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Effects of eccentricity on behavior of a damaged R/C building during 2004 Chuetsu Earthquake

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ABSTRACT :

A four story reinforced concrete building suffered serious damage during the 2004 Chuetsu Earthquake occurred in Niigata prefecture, Japan. This building was built in 1972 and located at the center of Tokamachi city (seisimic intensity 6+). Some columns located at half side of the building failed in shear severely and the remaining seismic performance was found to be 62% of the non damaged original structure. This was supposed to be caused by rotational vibration generated by eccentrically located reinforced concrete and block walls. The objectives of this study were to examine the cause of the damage of the building using earthquake response analysis and evaluating method of seismic performance calculated according to currently used standard in Japan.

KEYWORDS: Reinforced concrete building, Chuetsu Earthquake, Eccentricity, Response analysis, Axial load capacity

1. INTRODUCTION

It is not easy to simulate seismic damage of buildings by seismic response analysis. Thinking of only highly influential factors, there are many problems such as modeling method of buildings, weight evaluation of buildings, evaluation method for strength and deformability of members, interaction between ground and buildings, and evaluation of input earthquake motion. Though many studies have been carried out to examine those affects, it is important that they are done on buildings that were actually damaged by the earthquake.
Authors reported a summary of the damage and the results of seismic evaluation upon a four-story R/C structure building (described as S building hereafter), which had been damaged by Chuetsu Earthquake (LI(2006)).In this study we examined how much damage of the S building could be evaluated by seismic response analysis. Since S building was damaged by torsion, our primary objective was to comprehend how much it was affected by eccentricity.

2. SUMMARY OF THE EARTHQUAKE MOTION AND THE BUILDING

2.1 summary of the earthquake motion

In Tokamachi city, at the time of Mid Niigata Earthquake of October 23rd in 2004, the earthquake motion was measured 6 lower on Japanese intensity scale during the main shock and 6 upper during the aftershock, which was 38 minutes after the main shock. The acceleration by K-NET strong motion seismograph of Tokamachi city was 1716 gal in NS direction, which was extremely big and reached a short period area. On the other hand, the observation at Tokamachi city hall recorded 933 gal in NS direction and 706 gal in EW direction. Damage around those sites was minor.

2.2 Summary of the building

A summary of S building is shown in Table1 and the floor plan of its 1st floor is shown in Fig.1. S building is a four-story office building completed in 1972(+ one penthouse floor). Its ridge direction (X direction) is 8 spans, span direction (Y direction) is 3 spans and it has a frame structure with earthquake-resisting wall in both directions. It has a pile foundation. Its tie hoop is Φ9@300, the average concrete strength tested by cored tested by cored test pieces was 20N/mm^2. Height of the 1st floor and 4th floor is 4.5m, 2nd and 3rd floor is 3.6m. Fig.2 shows the elevation of C Frame. In this figure the bearing walls and block walls are shown separately. There is one bearing wall between B-C on 2 Frame, another one between 8 -9 on C Frame. Those bearing walls make the center of rigidity on the C Frame side and 2Frame side, which increases the building's eccentricity. Furthermore, as shown in the elevation in Fig.2, since many block walls (100mm thick) are placed on the C Frame side and 2 Frame side, the eccentricity is increased even more and the building is twisted.

2.3 Summary of damage

The floor plan of Fig.1 shows the damage level of each member (in Roman numerals), horizontal displacement of columns, which is measured by plumb bob (numbers with arrows), and axial shrinkage of columns of 1st floor (shown as numbers in ()), which are measured by 2nd floor as level. Between 6 Frame and 9 Frame, there were many columns whose damage levels were V. Settlement of column A-8 was 64mm. Displacement of column B-9 of 1st floor was 36mm. By these accounts, we can assume that 9 Frame side of the building largely twisted toward the Y direction.

In the figure, the displacement of the 2nd floor at the time of the highest response is tinted. Estimated highest deformation of each column top at that time is also shown (numbers between {}). Those numbers were estimated by the least-squares method on displacement of the 2nd floor, which were on average appropriate to the residual deformation measured on each column of 1st floor. However, as the measured deformation was residual, we assumed that the highest response was as two times as high as the residual deformation. Looking at the tinted area, it is confirmed that 9 Frame side was largely displaced towards NS (Y direction) compared with 1 Frame side.

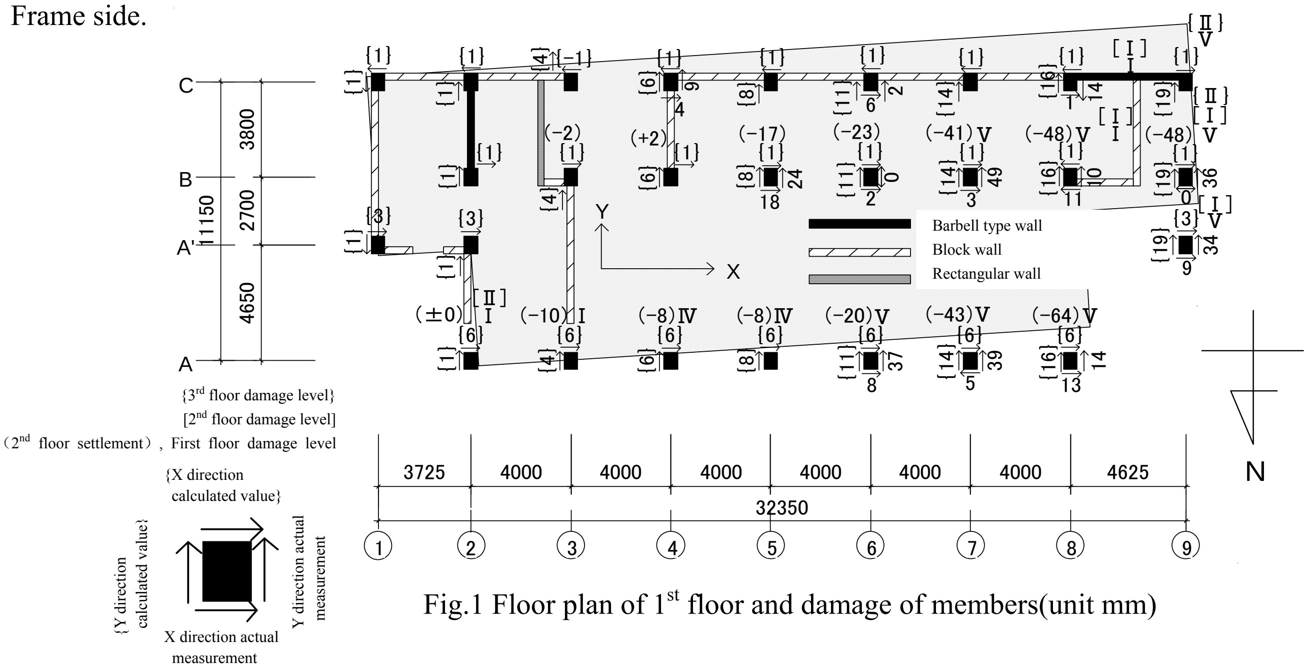


Fig.1 Floor plan of 1st floor and damage of members(unit mm)

Table-1 Summary of the building

Year completed	1972
Number of story	4(+ one PH floor)
Structure	Reinforced concrete structure
Span	8x3 span
Foundation	Pile foundation (Concrete pile)

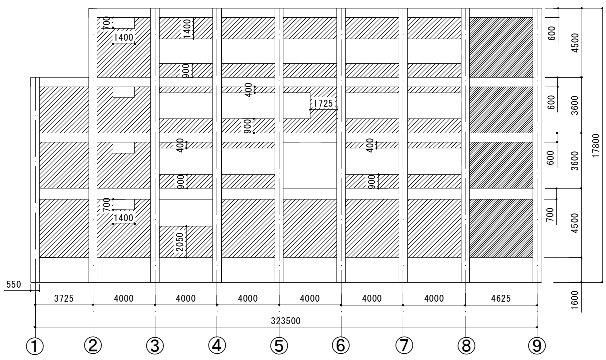


Fig.2 Elevation of C Frame

3. EFFECTS OF MODULUS OF ECCENTRICITY ON RESULTS OF SEISMIC EVALUATION

3.1 Results of seismic evaluation

Table2 shows the results of the seismic evaluation reported in Reference1). It examines both cases in which block walls were considered and not considered. Here, block walls are considered R/C bearing walls with 1/5 of their own thickness for which strength and rigidity was calculated. (In this case, block walls were 10cm.Thus they were considered as 2cm thick bearing walls). Also for evaluation of flexible length of columns, the effects of block walls were taken into consideration. The evaluation of modulus of eccentricity adopted a method based on the standard for seismic evaluation(2001), namely the method that evaluates stiffness by a cross section of a member.

The results shows that the Is value is 0.6 and over on both X and Y direction on the 2nd, 3rd, and 4th floors. For the X direction of the 1st floor,

Is=0.44, Y direction, Is=0.51. Looking at S_D index, the reduction in the 4th floor was due to the ratio of stiffness and weight. There was no reduction on other floors. This means that we were unable to simulate the building damage caused by the low modulus of eccentricity and twisting.

3.2 Discussion on modulus of eccentricity

Since Reference 2) failed to evaluate the reduction by modulus of eccentricity, we consider the method using modulus of eccentricity based on building standard law, which is an evaluation method using elasticity and rigidity of members.

Modulus of eccentricity Re is obtained by e/r. Here, *e* is eccentric distance; *r* is diagonal length of the building based on the standard for seismic evaluation. It is also stiffness radius based on building standard law. Usually, *r* based on building standard law is approximately 12 square roots of standard for seismic evaluation.

Evaluation of modulus of eccentricity by both the standard of seismic evaluation and building standard law, and results of reduction ratio of Y direction of 1st floor by the evaluation of modulus of eccentricity are shown in Table3 and Fig. 3**.** In Fig.3, the reduction ratio of Is value corresponding to modulus of eccentricity Re is shown in vertical axis. It shows the reduction by both the standard for seismic evaluation and building standard law. The evaluation result of the Y direction for the 1st floor is also shown. According to the Fig., *r* based on building standard law is about 6 times bigger than that based on the standard for seismic evaluation. From the view of evaluation of eccentric distance *e,* when it's obtained by cross sections (by standard for seismic evaluation), it seems about half of the value obtained by rigidity of members (by building standard law). Furthermore, their standards of reduction ratio corresponding to Re are not the same and give extreme difference to evaluation results as mentioned before. The standard for seismic evaluation doesn't consider columns for cross section of walls, which is required for evaluation of wall stiffness. That was the main reason for that eccentric distance *e* (by the standard for seismic evaluation) was half (by building standard law).

(a)Block wall unconsidered

Floor number	X direction					Y direction				
	C	F	SD	E0	Is	C	F	SD	E0	Is
4	0.31	2.59	0.80	0.80	0.64	0.65	1.60	0.80	1.05	0.84
3	0.39	1.60	1.00	0.63	0.63	0.49	1.27	1.00	0.63	0.63
2	0.45	1.40	1.00	0.63	0.63	0.53	1.27	1.00	0.67	0.67
1	0.35	1.27	1.00	0.44	0.44	0.42	1.20	1.00	0.51	0.51

(b)Block wall considered

Floor number	X direction					Y direction				
	C	F	S_D	E0	Is	C	F	S_D	E0	Is
4	0.68	1.00	0.80	0.68	0.54	0.78	1.00	0.81	0.78	0.63
3	0.78	1.00	1.00	0.78	0.78	0.67	1.00	1.00	0.67	0.67
2	0.39	1.40	1.00	0.54	0.54	0.60	1.00	1.00	0.60	0.60
1	0.74	1.00	1.00	0.74	0.74	0.62	1.00	1.00	0.62	0.68

Table-2 Seismic evaluation results (basic model)

Table-3 G1 and Is value by the eccentricity of 1st floor Y direction

	Modulus of eccentricity Re	G1 value	Is Value
The standard for seismic evaluation	0.052	1	0.51
Building standard law	0.317	0.67	0.34

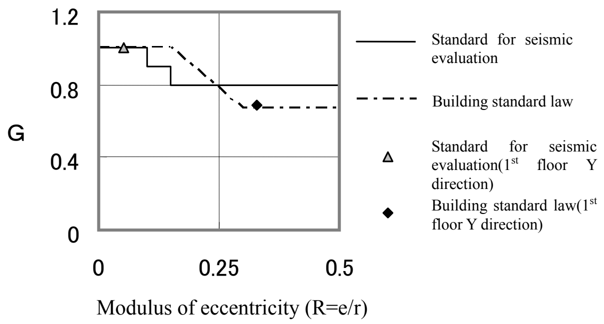


Fig.3 Relation between modulus of eccentricity and G1 value of 1st floor Y direction

4. SEISMIC RESPONSE ANALYSIS CONSIDERING TORSION

4.1 Analysis method

We analyzed seismic response considering torsion. Fig.4 shows the analytical model.

To analyze, we considered equilibrium condition of X, Y and rotating direction, replacing each frame into shear spring model, which are multi-mass systems at the position of coordinate, on both X and Y direction independently. Accordingly, its degree of freedom was 3Xn(N is floor number), correlation of XY application of stress as a member was not considered. Damping was proportional to initial stiffness and damping factor was assumed to be 5%.

Input earthquake motions were main shock and aftershock, which were based on records of strong motion seismograph, measured by Tokamachi city. Fig. 5 shows acceleration response spectrum of earthquake motion with 5% damping. The figure shows NS, EW directions of main shock and NS, EW directions of largest aftershock. According to the figure, in short period area the response of main shock is bigger. But the response of the largest aftershock is bigger around one second. To analyze, earthquake motions of EW direction were used to X direction, earthquake motions of NS direction were used to Y direction. Three cases were analyzed; main shock only, aftershock only and the composition of main shock and aftershock, leaving 10 seconds between the both cases.

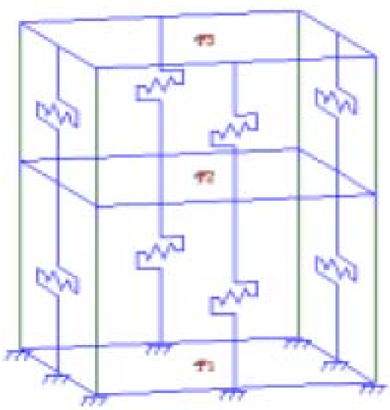


Fig.4Analytical model

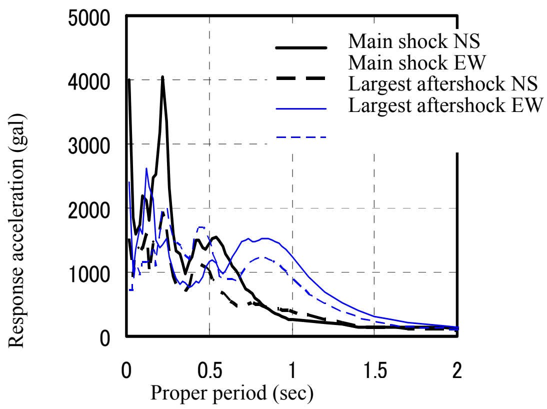


Fig.5 Acceleration spectrum of the earthquake (at Tokamachi City Hall)

4.2 Modeling of the building

Members of each floor were classified in columns, walls and block walls by each frame and restoring force characteristics were given to each member group. In Fig.4, in order to simplify, only one spring is shown in each frame. However, they have three paralleled springs, columns, walls and block walls in real. Yield strength is the ultimate strength of columns and bearing walls based on the results of seismic evaluation. As described before, strength of a block wall was calculated assumed as a 2cm thick R/C wall. Crack strength was assumed to be 1/3 of yield strength. Initial stiffness was obtained by Q-δ relationship of each frame at the time when the building was analyzed by frame model. Stiffness degrading ratio after cracking αy was evaluated by Sugano-method (empirical equation popularly used in Japan). Rigidity after yielding was given as 1/1000 of primal rigidity. As for restoring force characteristics of springs, Takeda model was used.

5. RESULTS OF ANALYSIS

5.1 Effects of block walls

Graphs of the maximum response displacement and maximum story deformation angle at center of gravity of the building are shown in Fig.6 (a)-(d). Input earthquake motions were main shocks and aftershocks. Three conditions were analyzed; block walls considered, block walls unconsidered, no eccentricity. No eccentricity is a model that walls on 2 Frame, which were the cause of eccentricity, was transferred to the center of the building (In a narrow sense, modulus of eccentricity can not be 0). As a result, on each model, response of Y direction was larger than that of X direction. Results of responses of X direction in order of ascending were in case of block walls considered, block walls unconsidered, no eccentricity. As for Y direction, the responses were almost the same in 3 conditions. That means that effects of block walls onto responses at the center of gravity of Y direction, which we focused on this time, were little. Looking at responses of each floor, it shows that the responses of 1-4 floors were almost the same level.

5.2 Effect of input earthquake wave

In order to comprehend the effects on the building by difference of input earthquake wave, we analyzed the earthquake wave in three conditions; main shock and aftershock, main shock only, after shock only using model of block wall unconsidered. Results are shown in Fig. 7(a)-(d). About responses at center of gravity, responses in the case of after shock only were significantly larger than that in case of main shock only in both X and Y directions. Furthermore, responses in case of main shock and aftershock were rather bigger than that in case of aftershock only. It tells that in case this earthquake motion is used, it is necessary to adopt the case of main shock and aftershock.

5.3 Responses of each frame

To examine effects by torsion on the building, we picked two representative frames of Y direction (wall part of 2 Frame and 9 Frame) and analyze the responses. Fig.8 shows relation between lateral shearing force and drift angle of those two frames. It shows 1-4 floor from left to right, upper four graphs are on 2 Frame, lower four graphs are on 9 Frame. In the Fig., maximum response of each member in different earthquake wave was shown.

These are results in case of block walls unconsidered. According to the figure, on both frames, responses of 9 Frame were larger than that of 2 Frame. That means the effect by eccentricity is large in all floors and displacement of 9 Frame side; west side of the building is larger.

Additionally, to confirm torsion of 9 Frame side, we would like to examine the response of each frame, including 4 Frame and 7 Frame. Fig.9 shows distribution of maximum displacement response of each frame. The response of 2 Frame was the smallest. And those are getting larger when the frame closer to 9 Frame.

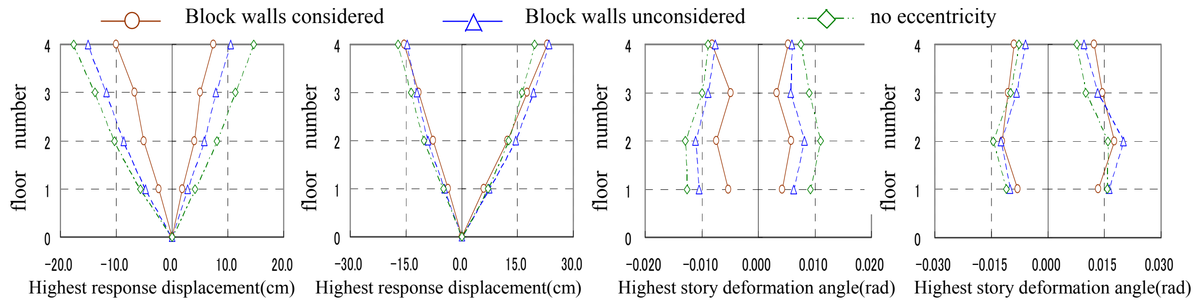
5.4. Effect by modulus of eccentricity

In this clause, we would like to examine the effect by modulus of eccentricity. Fig. 10 shows the changes in responses of the outermost frame in the Y direction because of the changing walls' position. The outermost frames of the 1st floor and 4th floor are shown (1 and 9 Frames). To change the modulus of eccentricity, four models with different wall positions were used and their responses were connected. The right extreme of the lines depict the actual condition. The left extreme shows the case of no eccentricity as depicted in 5.1. The other two points are in between. According to the Fig., the effect by eccentricity was large. As for 9 Frame of 1st floor, it was twice as big compared with in case of no eccentricity. Conversely, it was half as big as for 1 Frame. The modulus of eccentricity of the 4th floor is bigger than 1st floor due to setback. Also the response of the outermost frame by eccentricity changed the same as it did in 1st floor. However, response deformation for the 4th floor was smaller.

Fig.11 shows the relation between the distribution of maximum response displacement of Y direction on the 1st floor frame and drift angle where axial load capacity of a representative column is lost. It shows responses for the cases of block walls unconsidered, block walls considered and no eccentricity. When there was eccentricity, they show an almost straight upward line. The calculated value of drift angle where axial load capacity is lost proposed by KATO(2006)were shown as ●. At that time, we adopted the value of the column whose axial force was maximum. The values of twice the actual measurement of residual lateral displacement, which were described in 2.3 (twice the residual deformation was assumed to be the maximum deformation) were shown as ○. The response estimate value, which shows the average actual measurement described in 2.3**,** is shown in full line. The bottom graph of Fig.11 shows actual measurement values of column axial shrinkage strain.

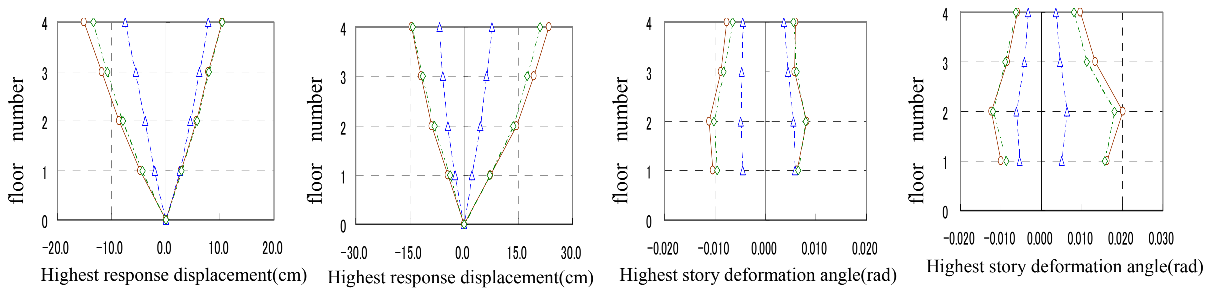
Looking at the figure, drift angle where axial load capacity is lost, which is shown as ●, is large on both sides because axis force were low. However, as for center frames, they were almost the same. In the area where calculated response value is larger than this, the actual measurement value of lateral deformation and axial deformation were large.

Also, the highest responses of 5-7 Frames where actual measurement value of lateral deformation were close to analysis result. That means that this analysis result was able to simulate behavior of S building damaged by torsion. In 9 Frame, axial load capacity was much higher than response. Though actual measurement of lateral deformation was lower than frame 7 and others, its axial deformation was large and it eventually led to axial collapse. This can be interpreted to the result of axial force concentration due to axial collapse of columns nearby.



(a)X direction (b)Y direction (c)X direction (d)Y direction

Fig.6 Effect of analytical model (maximum response displacement and maximum story deformation angle at the center of gravity in case of main shock and aftershock)



(a)X direction (b)Y direction (c)X direction (d)Y direction

Fig.7 Effect of input earthquake motions (maximum response value and maximum story deformation angle at the input of gravity at the center of gravity in case of block walls unconsidered)

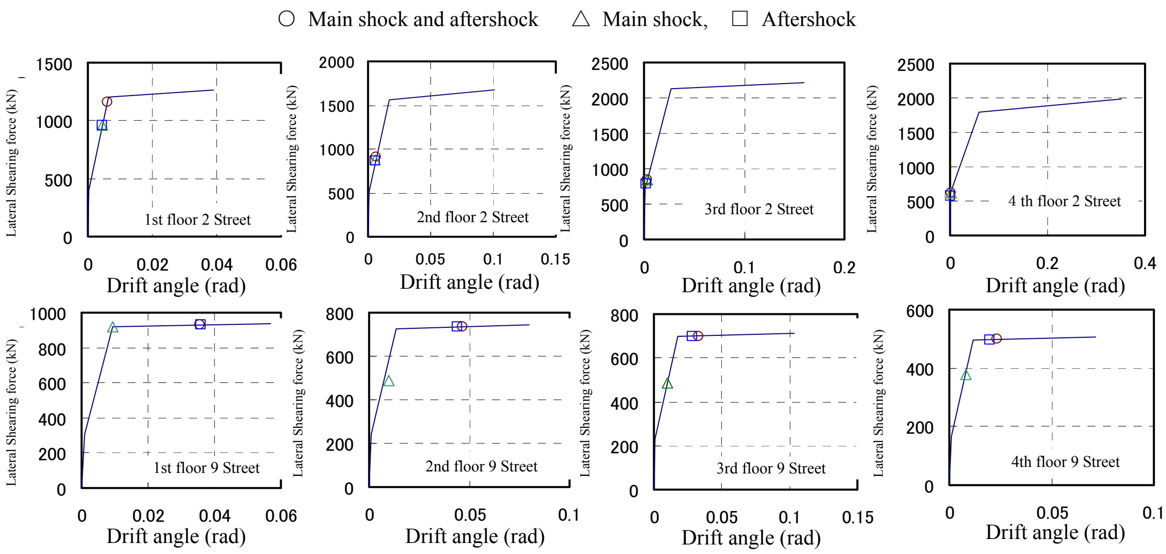


Fig.8 The maximum response of 2 Frame and 9 Frame of each floor

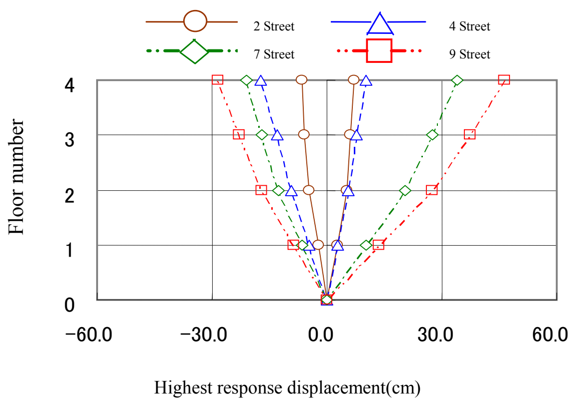


Fig.9 Maximum response displacement of each spring

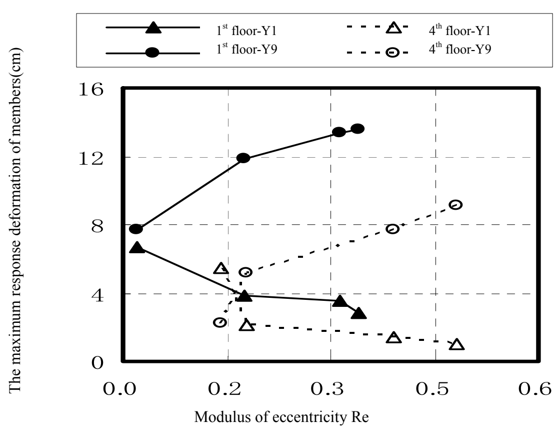


Fig. 10 Change of outmost frame response in Y direction by variation of wall position

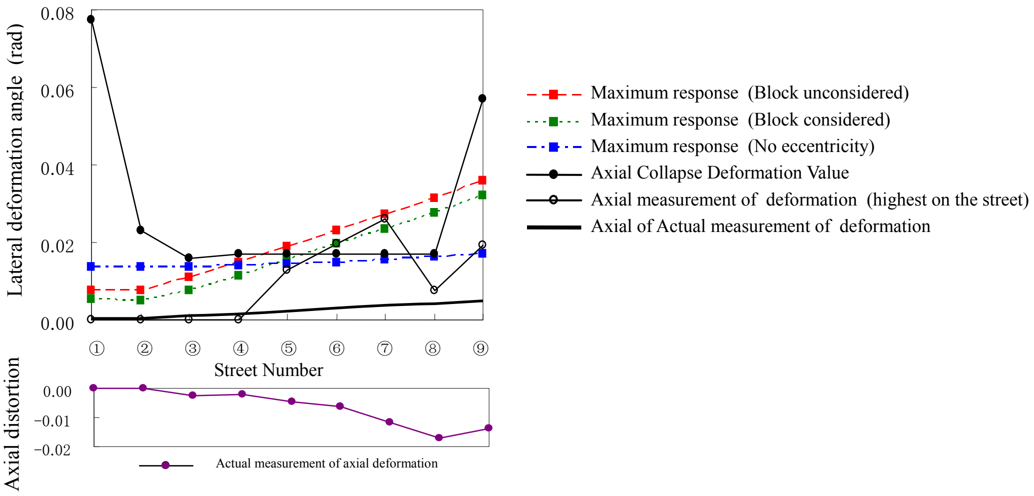


Fig.11. Distribution of maximum response of 1st floor Y direction and relations with drift angle where axial load capacity is lost

6. CONCLUSION

1) Effects of blocks on response at center of gravity in Y direction were minor.

2) As for input earthquake motion, response in case of combination of main shock and aftershock was larger than that of main shock only or aftershock only.

3) Results of responses were the same level as those in 1st-4th floor. The damage was concentrated to the 1stfloor. It is assumed that the collapsing of the 1st floor prevented damage on the upper floors.

4) On the 1st floor, in the area where the calculated response deformation was larger than the calculated drift angle where axial load capacity is lost, the actual measurement of lateral deformation and axial deformation was large. Moreover, the maximum response of the frames that had a large actual measurement value of horizontal deformation was close to the analyzed result. Therefore, the result of this analysis generally simulates the behavior of the damaged S building

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