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SEISMIC RESPONSE ESTIMATION OF COMPOSITE MASONRY STRUCTURE BY EQUIVALENT LINEARIZATION METHOD

Tetsuya NISHIDA[1] , Naofumi TERAMOTO[2], Jun KOBAYASHI[3]
Norio INOUE[4], Norio HORI[5] and Xianhui CAI[6]

SUMMARY

To improve seismic performance of masonry structure, a simple and economical structural system called composite masonry structure is proposed. To discuss the seismic response characteristics of the proposed structural system, a series of pseudo-dynamic tests using 1/2 scale specimen was carried out. Based on the hysteresis loops obtained from the tests, equivalent viscous damping associated with inelastic displacement is studied. And the maximum response displacement is estimated by an equivalent linearization method. Based on the method, the seismic response of composite masonry structures is roughly estimated, although the estimated displacement is little smaller than the test results when the response exceeds the maximum strength point. But this study also suggests that the maximum response displacement can be approximately estimated by the equivalent linearization method, if the maximum response remains within the limit used in practical seismic design.

INTRODUCTION

Masonry structure is world-widely used for residential buildings in spite of its vulnerability during earthquakes. Severe damage of masonry structures caused by recent earthquake in Turkey and Taiwan has highlighted the need for enhancing seismic performance of such structures. In China, to improve seismic performance of masonry structure, a simple and economical structural system is proposed by Dalian University of Technology [1]. It is called composite masonry structure and it is shown in Figure 1. Composite block masonry wall consists of block masonry wall surrounded by post-cast reinforced concrete beams and columns with shear keys.

The strength and ductility of composite block masonry wall was discussed by experimental and analytical studies in recent papers [2]-[5]. In this study, to discuss the seismic response estimation of composite

1 Lecturer, Akita Prefectural University, Japan, tetsuya.nishida@akita-pu.ac.jp
2 Research Associate, Akita Prefectural University, Japan, tera@akita-pu.ac.jp
3 Professor, Akita Prefectural University, Japan, jun.kobayashi@akita-pu.ac.jp
4 Professor, Tohoku University, Japan, inoue@struct.archi.tohoku.ac.jp
5 Research Associate, Tohoku University, Japan, hori@struct.archi.tohoku.ac.jp
6 Associate professor, Dalian University of Technology, P.R. China, caixianhui@263.net

masonry structure, a series of pseudo-dynamic tests using 1/2 scale specimen was carried out. Based on the hysteresis loops obtained from the tests, equivalent viscous damping associated with inelastic displacement is studied. And the maximum response displacement is estimated by an equivalent linearization method.

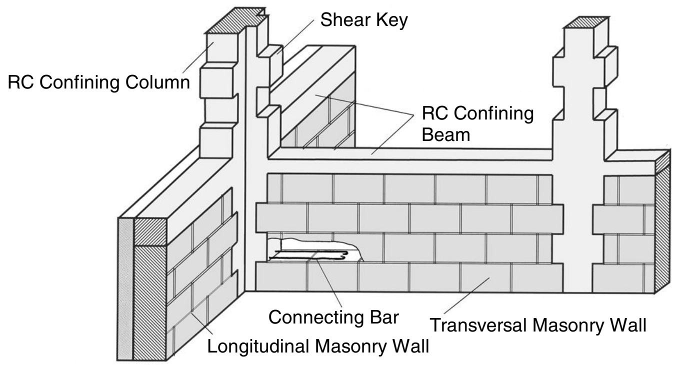


Figure 1 Composite block masonry structure

SEISMIC DESIGN OF COMPOSITE MASONRY STRUCTURE IN CHINA

In China, three-level seismic target is considered in the seismic design of composite masonry structure, and the building structure is designed in two steps [6]. The three-level seismic fortification target and the two-step seismic design are as follows:

The three-level seismic fortification target

Level 1: The building structure should not be damaged in small earthquake.

Level 2: The building structure might be damaged, but should not be seriously damaged and could be repaired, in moderate earthquake.

Level 3: The building structure might be seriously damaged, but should not collapse in strong earthquake. Peak acceleration and shear coefficient according to the different earthquake intensity are tabulated in Table 1.

The two-step seismic design

Step 1: Strength check according to seismic fortification level 1 and 2.

Step 2: Inelastic deformation check according to seismic fortification level 3.

In the seismic design of composite masonry structures, only the first step design (strength check) is performed. Because the inelastic calculation of composite masonry structures is complicated, the second step design (deformation check) is omitted. And the second step is assured by structural detail regulations instead. One of the purposes of this study is to provide a practical method for examining the maximum deformation during earthquakes.

Table 1 Assumed peak acceleration and shear coefficient of strong earthquake

Earthquake Intensity	Peak Acceleration (Gal)	Shear Coefficient
7	220	0.50
8	400	0.90
9	600	1.40

STRUCTURAL TEST

Specimen

The test specimen of composite masonry wall is shown in Figure 2. The inner panel is composed of concrete blocks without vertical reinforcement and surrounded by reinforced concrete beams and columns with shear keys. Horizontal reinforcements are provided in horizontal mortar joints in the alternate layer up to 800 mm from the inner sides of both columns. The specimen has one bay and one story, but additional upper parts are attached on the beam to distribute axial load uniformly over horizontal wall section. The scale of the test specimen is 1/2 but concrete blocks have actual size. The sequences of construction of specimen are: (1) fastening of column reinforcement, (2) constructing block masonry wall, (3) fastening of beam reinforcement, and (4) casting concrete into reinforced concrete columns and beam. The strengths of main materials are shown in Table 2.

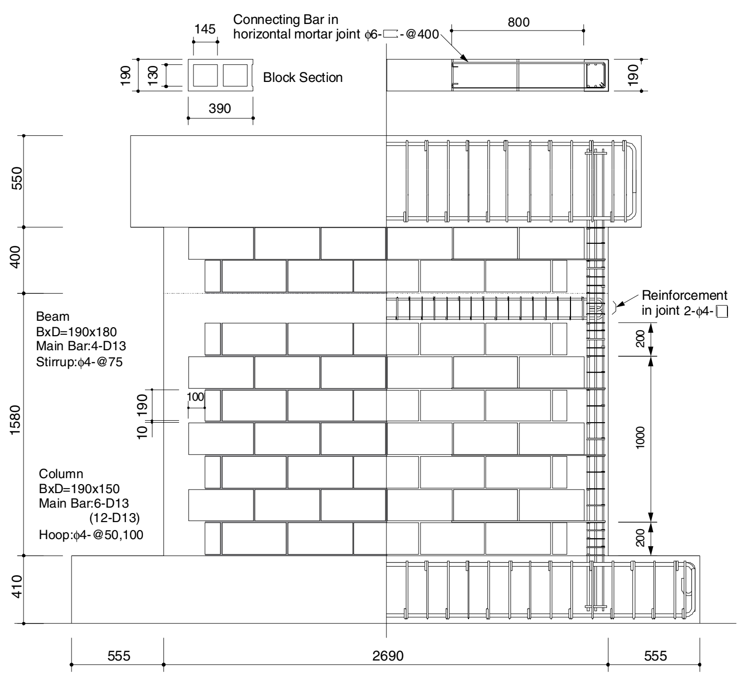


Figure 2 Test specimen

Table 2 Strength of materials

Yield strength of Main Bar D13	σ_y=364 N/mm^2
Tensile strength of Hoop φ4	σ_u=537 N/mm^2
Compressive Strength of Concrete	σ_B=25.1 N/mm^2
Compressive Strength of Joint Mortar	σ_B=11.8 N/mm^2

Pseudo-dynamic test

The loading system used for pseudo-dynamic test series is shown in Figure 3, and the parameters of specimens are tabulated in Table 3. The constant vertical load to simulate the dead load was applied to the specimen by the three vertical hydraulic jacks. The simulate earthquake load was applied to the center of beam by horizontal hydraulic jack controlled by the personal computer which calculates the earthquake response using Single-Degree-of-Freedom (SDOF) model.

In the pseudo-dynamic test series, shear span ratio, column reinforcement and input motion are parameters. The specimen with shear span ratio 0.55 is a cantilever type and subjected only to horizontal force. In the case of shear span ratio 1.0 and 1.5, representing walls in multi-story building, the axial loads applied by the vertical jack J1 and J3 were changed to generate the additional moment corresponding to the shear span ratio.

The mass of SDOF model was assumed 1000 ton in order to obtain the natural period (about 0.2 second) corresponding to 5-story composite block masonry building. Newmark-β method is used for numerical integrations, and the time increment for analysis and pseudo-dynamic test was 0.01 second. The damping factor of the model was 0 %.

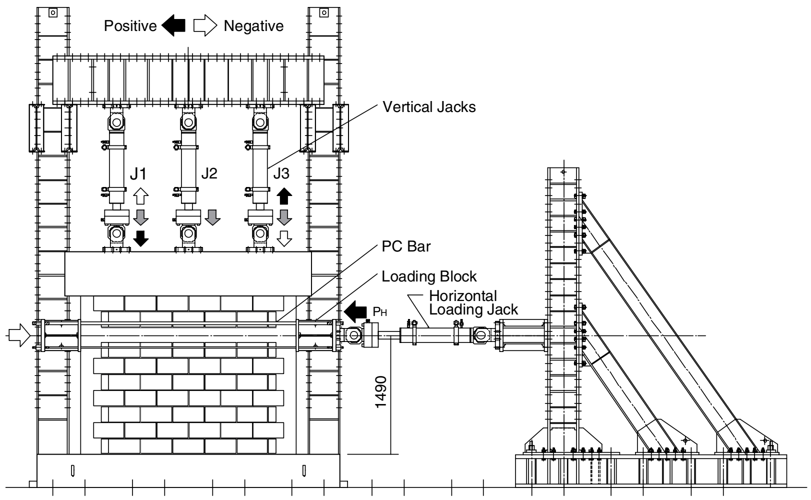


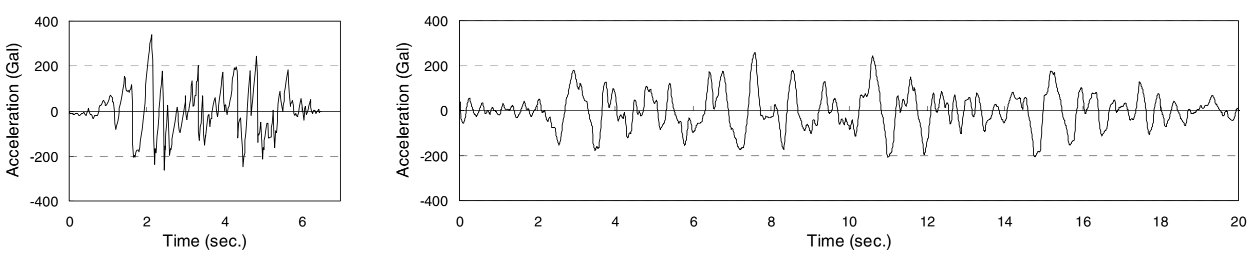
Figure 3 Loading system

Table 2 Test parameters

Specimen	Shear Span Ratio	Column Reinforcement	Input Motion
CM1	0.55	6-D13 (p_g=2.67%)	El Centro NS : 10,60 Gal
CM2	1.0		El Centro NS : 10,20,30,45 Gal
CM3	1.5		
CM4		12-D13 (p_g=5.35%)	
CM5	1.0	6-D13 (p_g=2.67%)	Tohoku-Univ. NS : 10,30 Gal

Input motion

El Centro NS component of the 1940 Imperial Valley earthquake and Tohoku-Univ. NS component of the 1978 Miyagiken-oki earthquake were used as input motions. These input motions are shown in Figure 4. As shown in Table 2, the peak acceleration of the input motion was gradually increased in the pseudo-dynamic tests. When the level of the input motion is compared with the level of the actual earthquake motion, these values have to be multiplied by 20 according to Modeling Law. So the input level of 30 Gal corresponds to the earthquake intensity of 9 (600 Gal) in Chinese design code.



(a) El Centro NS (b) Tohoku-Univ. NS

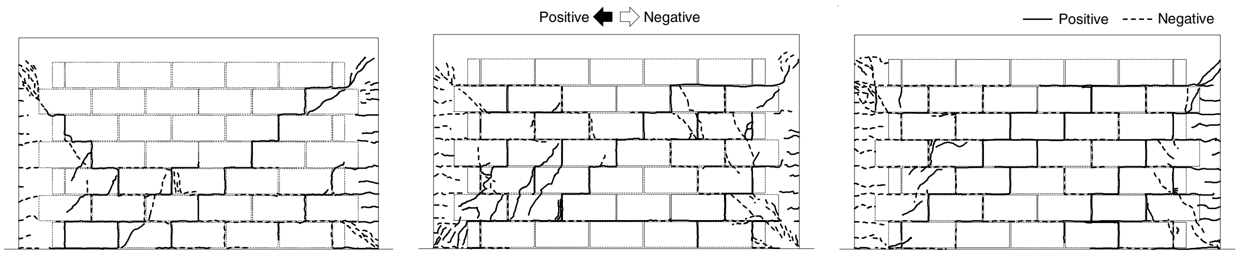
Figure 4 Input motions

Test result

Final crack patterns

Final crack patterns of specimens CM1, CM2 and CM3 are shown in Figure 5. In general, dominant cracks were stairs-like diagonal cracks, and most cracks took place at mortar joint. The initial cracks occurred at the bottom of the tensile side column when the deformation angle was about 1/3000 rad. Then the stairs-like diagonal cracks were observed at about 1/1000 rad. In the specimens CM1, CM2, CM4 and

CM5, some inclined cracks traversed block units and RC column in the lower part of the compressive side of the wall at about 1/500 rad. Finally, shear failure was observed at the bottom of the compressive side column. On the other hand, in the specimen CM3, some inclined cracks were observed in the lower part of the compressive side of the wall, but shear failure was not observed.

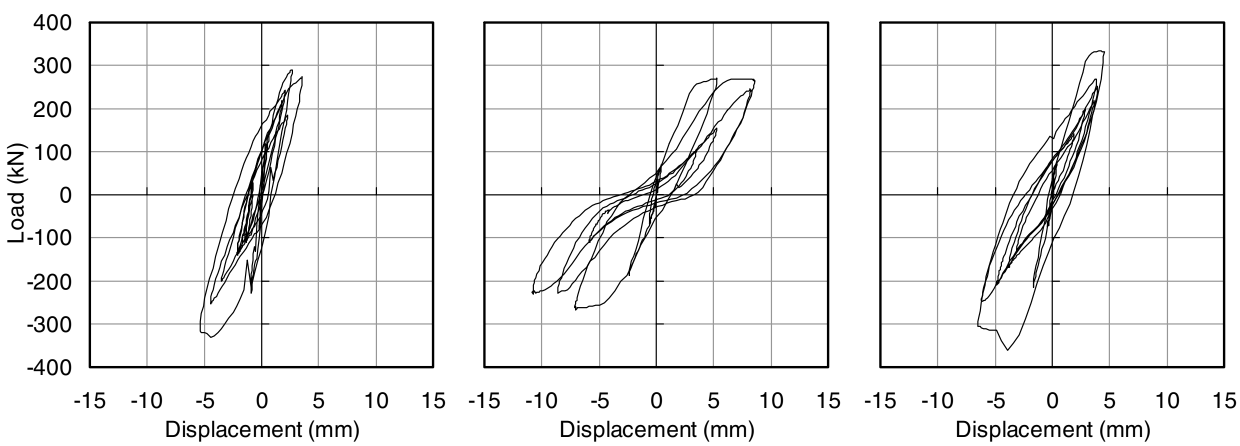


(a) Specimen CM1 (b) Specimen CM2 (c) Specimen CM3

Figure 5 Final crack patterns

Load – displacement relationships

Figure 6 shows Lateral load – story displacement relationships of specimens CM2, CM3 and CM4 when the level of input motion is 30 Gal. In case of the specimens CM1, CM2, CM4 and CM5 with shear failure mode, the load – displacement relationships are spindle shape, and the area within the hysteresis loop of each cycle is large. On the other hand, in case of the specimen CM3 with flexural failure mode, the hysteresis curves show pinched shape with small loop areas.

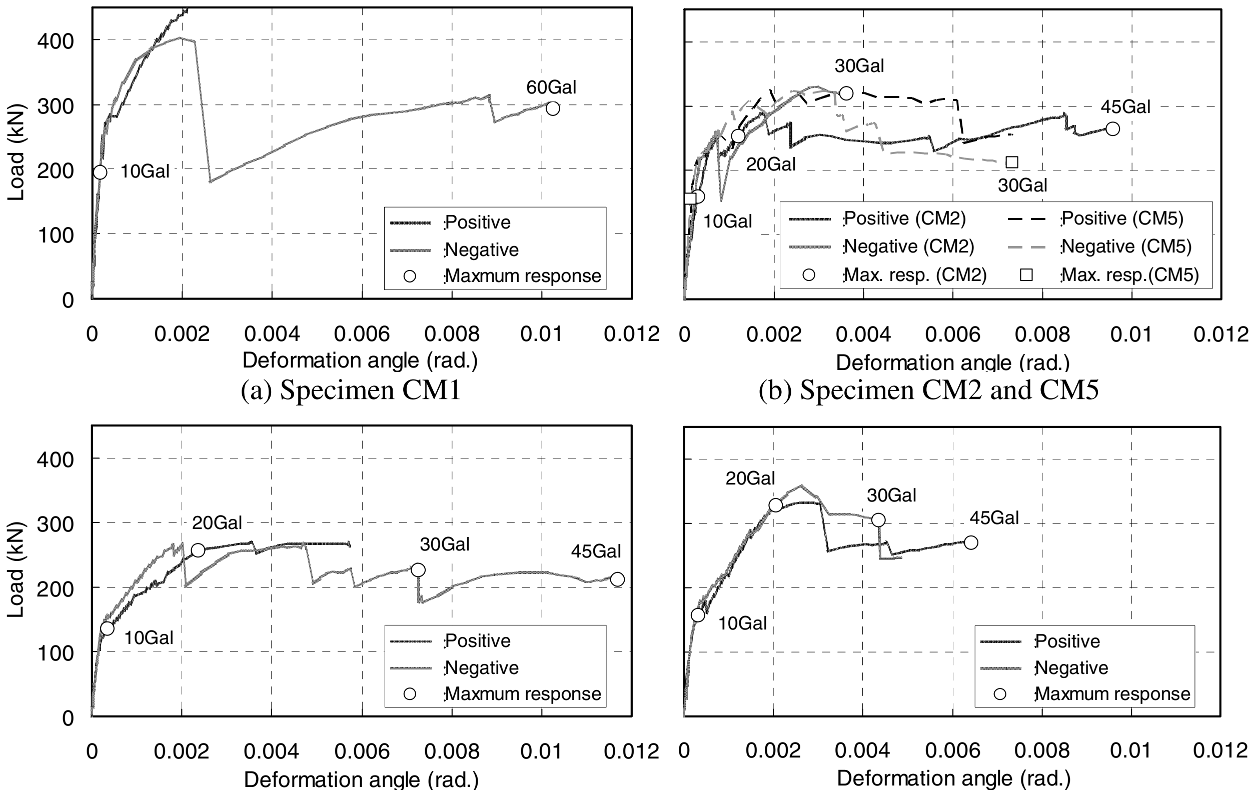


(a) Specimen CM2 (b) Specimen CM3 (c) Specimen CM4

Figure 6 Lateral load – story displacement relationships

Maximum responses

The envelope curves of load – displacement relationships and the maximum responses of each input motion level are compared in Figure 7. In the test of input motion of 10 Gal, the maximum displacements of all the specimens are almost same. In the input motion level of 30 Gal corresponding to the earthquake intensity of 9 (600 Gal), the maximum displacement of specimen CM3 of flexural failure mode is about twice larger than specimens CM2 and CM4 which failed in shear mode. In comparison between specimen CM5 subjected to the input motion of Tohoku-Univ. NS which has longer duration time and specimen CM2 subjected to that of El Centro NS, the maximum displacement of specimen CM5 is nearly double of specimens CM2.



(a) Specimen CM1 (b) Specimen CM2 and CM5

(c) Specimen CM3 (d) Specimen CM4

Figure 7 Envelope curves and maximum responses

SEISMIC RESPONSE ESTIMATION BY EQUIVALENT LINEARIZATION METHOD

To check the inelastic deformation according to the seismic target level 3, equivalent linearization method used in the Japanese standard is applied to composite masonry structures. This simplified method is widely used for estimation of RC buildings in Japan. Application of this method requires equivalent viscous damping - deformation relation and load - displacement relation.

Equivalent viscous damping

The relations between equivalent viscous damping and the maximum deformation are obtained by each input motion level of the tests, and the results are shown in Figure 8. The result of the specimen (shear span ratio: 0.55, input motion: El Centro NS; 10, 15, 20, 30, 45 Gal) which was discussed in Ref.5 is also shown in this figure. The equivalent viscous damping which is generally denoted as substitute damping was calculated by the following Equation (1).

$$h_s = \frac{-\int_0^T \ddot{x}_0 \cdot \dot{x}\, dt}{2\omega_e \int_0^T \dot{x}^2 dt} \tag{1}$$

where: $\dot{x}$:response velocity, $\ddot{x}_0$:ground acceleration, T :duration time, ω_e :equivalent circular frequency calculated using the secant stiffness at the maximum displacement of each input motion level.

In the smaller deformation level, the equivalent viscous damping is about 8 % in all the specimens. When the deformation angle exceeds about 1/1000 rad. at which the stair-like diagonal cracks occur, the equivalent viscous damping tends to increase with increase of the drift angle. In the specimen CM3, the equivalent viscous damping is not so high in the large deformation level because the hysteresis curves of flexural failure mode are pinched shape of which areas are comparatively small.

To discuss the effects of shear span ratio and failure mode on the equivalent viscous damping, the equivalent damping factors of each specimen are compared with the calculated values using the following equation (2) which is used for estimation of RC structures in Japan.

$$h_e = \gamma\left(1 - \frac{1}{\sqrt{\mu}}\right) + h_0 \tag{2}$$

where: γ :constant, μ :ductility factor, h_0 :initial damping factor.

To apply equation (2) to composite masonry structures, the initial damping factor and ductility factor are assumed as follows:

(1) The initial damping factor is 8 %.

(2) The reference deformation angle used for calculation of ductility factor is 1/1000 rad. at which the stair-like diagonal cracks occur.

The calculated results when the value γ changes to 0.1, 0.2 and 0.3 are shown in Figure 8. From the comparison between tested and calculated results, the equivalent damping factors of the specimens of shear span ratio 0.55 and 1.0 are larger than those of other specimens. And those values are nearly same to the estimated values calculated with $\gamma = 0.3$. The tested values of specimen CM4 of shear span ratio 1.5 (failed in shear) are close to the estimated values calculated with $\gamma = 0.2$. And the tested values of specimen CM3 of shear span ratio 1.5 (failed in flexure) are smallest and these values agree with the estimated values calculated with $\gamma = 0.1$.

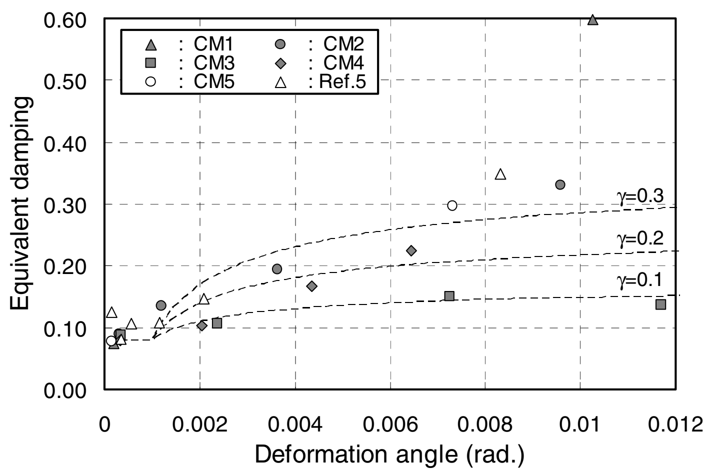


Figure 8 Equivalent viscous damping – maximum response deformation relations

Estimation of maximum response displacement

In order to examine the possibility of estimating the maximum response displacement of composite masonry structure, equivalent linearization method using the S_A-S_D response spectrum is applied to the test results. The load – displacement relation is defined with a multi-linear relation considering the initial stiffness, the crack points, the maximum strength point and the part after the maximum strength point based on the envelope curves shown in Figure 7. The equivalent viscous damping – deformation relation is assumed by equation (2), the equivalent damping is calculated with $\gamma = 0.3$ in case of specimen CM1, CM2 and CM5, $\gamma = 0.2$ in case of specimen CM4 and $\gamma = 0.1$ in case of specimen CM3.

Figure 9 shows an example of the estimation using the S_A-S_D response spectrum. S_A-S_D contours are calculated for each input motion level. The intersections of S_A-S_D contours with skeleton curves give the estimated displacement and they are plotted in Figure 9 as circular marks. Test results are shown by the square mark in this figures, and compared with the estimated values. Figure 10 shows the estimated response of all the specimens compared with test results. Some estimated displacements are smaller than the test results in the region of drift angle larger than 0.3 %. In the seismic design of composite masonry structures, in order to prevent to collapse of the building in the strong earthquake, it is necessary that the seismic response dose not exceed the ultimate strength. So the structures have to be designed in the range of smaller drift angle than about 0.2 %. In this sense, the estimated values agree with the tested values in the region used in practical design level.

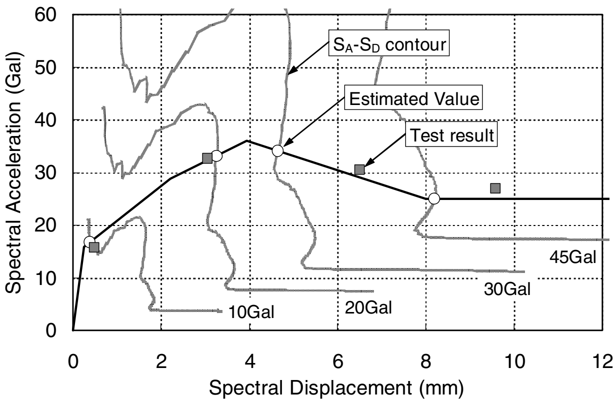


Figure 9 Example of estimation (Specimen CM4)

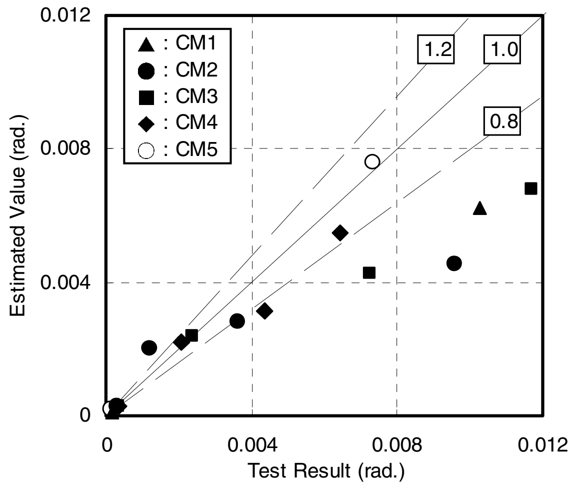


Figure 10 Comparison between test results and estimated values

CONCLUSIONS

Based on the results obtain by the pseudo-dynamic tests, equivalent viscous damping associated with inelastic displacement is studied. And the possibility of estimating the maximum response displacement of composite masonry structure and the applicability of equivalent linearization method to composite masonry structures are discussed. The seismic response of composite masonry structures is roughly estimated by equivalent linearization method, although the estimated displacement tends to be little smaller than the test results in case the response exceeds the maximum strength point. As a result, this study suggests that seismic response estimation by simplified method is applicable to composite masonry structures when the maximum response displacement is within the practical design limits.

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