walls was investigated.
(2) direct shear loading test of mortar bed joints of the masonry;
influence of normal stress on the shear strength of masonry walls was estimated.
(3) diagonal shear loading test of retrofitted brick masonry walls;
retrofitting techniques for brick masonry walls were discussed.
(4) Proposal of a simplified estimation method for reinforced brick masonry walls:
contribution of various reinforcing elements to shear strength of the wall was determined.

2. Test of brick masonry walls

The structural system of this building consists of the next three components; (a) brick masonry walls, (b) steel frames or elements encased inside the brick masonry walls, and (c) floor slab diaphragms supported by the steel frames. Typical detail of the frame is shown in Fig. 1. The walls reinforced with the encased steel elements is considered to resist earthquake load, therefore testing was carried out to evaluate the seismic performance of this structural wall.

Test specimens were five brick masonry walls and were subjected to diagonal compression shear loading. One specimen, named BW0, was cut off a structural brick masonry wall in this existing building. The other four specimens were newly constructed to investigate reinforcing effects of the steel structural elements which were encased inside the brick walls. The list of the specimens is shown in Table 1. The major test variable was presence of steel structural elements.

Specimens BW0, BW1, and BW2 were pure brick masonry walls. Specimen BWS had steel reinforcing elements inside and outside the brick wall, which were corresponding to web reinforcing bars and main bars of usual reinforced concrete walls, respectively. The dimensions and detailing of the steel structural elements were determined under consideration of correspondence to original ones. Specimen BWSC was provided larger steel columns than BWS in order to represent confinement of adja-

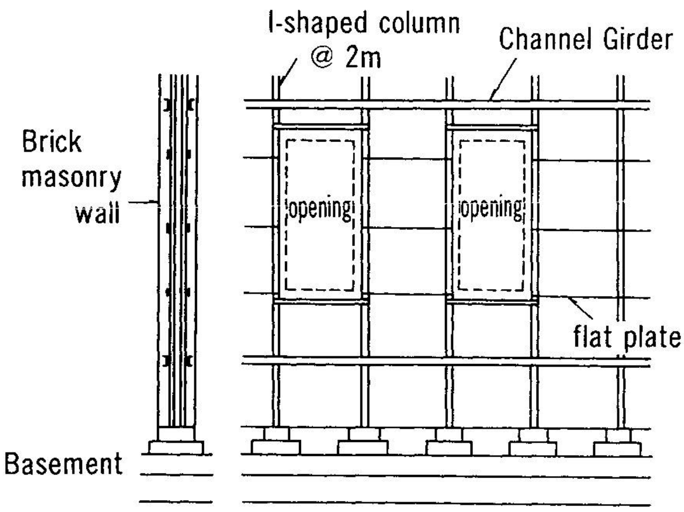


Fig. 1 Typical Steel Frame

Table 1 List of specimens

Specimen	Component
BW0	Brick masonry wall (existing)
BW1	Brick masonry wall (new)
BW2	Brick masonry wall (new)
BWS	Brick masonry wall (new) Steel columns and tie bars
BWSC	Brick masonry wall (new) Steel strong columns and tie bars

Table 2 Material properties of wall specimens

a) Bricks and brick masonry	E	σb
Brick unit (new)	30.0	113.8
Brick unit (existing)	6.0	30.4
Bed joint mortar	3.9	3.77
Brick masonry pile (new)	8.0	30.0
Brick masonry pile (existing)	3.0	14.0

E : Modulus of elstiticity (GPa)
σb: Compressive strength (MPa)

b) Steel elements	E	σy	σt
Tie bar FB-32×2	208	709*	772
Column H-100×50	207	265	420
Column H-100×100	209	327	468

E : Modulus of elsticity (GPa)
σy: Yield strength (MPa) *0.2% off set
σt: Tensile strength (MPa)

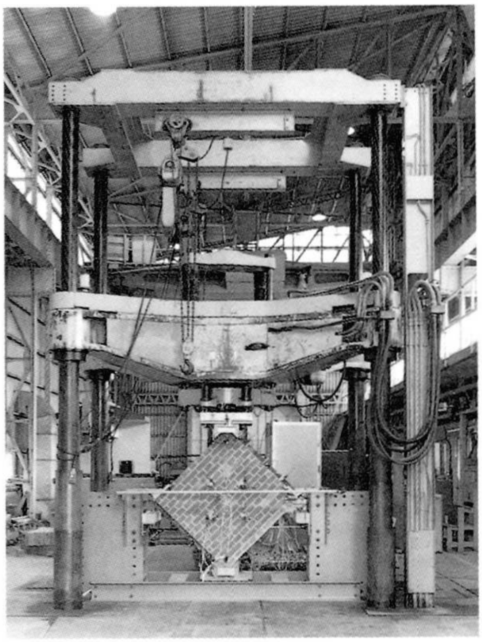


Photo 1 Loading Apparatus

cent walls in a multi-bay wall. Dimensions of the specimens are shown in Fig. 2. Material properties are shown in Table 2.

Static and monotonic diagonal compression load was applied. The shear strain of the walls was calculated from the measured displacements of the two diagonals. The shear force was also calculated from the applied load. The test set-up employed is shown in Photo 1.

Shear force - shear strain relationships of the specimens are shown in Fig. 3. Representative crack pattern after the testing is shown in Fig. 4. The new pure brick masonry walls (Specimens BW1 and BW2) showed very brittle failure. When a diagonal crack appeared in the wall, load was completely lost simultaneously. Most of cracks were observed along joints of the brick masonry. The old pure brick masonry wall (Specimen BW0) showed more ductile manner because the measurement point of

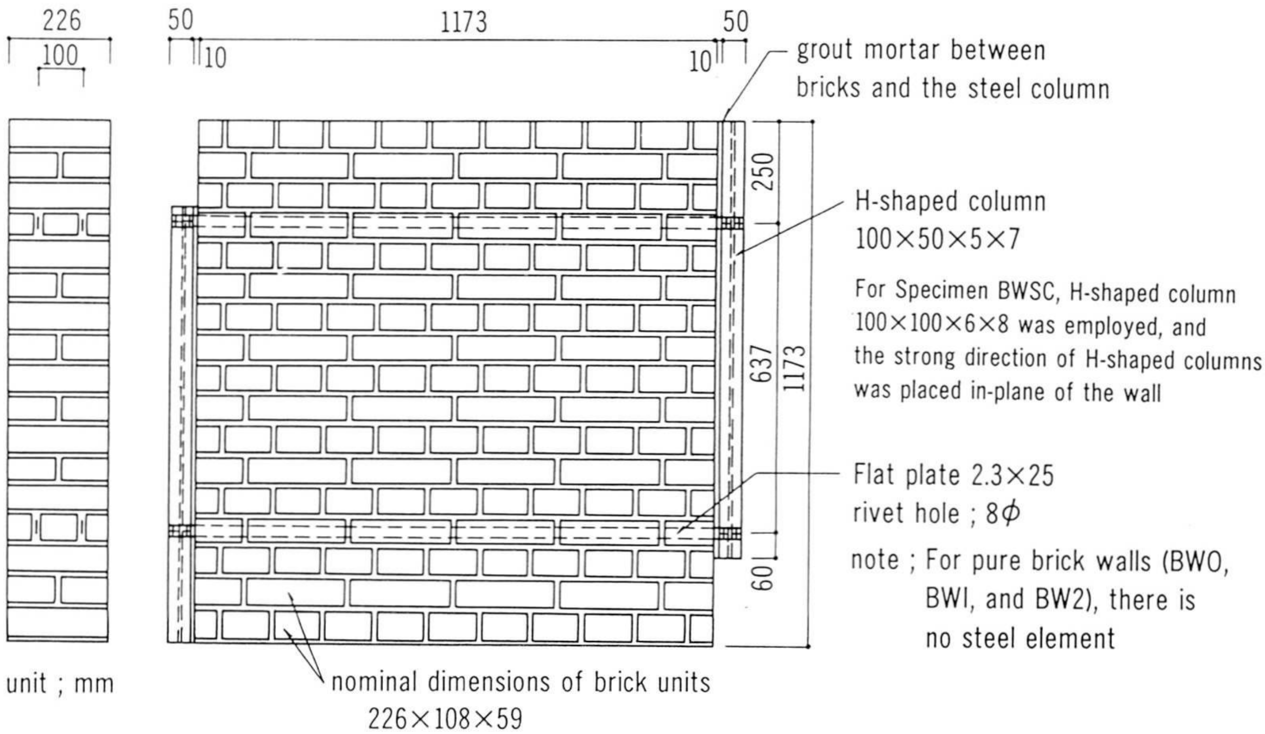


Fig. 2 Dimensions and detail of brick masonry wall specimens

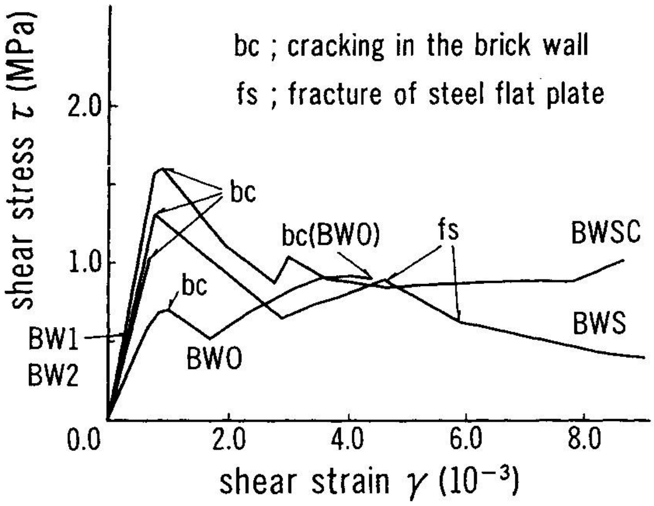


Fig.3 Shear stress and shear strain relationship

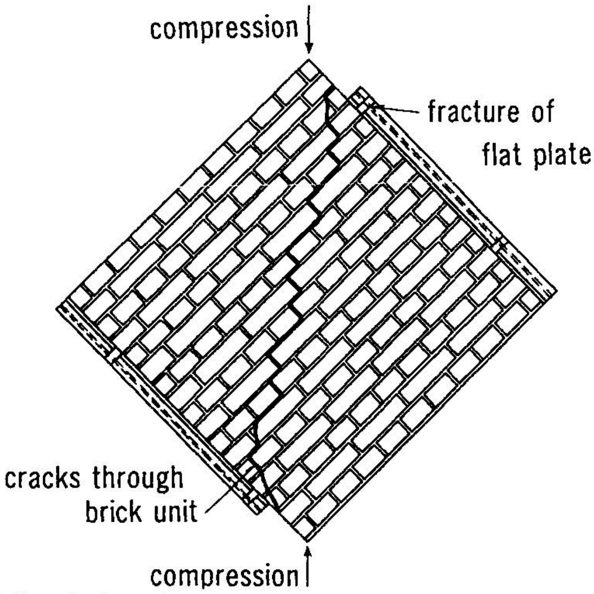


Fig.4 Crack pattern of a wall specimen

the displacement failed locally. This old wall might show the same brittle behavior as the new walls.

On the other hand, Specimen BWS kept up a reduced load after cracking of the masonry wall. This was due to the frictional resistance at the masonry bed joints and the tensile capacity of the steel structural elements inside the masonry wall. For this type of reinforced walls, it is possible to expect such post-cracking strength. At the ultimate stage, the wall steel elements fractured at the connection to the column steel element. This was due to stress concentration at the rivet holes in the connection. Specimen BWSC showed almost same behavior as Specimen BWS.

3. Test of bed joints

Bed joints of brick masonry walls were tested under combination of direct shear stress and normal stress to quantify the influence of normal stress on bed joint sliding shear strength. Major test variables were (1) normal stress level and (2) construction of brick masonry specimens.

The normal stress level of the existing building is approximately 0.5MPa, so 4 stress levels distributing around this value were applied as testing normal stress levels. Two types of specimens were employed. One was cut off the existing brick masonry building, so dimensions of the specimens were slightly distributed. The other was newly constructed with the same materials and methods as the brick masonry wall specimens mentioned before. Dimensions of a typical specimen are shown in Fig. 5. For each test variable, three specimens were tested to grasp scatter of test results.

Loading concept is shown in Fig. 6. A lateral hydraulic jack applied direct shear force to the bed joint. A vertical hydraulic actuator applied constant axial force to the upper side of the masonry specimen. Relative displacement between the upper and lower parts of the specimen was measured as the sliding displacement at the bed joint.

Representative relationships of shear stress - sliding displacement at the bed joint are shown in Fig. 7. It was observed that;
(1) Initial stiffness was very high.

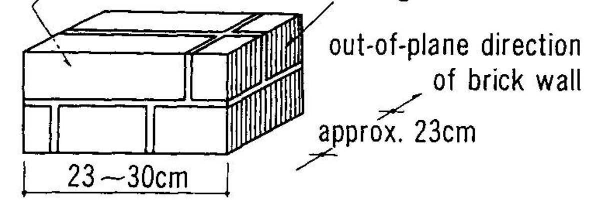


Fig.5 Dimensions of bed joint specimens

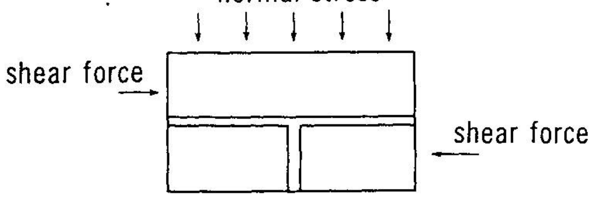


Fig.6 Loading concept

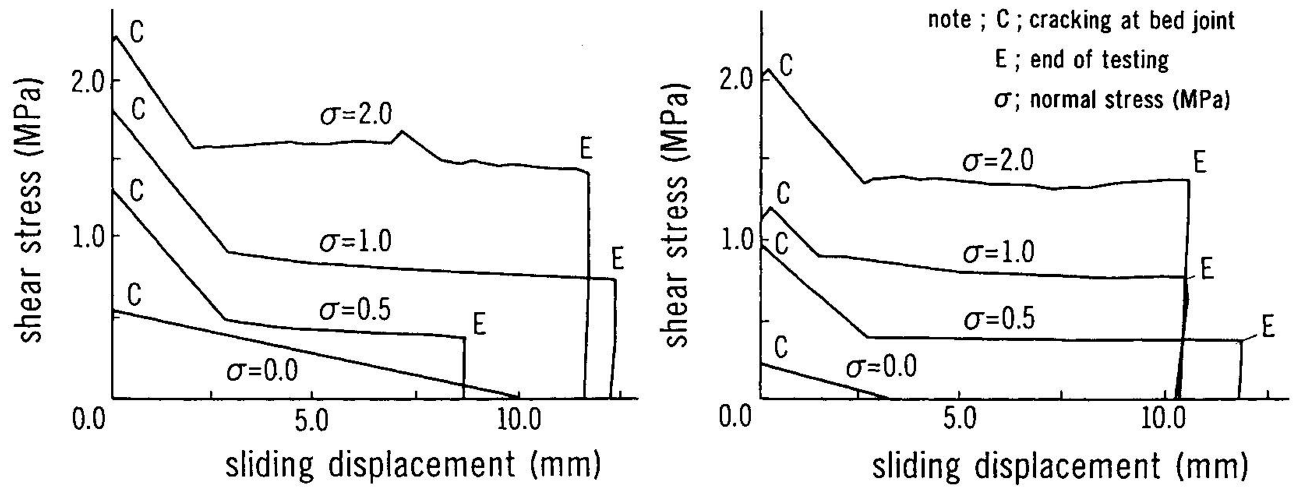


Fig.7 Shear stress and sliding displacement relationship

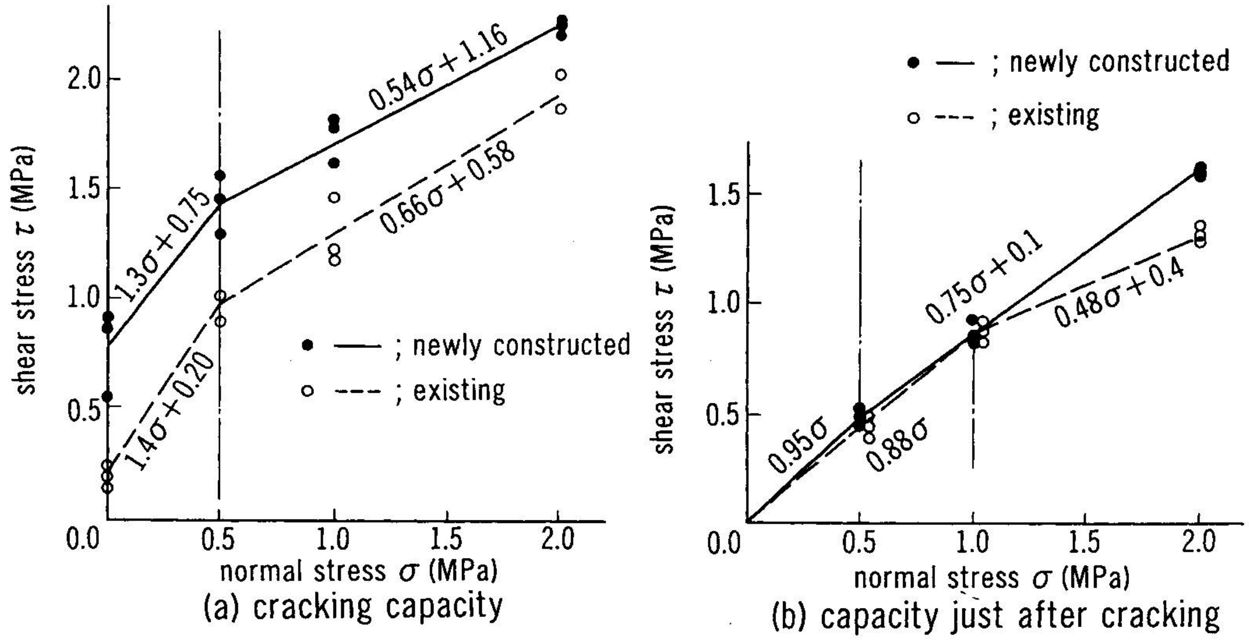


Fig.8 Sliding shear capacities

(2) When cracking occurred at the bed joint, the shear resistance was reduced very rapidly, however, this reduction stopped at a certain force level corresponding to the normal stress level.
(3) After the load reduction, the shear stress level was almost constant while the sliding displacement increased.
This post-cracking behavior is due to friction at the cracked bed joint interface.

Both shear capacities at the maximum load and in the post-cracking stage are shown in Fig. 8. The horizontal axis of this figure indicates the normal stress level. It is evidently understood that both shear capacities linearly increase as the normal stress level grows, where the normal stress is low. However, this increasing rate is reduced where the normal stress is high. Equations for evaluation of these shear capacities can be experimentally established as shown in this figure.

4. Test of retrofitted brick masonry walls

Two retrofitted brick masonry walls and one reinforced concrete wall were tested to verify structural performance of such retrofitting methods. Major dimensions of specimens and testing procedures, loading and measurement, were the same as employed in the previous testing of brick masonry walls.

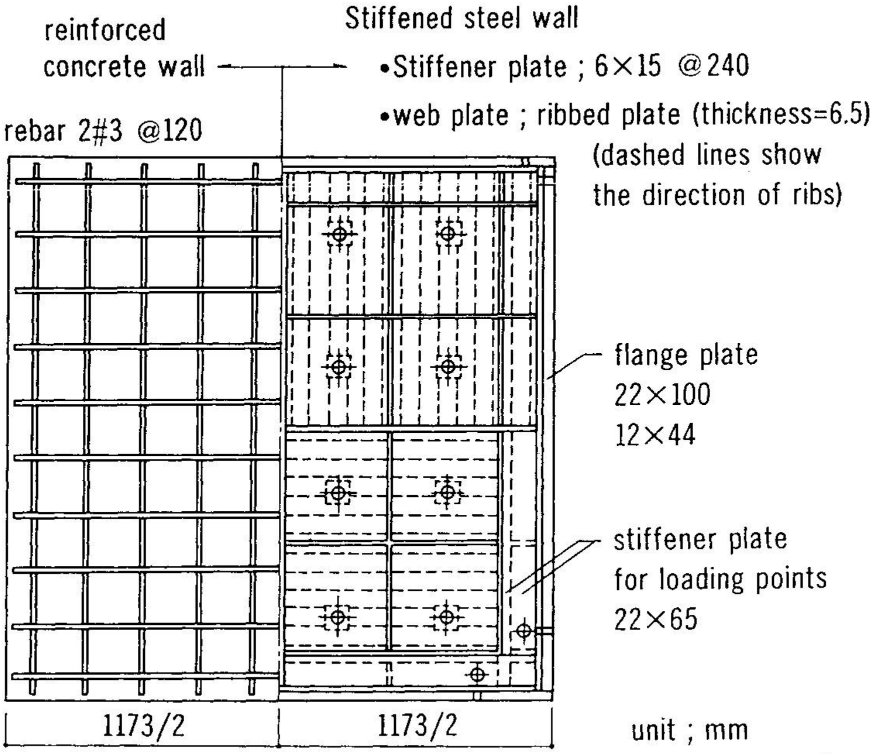


Fig.9 Dimensions of retrofitted wall specimens

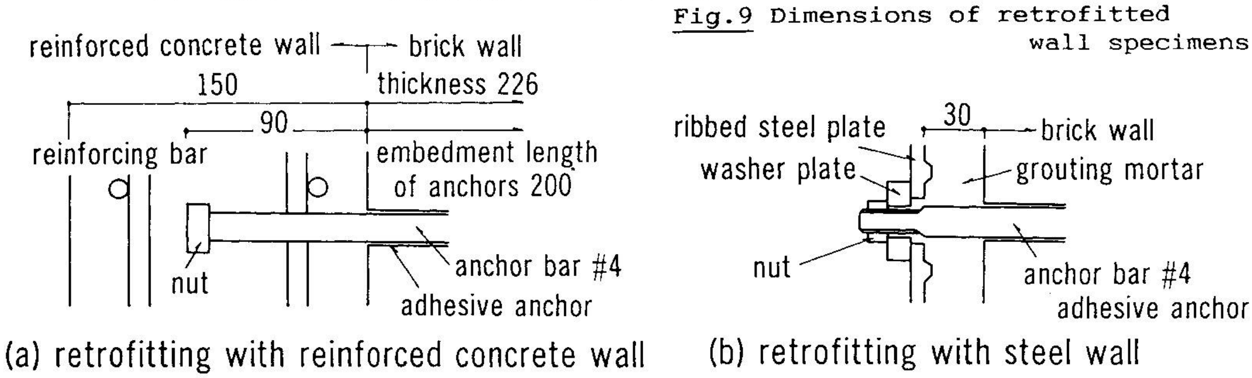


Fig.10 Detail of connections

Table 3 Material properties of retrofitted wall specimens

a) Concrete and mortar	E	σb
Concrete	22.6	24.7
Grout mortar	24.0	44.8

E : Modulus of elstiticity (GPa)
σb: Compressive strength (MPa)

b) Steel elements	E	σy	σt
Rebar #3	188	335*	557
Anchor bar #4	189	375	543
Ribbed plate t=6.5	210	327	429
Flange FB-22×100	209	292	455

E : Modulus of elsticity (GPa)
σy: Yield strength (MPa) *0.2% off set
σt: Tensile strength (MPa)

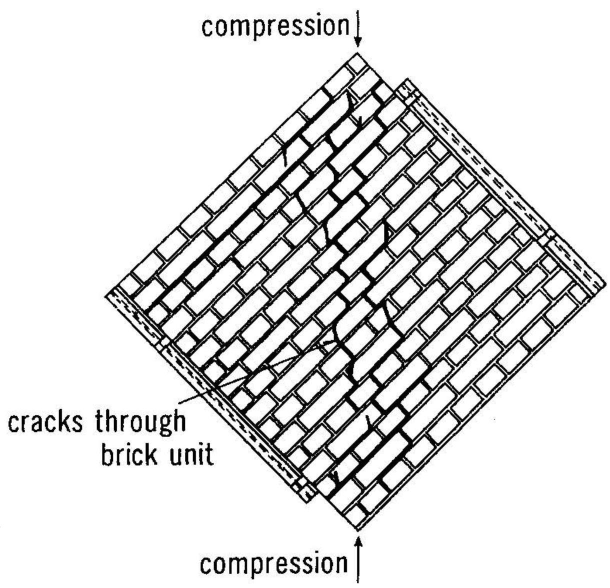
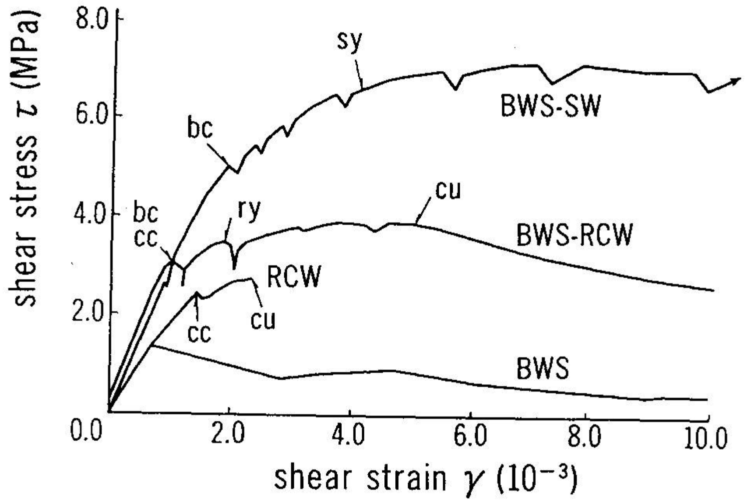


Fig. 11 Crack pattern of a retrofitted wall specimen



bc ; cracking in the brick wall
cc ; cracking in the concrete wall
ry ; yielding of reinforcement bars
sy ; yielding of steel panel
cu ; crushing of concrete

stress was obtained from division of shear force by area of section of brick masonry

Fig. 12 Shear stress and shear strain relationship

The retrofitted prototype was a brick masonry wall reinforced with steel elements, like Specimen BWS.

Dimensions and detailing of the specimens are shown in Fig. 9 and 10, respectively. Material properties are shown in Table 3. One retrofitted specimen, BWS-RCW, was a reinforced brick masonry wall connected to a new reinforced concrete wall. For the connection between the two walls, adhesive anchor bolts were employed. At the first stage of construction, these anchor bolts were embedded into the brick wall, and arrangement of rebars and casting of concrete for the additional wall was carried out. For comparison, a reinforced concrete wall with the same dimensions and detailing, Specimen RCW, was also tested. The other retrofitted specimen, BWS-SW, was a brick masonry wall connected to a stiffened steel wall with grout mortar and adhesive anchors. For the web plate of the steel wall, ribbed steel plates were employed to integrate grout mortar and the steel wall. Construction of the specimen was conducted as the following process: embedding of the adhesive anchors, setting of the steel plate wall, and pouring of grout mortar to the gap between these walls.

Cracks in the brick wall of the retrofitted specimens were more distributed than a non-retrofitted specimen, Specimen BWS, as shown in Fig. 11. Shear force - shear strain relationships of the specimens are shown in Fig. 12, comparing Specimen BWS. The retrofitted specimens showed much higher strength and deformation capacity than the non-retrofitted specimen. From these results, it is concluded that these two techniques are available for retrofitting of steel-reinforced brick masonry walls.

5. Proposal of a simplified evaluation method for brick masonry wall strength

One common design criterion for this type of brick masonry buildings is "not to allow cracking of brick walls." However, this criterion is too severe and not realistic for Tokyo station building, considering very large earthquake load which is regulated in the Japanese building code. From our testing, it can be predicted that brick masonry walls crack approximately at 0.1% of shear strain, and that the earthquake response of the walls of the station building is larger than this critical strain level.

The other criterion is "to allow cracking of brick walls but to avoid heavy damage, such as collapsing, etc." Fortunately, the walls of this station building are reinforced. It can be expected that in the post-cracking stage, friction at the cracked interfaces can transfer earthquake loads as long as reinforcing steel elements do not fracture. The authors carried out testing of the connections of the steel structural elements which were cut off the existing station building. Test results showed that yielding of the steel elements occurred before fracturing at the rivet bolt holes and that elongation of the steel elements at the fracturing was larger than 1%. That is, some plastic deformation capacity after cracking may be expected. It can be concluded that the design criterion, to allow cracking, is available. On the basis of this discussion, the authors propose a simplified design strength evaluation as follows.

Contribution of brick masonry walls is defined as a function of axial stress σ_L, or long-term axial stress. In actual, shear stress transfer is influenced by aspect ratio of the wall, strengths of materials and so on. These influences, however, are very complicated. It is judged that the following equation is adequate on the point of view of simplicity. The constant, 0.8, is determined from the tests of bed joints.

$\tau=0.8\sigma_L$ Eq.1

Contribution of reinforcing steel elements is evaluated as the following equation, based on the testing of reinforced brick masonry walls.

$\tau=p_s\sigma_s$ Eq.2

where, p_s: ratio of horizontal reinforcing steel elements inside the brick wall panels, σ_s: yield stress of these steel elements.

From the test results of the retrofitted specimens and usual design assumptions in Japan, contributions of retrofit walls are estimated as follows.

$\tau=F_c/10*(t_{RC}/t_b)$ for retrofitting reinforced concrete walls Eq.3

$\tau=\tau_y*(t_s/t_b)$ for retrofitting steel walls Eq.4

where, F_c: concrete compressive strength, τ_y: shear yielding stress of steel, t_{RC}: thickness of the reinforced concrete wall, t_s: thickness of the steel wall, t_b: thickness of the brick wall.

Total shear strength is summation of these contributions.

Table 4 shows comparison between test results and the estimated values by this method. However, for application to the testing mentioned before, slight modification was needed as follows; (a) axial stress was equal to shear stress due to loading condition, (b) flat plate did not show yielding up to fracturing at the connection due to high material strength, so the calculation of Eq. 2 was carried out employing the predicted stress which might occur at fracturing of the connection, and (c) for the retrofitted specimens, it was impossible to distinguish the bearing axial stress of the brick masonry from that of the retrofitting element, thus capacity was calculated by summation of the brick masonry part, namely BWS, and the retrofitted part. The evaluation is higher than the test results for brick masonry walls, however lower for the retrofitted specimens. Further study is necessary to verify the evaluating method for contribution of brick masonry walls.

Table 4 Estimation of shear strength of reinforced brick walls

Specimen	Test*	Estimate
BWS	8.13	11.06
BWSC	8.64	11.47
BWS-RCW	38.5	27.4
BWS-SW	66.6	65.4

*: capacity at 0.4% of shear strain

6. Conclusion

Testings of brick masonry walls were carried out for evaluation of seismic performance and establishment of seismic retrofitting methods of a historical brick masonry building reinforced with steel elements. From the testing of five brick masonry wall specimens, it was found that some bearing capacity in the post-cracking stage can be expected due to friction of cracked interface caused by reinforcement with steel structural elements. Contribution of the friction was determined from the testing of mortar bed joints of brick masonry. Seismic retrofitting methods, addition of reinforced concrete walls or that of steel walls, was tested. A simplified evaluation method of the bearing capacity of such brick masonry walls in the post-cracking stage and that for retrofitted walls was proposed, as shown eqs. (1) to (4). In this method, shear strength of the wall is evaluated as summation of the next three components; that is, (a) friction, which is a simple function of axial stress, (b) tensile resistance of steel elements, which is corresponding to yield force of the elements, and (c) contribution of retrofitting walls, if any.

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