A MACRO MODEL FOR REINFORCED CONCRETE STRUCTURAL WALLS HAVING VARIOUS OPENING RATIOS

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ABSTRACT :

In the current design cord of AIJ(Architectural Institute of Japan), the minimum strength reduction factor of a structural wall due to the openings is limited to 0.6 by restricting the maximum ratio of opening sizes to the corresponding wall length and height. In this study, a modified macro-model is proposed to establish a simple method to design a reinforced concrete structural wall having large openings with a significant eccentricity. The model is composed of column elements and strut-and-tie elements which represents the wall parts. The proposed model was established by modifying the model proposed by Tekehara et al. to obtain more accurate load-deformation relationships in the case of multi-story walls with eccentric openings. The main modified item is the additional modeling of the concrete struts and tension ties located around the openings. The proposed model was applied to three of the three-storied reinforced concrete structural walls with eccentric openings, where the adjacent walls between the stories were connected by rigid beam elements and the springs expressed by bi-linear models. The results obtained from the proposed model were compared with those from experiments and the model by Takehara et al. and the adequacy of the proposed model was confirmed.

KEYWORDS : Reinforced Concrete, Structural Walls, Macro Model, Various Opening Ratios

1. INTRODUCTION

The structural walls have been usually adopted as the main earthquake-resistant element of the reinforced concrete buildings, but they often have some openings according to the intention of the architectural design. In the current design cord of AIJ(Architectural Institute of Japan), the minimum strength reduction factor of a structural wall due to the openings is limited to 0.6 by restricting the maximum ratio of opening sizes to the corresponding wall length and height. These limit and restrictions are provided to make the conventional method applicable to wall structures, where the method is based on the past experimental data and observation. If the opening sizes do not satisfy the above restrictions, it is required to solve the structure as a frame having finite rigid area for the beam column joint. When the opening locations are too eccentric and/or the shapes of large openings are not rectangular, linear or non-linear FEM analyses are often used, although it takes more time for the design. From the above point of view, it is desirable to establish a simple but rational method. TAKEHARA et al. tried to evaluate the strength and deformation of a single-story structural wall with an eccentric opening by a macro model based on the strut-and-tie model. Though the good result was found, the applied examples were only a few, and also no method for applying to the cases of continuous walls over the height of a multi-story building was exhibited. Moreover, the interaction between wall reinforcement and concrete struts formed possibly along the perimeter of openings were neglected in their model. The neglect of such interaction will result in an underestimate of seismic strength of walls when the openings are comparatively large. In this study, a modified multiple macro model which takes account of the possible interaction mentioned above was proposed by modifying the model by Tekehara et al.

The adequacy of the proposed model was testified by carrying out seismic loading tests on three of the 40% scale specimens of reinforced concrete structural walls having various opening ratios. From the comparison between the experimental and the analytical results, it was concluded that the proposed macro model was applicable even to the cases of multi-story structural walls having large eccentric openings.

2. EXPERIMENTAL PHASE

2.1. Specimens

As an analytical object of this study, three test specimens named as S1, M1 and L1 were used (see Table 1～4). They were 40% scale models of concrete structural walls with eccentric openings. The experiments were conducted at Kyoto University in 2006 and 2007. The height and the span length of the specimen can be seen in Fig. 1. Table 1 shows the sectional dimensions of beams and columns and also some details of reinforcement. The opening ratios adopted as an experimental parameter are listed in Table 2, where it was defined as $\sqrt{h_0 l_0 / hl}$, h_0 and l_0 is the height and length of the opening, respectively. h is the center to center spacing between the upper and lower beams, l is the center to center spacing between two side columns. The opening ratio was assumed to be 0.30 in S1, 0.34 in M1, and 0.46 in L1. Material properties of reinforcement and concrete adopted for the specimens are listed in Tables 3 and 4, respectively. As one of the purposes of this study is to clarify the influence of openings on the shear capacity of a structural wall, both specimens were designed to fail in shear, not in flexure. It is noted that the maximum opening ratio is limited to 0.4 in the design cord of AIJ when the wall strength reduction factor due to opening is determined by the empirical equations specified in the code.

Table 1 Section Size and Reinforcement Arrangement

Member	Section size	Main bar type	steel ratio	Hoop bar type	Steel ratio
Side column	300×300	8-D19	2.55%	2-Φ10@75	0.63%
Beam	200×300	2-D13	0.47%	2-Φ6@100	0.32%
Foundation beam	600×400	4-D25	1.62%	4-D10@100	0.52%
Load beam	400×400	2-D25	1.22%	2-D10@100	0.39%

Table 2 Reinforcement of wall

specimen	thickness	Wall bar type	Vertical reinforcing bar of opening	horizontal reinforcing bar of opening(upper)	horizontal reinforcing bar of opening(down)	Opening ratio
S1	80mm	D6@100	1-D13	2-D10	1-D13	0.30
M1	80mm	D6@100	3-D13	3-D10	/	0.34
L1	80mm	D6@100	1-D16	2-D13	1-D16	0.46

Table 3 Reinforcement Material Properties

Type	D6	D10	D13	D16	D19	Φ10
Yield strength(MPa)	425	366	369	400	384	985
Maximum strength(MPa)	538	509	522	569	616	1143
Young’s modulus(GPa)	204	180	189	194	183	197

Table 4 Concrete Material Properties

Specimen	Compressive strength(MPa)	Tensile strength(MPa)	Young’s modulus(GPa)
S1	25.1	2.2	21.7
M1	21.7	2.1	15.8
L1	28.9	/	26.0

2.2. Loading System

Figure 1 shows the loading system and the specimen (M1). The lateral load Q was applied statically to the loading beam through the arm and the steel plates by two 2000kN hydraulic jacks. Cyclic reversed horizontal loads were statically applied to the specimens in both positive and negative directions (see Fig.1), simulating earthquake forces. Loading was mainly controlled by measured displacement in terms of the story drift angle. The first cycle of loading was performed up to 200kN, subsequently two cycles of repeated loading were applied for each drift angle.

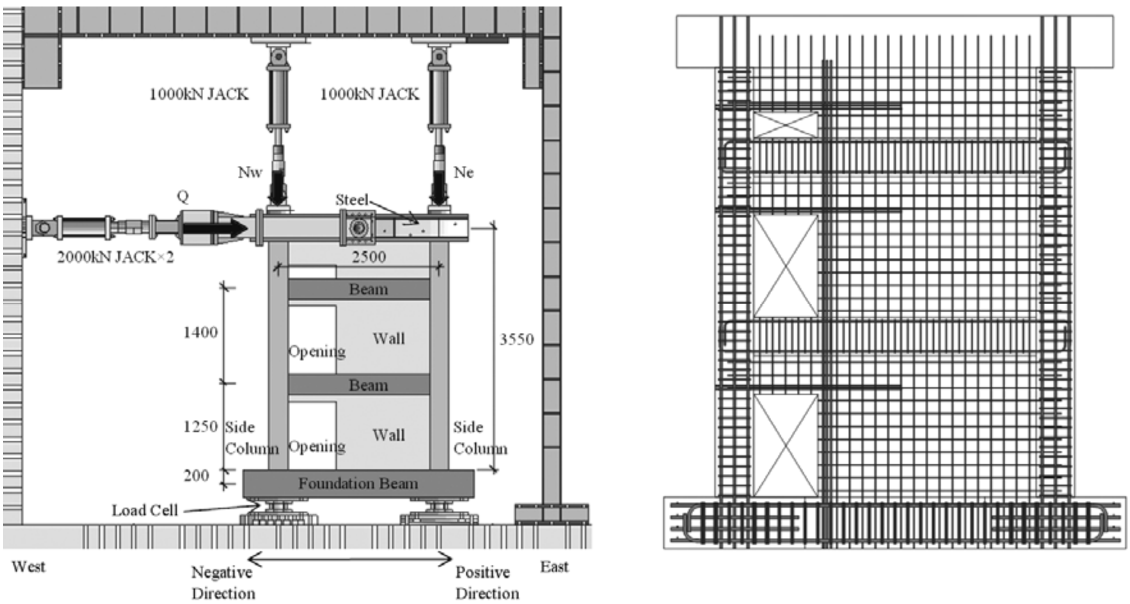

Figure 1 Loading System and Experimental Specimen(M1)

During the cyclic horizontal loading, vertical axial loads were also applied by two 1000kN hydraulic jacks assuming that the specimens are representing a part of the lower three stories of a typical reinforced concrete building with six stories. Hence, the vertical axial load levels were determined in accordance with the assumed long-term axial loads for a six story wall with three spans for S1 and that with one span for M1 and L1, respectively. Thus, 800kN (400kN for each jack) for S1 and 488kN (244kN for each jack) for M1 and L1 were determined as the basic axial loads. Moreover, controlling two hydraulic jacks, two vertical axial loads were adjusted each other as to keep the apparent shear span ratio (M/Qd) being always 1.0, where M=flexural moment applied to the base of the wall, Q=horizontal load applied to the loading beam, d=the distance between the center to center spacing between two side columns. This is to make the shear damages in the wall precede the flexural yielding of the wall. However, the influence of the axial load level on the shear capacity of each wall was insignificant as far as these test results concerned, because the side columns were not damaged till the end of the test.

3. MACRO MODEL

3.1. The Original Macro Model (Takehara et al.)

The original macro model shown in Fig. 2 was established by Takehara et al. In this study, this model was introduced assuming that each of the objective walls can be treated as a single-story wall taken out from the whole wall structure. This model corresponds to the situation that the diagonal cracks have been formed and significantly developed in the wall. It is composed of upper and lower beams (that is, loading beam and foundation beam), the side columns, the compressive struts, and the tensile reinforcement ties of the wall. The angle of the inclined compressive struts is assumed to be a constant degree against horizontal axis and denoted by θ. The compressive struts and the vertical/horizontal reinforcements which are placed in the direction passing through the openings are assumed to be ineffective and hence neglected. The loading beam and the foundation beam are treated as rigid elements. The side columns are represented by a model composed of the rigid elements, the elastic-plastic axial springs and the shear springs. The axial springs are placed at the center of the main reinforcement of each side column.

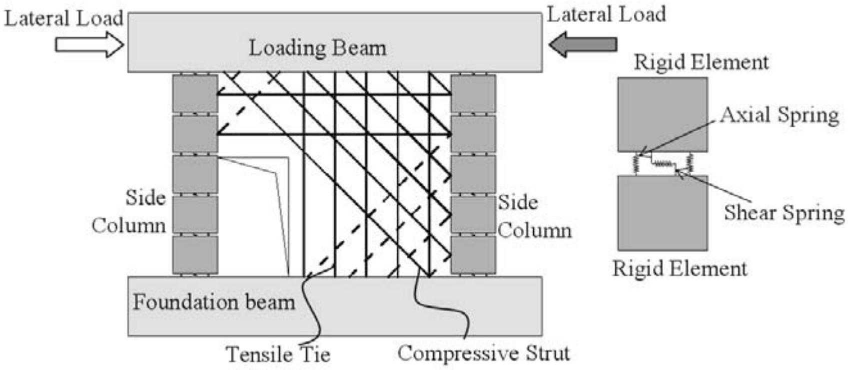

Figure 2 Original Macro Model by Takehara et al.

The strength and the rigidity of the axial spring and the rigidity of the shear springs are expressed as follows. Note that two axial springs are placed in each side column, hence the spring strength and rigidity are divided by 2 in Eqs. 1 to 4.

Axial strength of the spring in tension:
$$ {}_cN_{nt} = \frac{A_g \times {}_g\sigma_y}{2} \tag{1} $$

Axial rigidity of the spring in tension:
$$ {}_cK_{nt} = \frac{E_s \times A_g}{2\Delta h} \tag{2} $$

Axial strength of the spring in compression:
$$ {}_cN_{nc} = \frac{A_g \times {}_g\sigma_y + b \times D \times \sigma_d}{2} \tag{3} $$

Axial rigidity of the spring in compression:
$$ {}_cK_{nc} = \frac{E_s \times A_g + E_c \times b \times D}{2\Delta h} \tag{4} $$

Rigidity of the shear spring:
$$ {}_cK_s = \frac{{}_cK_n}{{}_cK_{nc}} \bullet \frac{G \times b \times D}{\Delta h} \tag{5} $$

Where A_g is the main bars area of side column, ${}_g\sigma_y$ is the yield strength of side column main bar, E_s is the reinforcement young's modulus, Δh is the center to center spacing between two rigid elements, $b \times D$ is the section area of side column, σ_d is the concrete compressive stress, E_c is the concrete young's modulus, G is the concrete shear modulus. ${}_cK_n$ is an average of the rigidity of two axial springs at the same loading cycle which is determined by taking account of the progress of the horizontal cracks in the column. Hence, the term of ${}_cK_n / {}_cK_{nc}$ in Eq.5 expresses the reduction of the shear rigidity due to such cracks. The model ties are substituted for the vertical/horizontal reinforcements placed within the strip element with width b_w, and their strength and rigidity are expressed as follows.

Tie strength:
$$ {}_bN_t = \rho_s \times {}_s\sigma_y \times b_w \times t \tag{6} $$

Tie rigidity:
$$ {}_bK_n = \rho_s \times E_s \times b_w \times t / L \tag{7} $$

Where ρ_s the wall reinforcement ratio, ${}_s\sigma_y$ is the yield strength of wall reinforcing bar, t is the thickness of wall, L is the length of wall. The compressive concrete struts follow the concrete constitutive relationship of Popovics below.

$$\sigma = \frac{n \bullet \xi}{n-1+\xi^{n}} \bullet \sigma_B^{'} \tag{8}$$

Where, $\sigma_B^{'} = 0.63\sigma_B$, $n = 0.57 \bullet 10^{-2} \bullet \sigma_B^{'} + 1$, $\xi = \varepsilon / \varepsilon_0$, $\varepsilon_0 = 4.29 \bullet 10^{-4} \bullet \sigma_B^{' \, 0.25}$. The first term, $\sigma_B^{'}$, is the effective concrete compressive strength of the wall where the parallel shear cracks are ideally formed and developed and the coefficient 0.63 was proposed from the experiment without openings by Takehara et al.

3.2. The Modified Multiple Macro Model

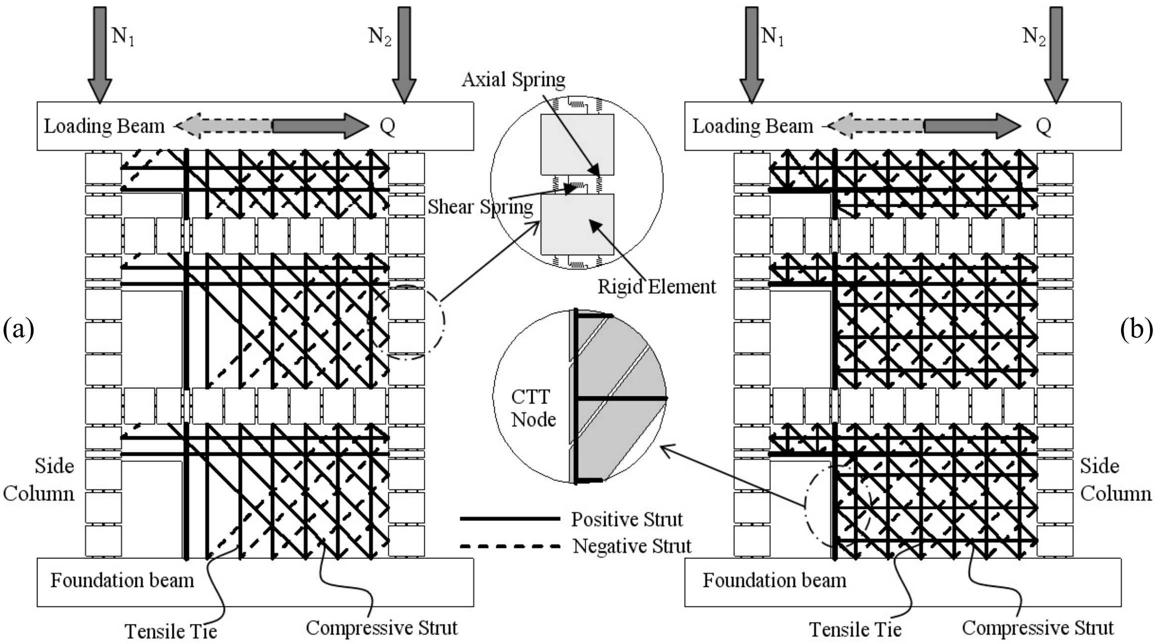

Figure 3 Multiple Macro Model (M1)

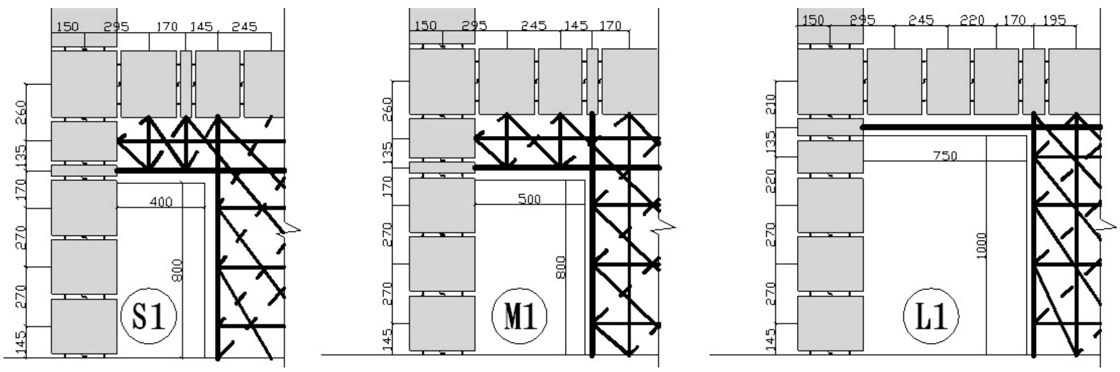

Figure 4 Dimensions of Rigid Elements around Opening

Figure 3(a) shows a multiple macro model which was assembled using the original macro model. Figure 3(b) shows a modified multiple macro model which was established by modifying the Takehara et al. Model. In this model, the effect of the reinforcement placed around the openings is taken into account. The main modified points from the original macro model are as follows.

1). As shown in Fig. 3, the beam was modeled using the rigid elements, the elastic-plastic axial springs, and the shear springs like the side column. The compressive struts and the tensile reinforcement ties in two adjacent story walls are connected by the beam rigid elements.

2). The multiple macro model shown in Fig. 3(a) didn't consider the effect of the reinforcements around the openings like the original model. However, the modified multiple macro model shown in Fig. 3(b) considered the effect of the vertical reinforcing bars around openings, on the premise that they were sufficiently anchored in the upper and lower beams. Moreover, it also considered the effect of the horizontal reinforcing bars around the openings, on the premise that they were anchored to the side columns at one end and also anchored to the vertical reinforcement at the other end. Therefore, the vertical and horizontal reinforcements around opening were modeled as the ties and they were connected to the struts which were assumed to be formed in the direction passing through the opening. In this model, the force as the resultant of stress transferred from the struts can be equilibrated with that of the stress in ties at the node. The strength of the node is calculated by the CTT node strength of the Strut-and-Tie model of ACI Code.
3). S. Popovic's concrete constitutive relationship was used to solve the multiple macro model shown in Fig. 3(a), while the Vecchio-Collins concrete constitutive relationship was used instead in the modified multiple macro model shown in Fig. 3(b) . The concrete tensile stress has not been considered in both the macro models.

3.3. Analysis Details

The side columns of all specimens were divided into 5 segments, and the compressive struts were arranged in the form of strip elements having the width of about 27cm. The dimensions of the rigid elements near the opening was modified for considering the effect of the reinforcing bars around openings and the difference of the opening ratio as shown in Fig. 4. The material constants were determined according to the measured mechanical properties of materials. The mechanical properties of axial spring, the shear spring, and the reinforcement tie were determined in accordance with the original macro model by Takehara et al. However, the concrete constitutive law based on the Vecchio-Collins's model was modified as follows.

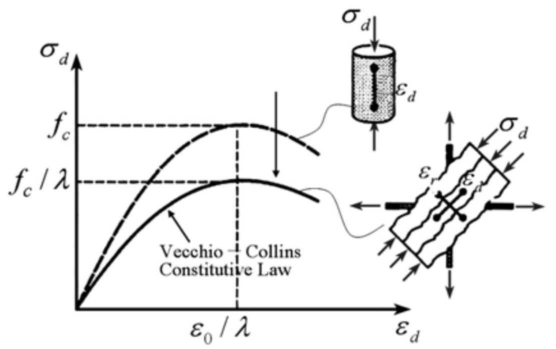

Figure 5 Concrete Constitutive Law

Ascending branch: $\varepsilon_d \le \varepsilon_0/\lambda$

$$\sigma_d = f_c\left[2\left(\frac{\varepsilon_d}{\varepsilon_0}\right) - \lambda\left(\frac{\varepsilon_d}{\varepsilon_0}\right)^2\right] \tag{9}$$

Descending branch: $\varepsilon_d > \varepsilon_0/\lambda$

$$\sigma_d = \frac{f_c}{\lambda}\left[1 - \left(\frac{\varepsilon_d/\varepsilon_0 - 1/\lambda}{2 - 1/\lambda}\right)^2\right] \tag{10}$$

Where ε_0 is strain corresponding to the concrete maximum compressive stress f_c, and is assumed to be 0.002. ε_d is strain corresponding to the concrete compressive stress σ_d. λ is the constant proposed by Hsu when applying to the wall without opening and can be expressed by $1/\cos\alpha$, where the angle α was assumed to be 45 degree based on the observed damage of the specimens. Stress-stain relationship of reinforcement was expressed by a bi-linear model where the tangential stiffness in the post-yield stage was assumed to be 1% of the initial stiffness. However, the compressive stress induced in the reinforcement was neglected. As the boundary condition, all of the nodes connected to the load cells were treated as pin supports.

The load-deflection analysis was conducted with a horizontal displacement increment of ΔD=0.5mm. As the first step, the assumed long term axial loads of 400kN for S1 and 244kN for M1 and L1 were applied to each of the side columns as shown in Fig. 3. For further loading, these axial loads were varied with pitches of ±0.42kN per 1kN of the horizontal load change in accordance with the increase of lateral displacement. The horizontal force was applied to the center of the loading beam in the same manner as the actual testing.

3.4. Analysis Result

The relations of lateral load versus drift angle for S1, M1 and L1 are shown in the Fig. 6. Figure 7 shows the deformation at the maximum lateral load and the damage of M1. The relations of the stress of the main strut versus drift angle for M1 shown in the Fig. 8. Table 5 shows the comparison between the maximum strengths obtained from the three models and the experimental results.

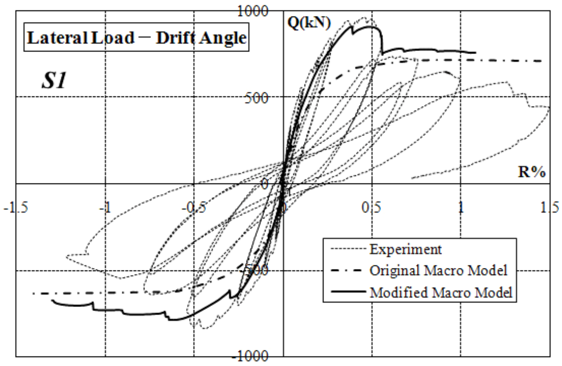

Figure 6a Lateral Load-Drift Angle (S1)

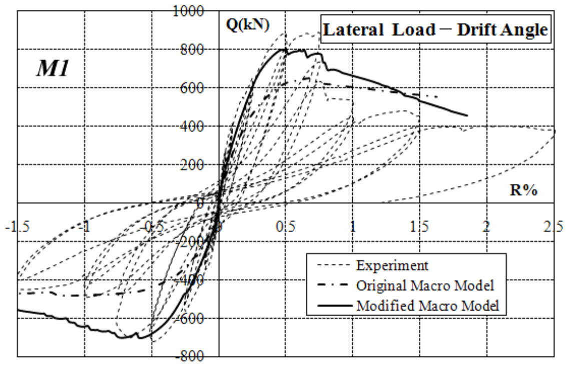

Figure 6b Lateral Load-Drift Angle (M1)

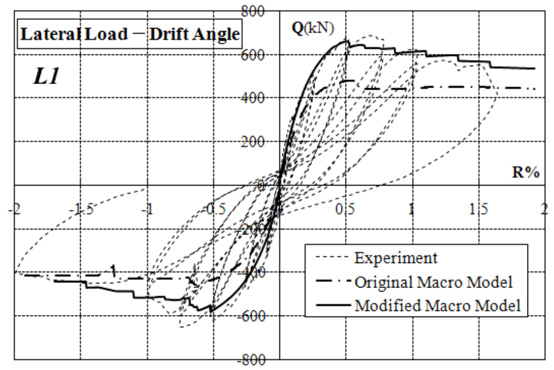

Figure 6c Lateral Load-Drift Angle (L1)

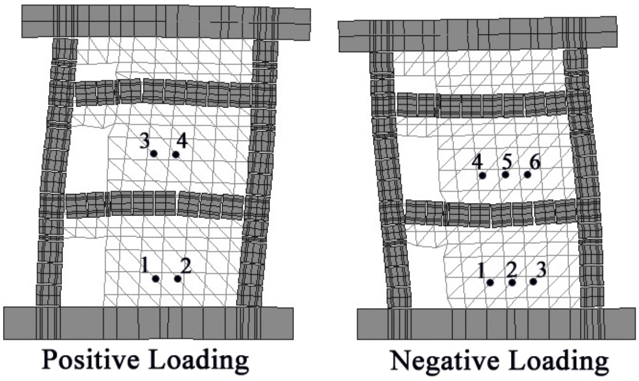

Figure 7 the Deformation and the Damage (M1)

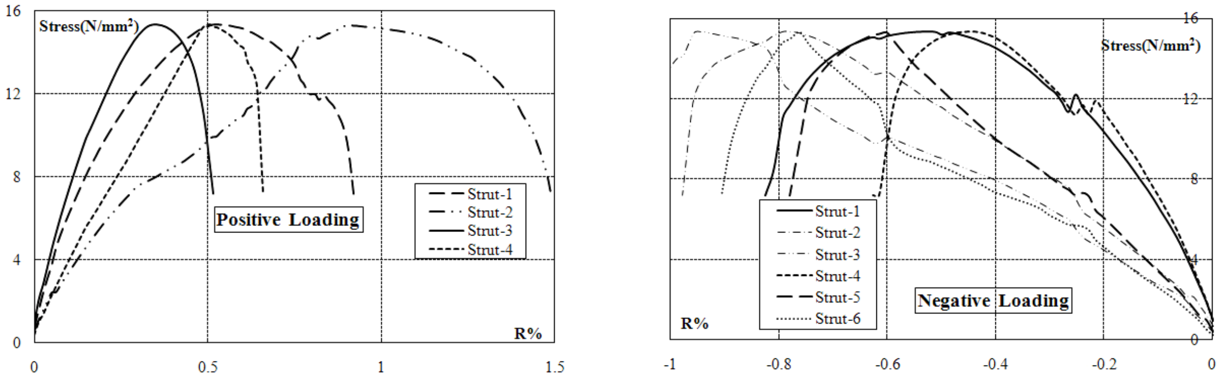

Figure 8 Main Strut Stress-Drift Angle (M1)

Table 5 Maximum Strength of Experiment and Macro Model

Specimen	Positive Direction			Negative Direction		
	Experimental Value(kN)	Takehara Model(kN)	Modified Model(kN)	Experimental Value(kN)	Takehara Model(kN)	Modified Model(kN)
S1	967	719 (0.743)	913 (0.944)	-838	-634 (0.804)	-791 (0.944)
M1	889	649 (0.730)	799 (0.899)	-723	-485 (0.671)	-704 (0.974)
L1	686	478 (0.696)	661 (0.963)	-649	-455 (0.771)	-582 (0.897)

Note: the value in the parenthesis is the maximum strength ratio of the models to experimental value.

The analytical results by the modified multiple macro model agree well with the experimental results of specimen S1, M1 and L1 in both positive and negative lateral loading. The computed strengths by the original models are much smaller than the experimental values in positive and negative loadings. This is because the vertical/horizontal reinforcements around the openings were neglected. On the other hand, the computed values by the modified multiple macro model corresponded well with the experimental observation where the strength in the positive loading was smaller than that in the negative loading. Moreover, the modified model predicted well the behavior of the specimens in the post-peak regions, although the computed value is a little bit bigger than the experimental value. This is because the strength deterioration of concrete struts were comparatively prompt due to the cyclic reversed loading in the actual test specimens, while the analysis was conducted in the manner of one way push-over.

In the case of M1 (see Fig. 8), at the drift angle of about ±0.5%, the concrete of the main strut yielded and the load carrying capacity started to decrease. However, any sudden decrease is not observed because of the stress redistribution. This analytical result corresponded well with the post peak behavior observed during the experiment. In the cases of S1 and L1, the compression struts of the first story yielded and the strength started decreasing at the drift angle of ±0.5%, but the decrease was not so rapid as the experiment. In the analysis conducted using the modified model, horizontal wall reinforced ties above the opening of the first and the second stories yielded, but other horizontal reinforcements did not yield. Moreover, the horizontal reinforced ties of the third story did not yield. These calculated results corresponded well with the experimental observation except for the behavior after the shear sliding of the walls.

4. CONCLUSION

In this study, the modified multiple macro model was established by modifying the original model proposed by Tekehara et al. to obtain more accurate load-deformation relationships in the case of multi-story walls with eccentric openings. The following conclusions were reached from the study mentioned above.

1). It has been confirmed by the comparison between the analytical and experiment results that the modified multiple macro model is applicable to the structural walls even for the case of the opening ratio exceeding 0.4.
2). When the results obtained from the modified multiple macro model were compared with those from the model by Takehara et al, the modified multiple macro model was found to be more adequate.
3). The modified multiple macro model could predict well the behaviors of a wall with an eccentric opening which become different depending on the loading direction.

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