ANALYSIS OF DAMAGE TO AN SRC APARTMENT BUILDING DUE TO THE FUKUOKA PREFECTURE WEST OFFING EARTHQUAKE

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ABSTRACT :

The non-structural walls of a 14 story apartment building of steel reinforced concrete suffered severe damage due to the 2005 Fukuoka Prefecture west offing earthquake. The cause of damage is clarified by the damage investigation results and a comparison with the analysis of response results of this building. We found that the level of damage to the non-structural walls can be evaluated with good accuracy from response analysis of a rigidity zone model and 3-way slit model, considering the seismic wave amplification from the subsurface ground. Also, deformation is particularly great in steel reinforced concrete structures, and a width of approximately 1/100 of the wall height is insufficient for the slit width in the case of using 3-way splits in non-structural walls, and approximately 1/50 is necessary. Furthermore, innovations to improve the deformation performance of non-structural walls are necessary.

KEYWORDS: Fukuoka Prefecture west offing earthquake, SRC building, Non-structural wall, Response analysis

1. INTRODUCTION

The Fukuoka Prefecture west offing earthquake hit at 10:53 on March 25, 2005. The seismic center was located approximately 27 kilometers northwest of Fukuoka City and the earthquake scale was a magnitude of 7.0. Though this earthquake devastated wooden structures in Genkai Island, reports state that damage to reinforced concrete structures and steel structures was minimal, except for some old buildings [1]. Even amongst the severe destruction seen on Genkai Island, two reinforced concrete structures thought to have been constructed using a new seismic resistant design method were left undamaged in contrast to the damage caused to wooden structures. We conducted an investigation centering in the Kego District in Fukuoka City, which suffered a large amount of damage in steel reinforced concrete apartment buildings. The damage was characterized by a focus along the Kego fault and extensive damage along the length of structures which have their long sides situated in a parallel position to the fault line. Also, damage was almost nonexistent in reinforced concrete apartment buildings ten stories and lower, and hit 14-15 story steel reinforced concrete apartment buildings the hardest. Damage to the lower portion of beam ends and non-structural walls was extensive, and hallway walls in particular had collapsed concrete areas revealing the interior of the structures. The ground around some structures sunk downwards as much as 20-30 centimeters, and deep sinking was seen particularly around structure foundations. The investigation particularly focused on a 14 story steel reinforced concrete structure in this area for which construction was completed in 1999-2000. The current study uses investigation results and design drawing to clarify the damage analysis and damage causes of the damage to this apartment building.

2. SUMMARY OF THE STRUCTURE AND GROUND

The position of the structure and a framing plan of a standard floor in the structure are shown in Figure 1 and Figure

2 respectively. The length of the structure runs parallel to the Kego fault and the structure is roughly 150 meters from the fault. The first floor consists of parking spaces and a lobby and floors two to fourteen hold residential units. The column thicknesses measured at 1300 millimeters on the first floor, 1000 millimeters on the second floor, and 650 millimeters on the fourteenth floor. The outer walling for the residential units is seismic resistant walling made of reinforced concrete. It is what is called a piloti type structure. Thus, it can be assumed that first floor columns were made particularly large to prevent first-story collapse. Also, the upper floors have very thin column thicknesses compared to the common practice of decreasing by 100 millimeters for every two stories in an upward direction. Non-structural walls on the first floor have incomplete slits, but slits were not seen from the second floor upwards (incomplete slits for these floors are marked on the design drawing, and it is possible that they are installed from the interior). The foundation uses a cast-in-place pile method with steel pipe reinforced concrete for the upper portion. The pile radius is 1500-1700 millimeters and the pile length is 43.0 meters.

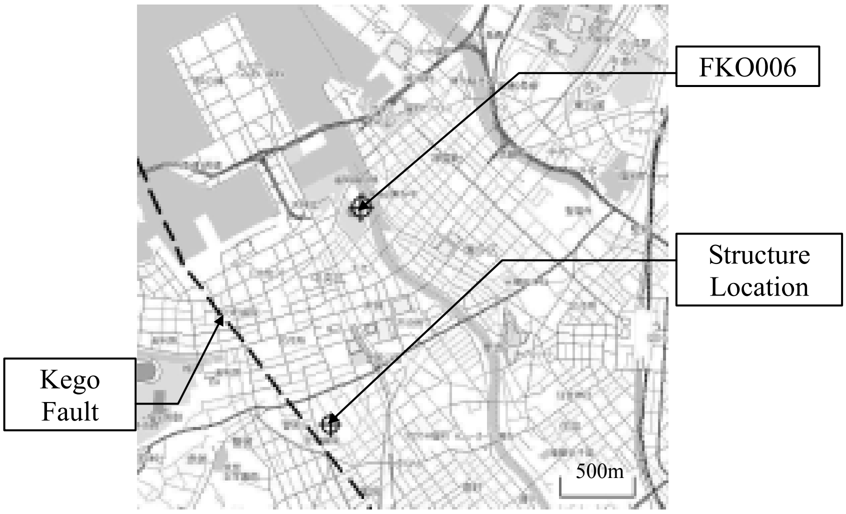

Figure 1 Position of building and observation point of FKO006

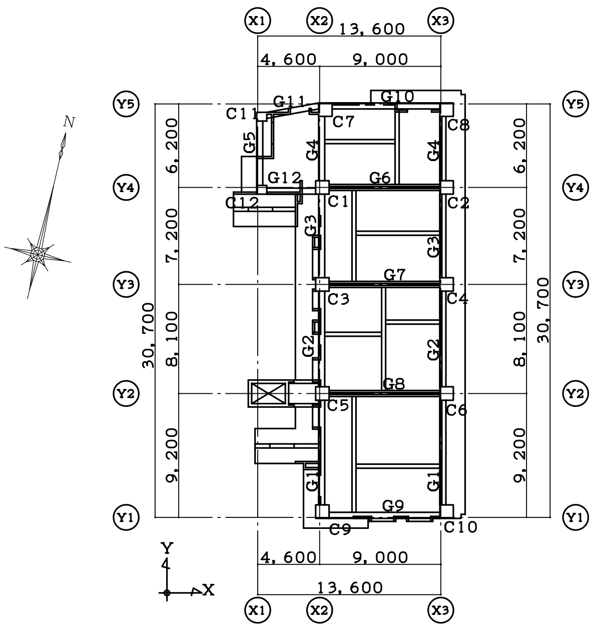

Figure 2 Framing plan of standard Floor(unit:㎜)

The ground state was assumed to be as indicated in Table 1 based on investigation materials for the surrounding area. Below is a summary of the characteristics of the structure.

- It is a piloti structure.
- The reduction in column thickness for the upper levels is high.
- The slits in non structural walls are incomplete slits.

Table 1 Ground model in FKO006 and construction site

	FKO006			Building site		
NO	Thickness m	Vs m/s	Density g/cm^3	Thickness m	Vs m/s	Density g/cm^3
1	2.0	110	1.78	2.0	102	1.80
2	6.0	130	1.76	4.9	146	1.80
3	4.0	150	1.66	6.1	204	1.80
4	3.0	180	1.94	7.0	175	1.70
5	10.0	320	1.87	7.6	203	1.70
6	—	600	1.90	14.5	262	1.80
7				3.0	293	1.80
8				—	600	1.90

3. DAMAGE STATE AND DAMAGE ANALYSIS METHOD

The state of hallway wall cracks on X2 from Y2 to Y4 in the damage investigation is displayed in Figure 3. Looking at the figure reveals several things. First of all, though damage to columns and beams was minor, damage to non-structural walls was considerable. Also, this damage was particularly prominent on the second to eighth floors where crumbling of concrete was observed. It can also be seen that damage decreased as floor number increased from the ninth floor upwards, but damage was still observed up to the highest floor. Based on the results of this investigation and the building design drawing, we conducted allowable stress calculation[2], static elasto-plastic analysis, dynamic elasto-plastic analysis of the mass system model, and dynamic response analysis of the member level, in order to analyze the damage state of this apartment building. From the fact that damage is the greatest in Y-direction non-structural walls, we conducted a comparison between the case of considering this non-structural walling and the case of considering both models with three-way complete slits and a rigid frame model, and attempted to assess the influence that the non-structural walls had on the structure. Regarding non-structural wall analysis, we considered the rigidity level and rigidity zone for the allowable stress calculation and the rigidity zone only for the response analysis.

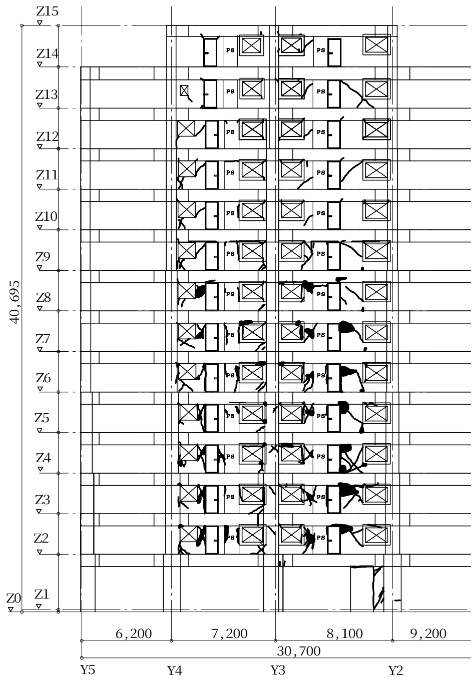

Figure 3 X2 kind of crack situation(unit:mm)

4. ALLOWABLE STRESS CALCULATION

For the allowable stress calculation, we assigned values to location coefficient Z and story-shear force coefficient Co using standard methods and defined them as 0.8 and 0.2 respectively. The natural period was T = 0.814 seconds from an approximated calculation based on the reported formula, the ground was classified as type 2 ground with Tc = 0.6 seconds, and the vibration property coefficient Rt was 0.975. As an example of calculation results, the weight for each floor, story-shear force coefficient at the time of the first design period, story-shear force, story drift, and rigidity modulus are listed on Table 2. We listed analysis results for the case of assuming three-way complete slits for all Y-direction wall areas. The base shear coefficient of each direction for the allowable stress calculation was 0.156. The maximum value for Y-direction story drift in the case of considering non-structural walling was 1/791 on the sixth floor, and the rigidity modulus was 0.708. The rigidity modulus was 0.6 or higher for each floor, and concentration of deformation was not observed. Though considerable column thickness results in a high rigidity on the piloti floors, because column thickness is significantly reduced on the upper floors, the rigidity modulus is smaller than a value of 1 on floors two to eleven in the Y-direction. The cross-section calculation results are not shown here; they were all within the allowable stress range and the allowable stress calculation results were satisfactory. By assuming 3 way complete slits, the story drift increases by 30%, but the impact on rigidity modulus is negligible.

Table 2 Result of allowable stress calculation

Current building (Considering non-structural wall)							Three-way complete slit	
Story	Wi	Σ wi	Ci	Qi	Y-direction		Y-direction	
	KN	KN		KN	Story drift	Rigidity modulus	Story drift	Rigidity modulus
14	2089	2089	0.556	1162	1/ 2282	2.040	1/ 1548	2.114
13	3498	5587	0.396	2210	1/ 1642	1.468	1/ 1024	1.398
12	4434	10022	0.328	3287	1/ 1283	1.146	1/ 766	1.047
11	4596	14617	0.291	4252	1/ 1097	0.981	1/ 655	0.895
10	4572	19189	0.266	5107	1/ 967	0.864	1/ 576	0.787
9	4612	23801	0.247	5881	1/ 878	0.785	1/ 526	0.719
8	4630	28432	0.231	6581	1/ 855	0.765	1/ 524	0.716
7	4666	33098	0.218	7214	1/ 814	0.727	1/ 501	0.684
6	4707	37805	0.206	7783	1/ 791	0.708	1/ 491	0.671
5	4754	42559	0.195	8290	1/ 813	0.727	1/ 517	0.707
4	4803	47362	0.184	8737	1/ 799	0.714	1/ 517	0.707
3	4849	52211	0.175	9122	1/ 830	0.742	1/ 556	0.760
2	4912	57123	0.165	9445	1/ 884	0.790	1/ 628	0.857
1	5188	62311	0.156	9716	1/ 1727	1.544	1/ 1419	1.938
F	11046	73357						

5. ESTABLISHING A SKELETON CURVE (Q – δ CURVE) FOR REACTION ANALYSIS IN STATIC INCREMENTAL ANALYSIS AND MASS SYSTEM MODELING

The model assumes rigid floors for each story and beam elements wherein the ends of columns and beams are rigidity zones. Curved nonlinear property was evaluated by rigid plastic rotating springs at both ends of the members (columns and beams). Bending rigidity was tri-linear and shear was elastic. The seismic resistant walling was column displacement, and the curve was evaluated by rigid plastic rotating springs of the plinths. The seismic force distribution for the load increment calculation was conducted based on Ai distribution, and we substituted this Q – δ curve into the tri-linear curve for reaction analysis. The substitution method designated the first break point and third point and set the second break point such that the floor area included up to the third point would be equivalent to the floor area included in the skeleton curve. The shear force of the first break point was set as the point where the ratio of the initial rigidity to the secant rigidity is 0.8 and the third point was set at story drift = 1/100. The tangent slope at this time was set as the third rigidity. Also, the first natural period in the Y-direction derived by eigenvalue analysis was T1 = 0.76 seconds in the case of considering non-structural walling and T1 = 0.95 seconds in the case of 3 way complete slits.

6. SEISMIC WAVES

Regarding seismic waves, we pulled the following back to the foundation using engineering techniques: three waveforms observed in El Centro 1940 NS, Taft 1952 EW, and Hachinohe 1968 NS standardized by a maximum velocity amplitude of 50 cm/s, three reported waveforms (location coefficient Z is not considered), the waveform observed by KNET in the Fukuoka Prefecture west offing earthquake (FKO006 5-1-23 Tenjin, Chuoku, Fukuoka City), and the seismic wave GENTI (FKO006 seismic waves at the building site. We then set these up at the building site and created seismic waves. In the pull back and set up, we considered the nonlinearity of the ground by an equivalent linear method and based it on one dimensional multiple reflection theory.) Table 3 displays the maximum acceleration and maximum velocity. And Figure 4 shows the acceleration response spectrum in the NS direction for FKO006 and GENTI. Figure 1 shows the location of the FKO006 observation point. This observation point is approximately 1.5 kilometers north of the structure used in the analysis and is a geological layer where sandy soil and silt has accumulated approximately 25 meters compared to the buildup of sandy soil and silt of 45 meters on the ground of the target structure. It can be assumed that this weak-layer buildup and transition of the response spectrum peak to an area with a longer oscillation time resulted in massive damage to mid to high level structures in this area.

Table 3 Adopted earthquake wave

Seismic wave	Abbreviation	Maximum acceleration (cm/s^2)	Maximum velocity (cm/s)
EL centro 1940 NS	ELCENTNS	510.8	50.0
Taft 1952 EW	TAFTEW	496.8	50.0
Hacinohe 1968 NS	HACHINS	330.1	50.0
Reported waveform-1	KOKUJI-1	310.8	89.1
Reported waveform-2	KOKUJI-2	361.6	62.4
Reported waveform-3	KOKUJI-3	256.3	96.0
FKO006NS	FKO006NS	276.5	59.5
FKO006EW	FKO006NS	239.3	32.6
Building site-NS	GENTINS	283.7	48.7
Building site-EW	GENTIEW	141.6	27.7

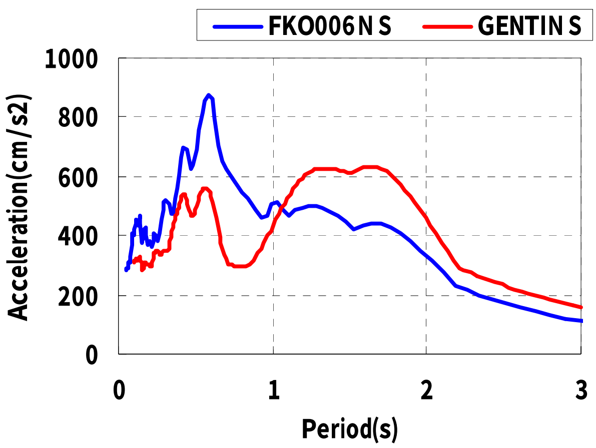

Figure 4 Acceleration response spectrum of FKO006NS and GENTINS

7. MASS SYSTEM MODEL SEISMIC RESPONSE ANALYSIS

In the oscillation model, we substituted the equivalent sheering-type fixed foundation model for 1 mass point on each level, assigned a degrading history to the skeleton curve, and set the damping constant at 3%. Damping was handled as an instantaneous stiffness proportion. Examples of response calculation results are shown on Table 4, Figure 5, and Figure 6 (TAFTEW, which had a high response level for the observed waveform, and KOKUJI-2, which had a high response level for the reported waveform, are shown). Figure 5 demonstrates that the response to seismic waves for GENTINS in the case of considering non-structural walling is large for this structure compared to the other three observed waveforms. Taking the story drift as an example, the value exceeds 1/100 on floors four to seven. Also, the base shear coefficient was 0.254. It is clear that the impact from the difference of seismic waves had a great influence on the response for each floor but negligible influence on story-shear force and overturning moment. Figure 6 demonstrates that the GENTINS seismic wave response in the 3-way complete slit case was also extremely large, displaying a response that is close to the reported waveform (Z = 1.0). Story drift exceeds 1/100 on floors four to nine with the maximum value of 1/59 on the sixth floor. However, the base shear coefficient is from 0.254 to 0.211 of the case without slits, with a result that is approximately 20% lower.

Table 4 The maximum response in direction of Y of
equivalent shearing type model (non-structural wall consideration)

Seismic wave	FKO006NS					GENTINS				
Story	Displacement	Story drift angle	Shear force	Shear force coefficient	Overturning moment	Displacement	Story drift angle	Shear force	Shear force coefficient	Overturning moment
	cm	1/	KN		KNm	cm	1/	KN		KNm
14	19.16	2739	798	0.459	2286	24.93	2598	832	0.478	2382
13	19.07	728	2203	0.451	8592	24.84	671	2307	0.472	8992
12	18.72	329	3903	0.435	19480	24.45	311	4046	0.451	20380
11	17.94	226	5579	0.422	35066	23.63	222	5647	0.427	36204
10	16.82	197	7038	0.404	54843	22.45	197	7045	0.404	55826
9	15.54	178	8157	0.376	77804	21.11	173	8169	0.376	78817
8	14.28	145	8955	0.345	102910	19.53	116	9096	0.350	104203
7	13.06	121	9878	0.324	130134	17.15	89	10130	0.334	132192
6	11.73	117	10683	0.308	159730	14.15	82	11082	0.320	162847
5	10.25	127	11498	0.294	191659	11.76	85	11946	0.306	196108
4	8.41	126	12537	0.288	226155	9.24	98	12861	0.296	232201
3	6.18	153	14030	0.292	263052	6.65	149	14132	0.294	270990
2	4.40	184	15255	0.290	302124	4.82	183	15289	0.291	311277
1	2.98	170	14581	0.254	374821	3.38	150	14594	0.254	385120

8. RESPONSE ANALYSIS OF MEMBER LEVEL

We conducted response analysis confirmation for the mass system model and a member level response analysis (we considered the restoring force characteristics of each member and created a rigidity level matrix for each member) to consider the concurrent effect of seismic waves in both X and Y directions. We set the seismic waves at FKO006 and GENTI and assigned seismic waves in the NS and EW directions simultaneously. The maximum response value in the Y-direction is shown in Figure 7. The response value in the Y-direction was larger for GENTI and the response value in the X-direction was lower for GENTI. As shown in Figure 4, this corresponds to the results of a high response acceleration for FKO006 in low wave frequency periods and a high response acceleration for GENTI in high wave frequency periods. The Y-direction story drift for GENTI in the non-structural walling case exceeded 1/150 from the fourth to tenth floor and was at or higher than 1/100 from the fourth to tenth floor in the 3-way complete slit case. However, shear force, shear force coefficient, and overturning moment were low.

9. COMPARISON OF THE ANALYSIS RESULTS AND DAMAGE STATE

In the comparison of the required ultimate strength in the ultimate strength calculation and the base shear coefficient in the Y-direction at the time of a major earthquake from the response analysis results, the required ultimate strength is Cb = 0.20 (when Ds = 0.25 and Z = 0.8) whereas all analysis values exceed 0.2 for response analysis. It is clear, then, that problems can occur when an earthquake hits an area with a low location coefficient. And in the comparison of reaction values for the mass system model and member system model, for

Y-direction story drift the mass system model is larger on the first floor while the member system model is larger on the fourteenth floor. Other floors have fairly equivalent values in comparison. However, because the seismic waves were entered simultaneously, the member level reaction value was larger than shear force, shear force coefficient, and overturning moment. This result indicates that if design is conducted using the results of a single-directional seismic force, the bi-directionality of seismic force must be considered. Taking the equivalent sheering-type model as an example, story draft is 1/150 and above for floors three to eight in the case of considering non-structural walling and 1/150 and above for floors two to ten in the case of 3-way complete slits. However, the non-structural walling of the structure targeted by this study uses single reinforcement distribution (D10 @ 150) that has no core portion and the structure does not have a drift capacity of a level that can compliment this sort of large story drift. It is conceivable that this led to the formation of large cracks and crumbling and collapsing of concrete [3]. Comparing the case of and non-structural walling and 3-way slits for the maximum base shear coefficient value of the mass system model yields Cb = 0.25 for non-structural walling and Cb = 0.21 for 3-way slits. Likewise, maximum story drift value yields 1/82 and 1/51, respectively. This data indicates that using slits lessens the operating shear force but increases the drift, and that as long as the drift capacity is sufficient, load capacity can be low. Interpreting these results in a comprehensive manner reveals the following. First, response was as per a model considering non-structural walling at the initial stage in the Y-direction. However, non-structural walling that could not accommodate deformation when such deformation became great crumbled, giving a response that approximates the 3-way slit model. Moreover, the response spectrum map shows that a reduction in the acceleration response spectrum due to a decrease in rigidity level did not occur, and an even further increase in deformation occurred as seen in Figure 5 and Figure 6. However, from the fact that damage to columns was negligible, deformation approximating 1/150 seen in the analysis results from the 3-way slit model cannot have occurred. That is to say, it is likely that the damage to non-structural walling in this area contributed effectively to energy absorption, displayed a responsive condition lying between the non-structural walling model and 3-way slit model, and reduced impact to the columns and beams. This damage to non-structural walling can also be thought of as being effective in reducing story drift to the overall structure [3].

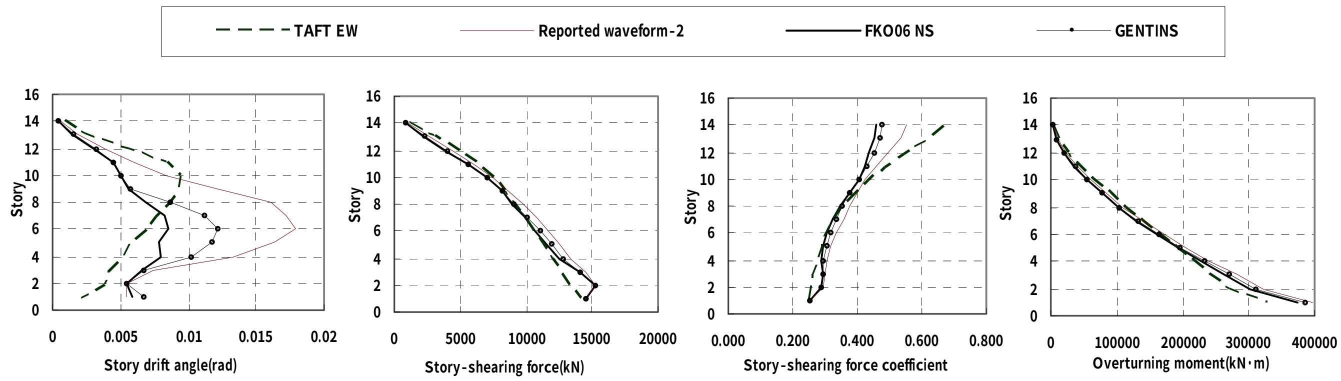

Figure 5 The maximum response in direction of Y of equivalent shearing type model (non-structural wall consideration)

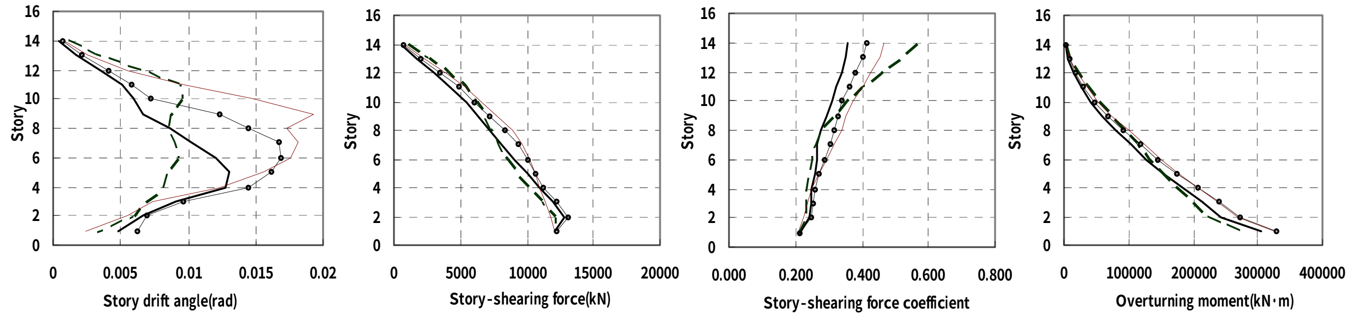

Figure 6 The maximum response in direction of Y of equivalent shearing type model (Three-way complete slit)

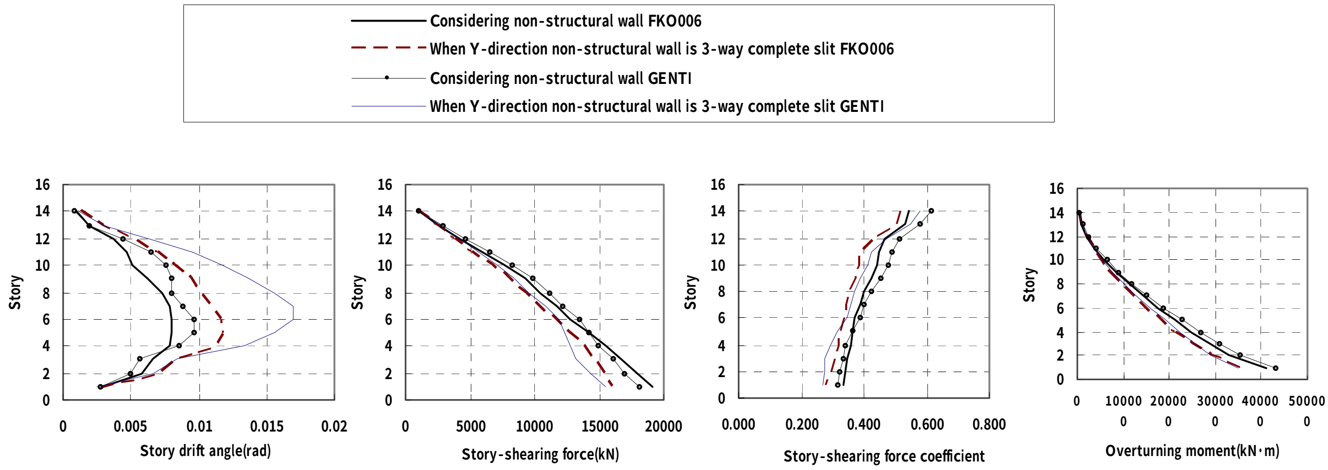

Figure 7 The maximum response in direction of Y of member model type

10. CONCLUSION

Good convergence with damage investigation results showed that an accurate seismic responsive condition can be assessed through analysis of a non-structural wall rigidity zone model and 3-way slit model. It was also shown that considering the relation with the surrounding ground is essential in order to evaluate the damage to non-structural walls in high-rise steel reinforced concrete apartment buildings in the target area. In order for the overall structure of a steel reinforced building to exhibit its retention horizontal load capacity, in most cases story drift must be 1/50. And in order for that to occur, the non-structural walls must be arranged so as to accommodate that level of deformation. Research must be pursued for methods of assuring that the non-structural walls do not influence the deformation of the building frame and methods of increasing the deformation capacity of the non-structural walls.

11. ACKNOWLEDGMENTS
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