SEISMIC PERFORMANCE ESTIMATION BY RESPONSE ANALYSIS OF A BUILDING DAMAGED IN THE 2016 KUMAMOTO EARTHQUAKE

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Abstract

In the 2016 Kumamoto earthquake, a reinforced concrete building near the epicenter was damaged and investigated in detail. The seismic motion was recorded by two stations: one located on the 1st floor of this building, and the other by station (KMMH16) by NIED strong-motion seismograph networks called KiK-net located in field of a nearby park about 500 meters away. Furthermore, the structural drawings, material strength (concrete, steel), damage situation of piles, and details of the soil condition are obtained. In other words, we have data of not only the damage of the building but also the seismic load inputs, the 1st floor response records, the soil and pile condition in the ground, and the structural drawings of this building that have experienced a major earthquake. The purpose of this study is the estimation of the damage by the simulation of the earthquake response analysis. A lumped-mass model is applied to investigate the damage of the building, the damage of the piles, and the response record of the 1st floor. The analysis is arranged as follows: First, in order to model each floor, the seismic performance of the building was evaluated from non-linear pushover analysis using the drawings and material test results. Second, the sway rocking behavior and damping characteristics of the soil and piles were evaluated from the tremor observation record. Third, the input seismic motion was predicted from the observation record (KiK-net) and the soil condition.

From the above investigations using response analysis, followings would be the main results: First, the horizontal response drift angle of the building is consistent with the drift angle expected from the damage investigation. Second, the 1st floor response matches the earthquake observation record on the 1st floor in this building. Third, the response of the sway behavior is consistent with the displacement expected from the damage investigation of the piles and foundations.

Therefore, the following conclusions were obtained: Assuming the static loading distribution and considering the slab effective of one-meter width to evaluate ultimate bending moment of beam, concrete material compression test results, and rebar strength of 1.1 times yield strength underestimate the actual stiffness and strength of the building. The maximum acceleration, envelope, and response spectrum of seismic observation records on the 1st floor could be roughly estimated from the response of the 1st floor by this analysis. The relative horizontal displacement between the building and the engineering bedrock due to the damage of the pile head and the soil could be roughly explained by adding a sliding behavior to the sway spring.

Keywords: Seismic performance estimation; R/C building; Damage evaluation; Observed record; 2016 Kumamoto EQ

1. Introduction

Many buildings were damaged in the 2016 Kumamoto Earthquake. A reinforced concrete government building shown in Photo 1, which was built in 1980 and retrofitted in 2012, was also damaged, making it difficult to continue to use. For this reason, a detailed damage investigation of this building and the foundations [1],[2],[3] was conducted. In this paper, we try to estimate these damages by the earthquake response analysis.

Photo 1 –Retrofitted R/C building damaged by the 2016 Kumamoto EQ

In the foreshock that occurred at 21:26 on April 14th and the mainshock that occurred on April 16th at 01:25, the maximum seismic intensity “7” was recorded on the 1st floor in this building. Also, the nearby park located about 500 meter north from this building has an observation station (KMMH16) of the strong-motion seismograph networks by the National Research Institute for Earth Science and Disaster Resilience[4], and a seismometer is installed on the ground and under the ground GL-197m, and the acceleration record has been observed.
In addition to the structural drawings at the time of construction and seismic retrofit and the ground boring data, there are the detailed damage investigation report of the structure and foundation piles.
To estimate the damage and compare with the 1st floor response acceleration, a static push over analysis and a seismic response analysis are conducted. The response story deformation angle, the response deformation of piles, and the response acceleration at the 1st floor are compared with the damage situation and observation records at the 1st floor. Then, the relationship between the seismic behavior of the building and the performance expected by the Japanese design standard and guidelines is clarified.

2. Outline of the building

2.1 Structural Outline

The target R/C building is a government building built on a slope ground in Mashiki town, Kumamoto prefecture and has three floors, and the plan is about 60m width in the x-direction (east to west) and about 28m width in the y-direction (north to south). The first story height is 4.5m and the second and third are 3.8m respectively. The basement and the 1st floor structural plan are shown in Fig. 1, showing the seismic record observation point and the placement of the seismic retrofit. A structural elevation is shown in Fig. 2 and 3.

The main column size of this building is 600mm x 750mm. The x-direction beam is mainly 400mm width and 700mm depth, and the y-direction beam is mainly 400mm width and 900mm depth.

A pre-cast concrete pile foundation (pile diameter of 400mm, pile length of 26m to 32m, long-term allowable support strength of 500kN per one pile) is arranged by four to six piles per one column.

In 2012, seismic retrofit by the pre-cast R/C outer frame in the south facade and the adding wall in the existing frame have been installed. The dimensions of the column of the pre-cast R/C outer frame is 900mm x 650mm. A seismic slit to improve the ductility of column constructed between column and the spandrel wall in Y4 frame. The pile foundation under the outer precast reinforcing R/C frame is a steel pipe pile constructed by torque (pile diameter of 318.5mm, wing diameter of 637mm, length of 27m) is arranged two piles per one column.

2.2 Seismic evaluation results calculated in 2012 retrofitted

The seismic evaluation at the time of the seismic retrofit in 2012 had been conducted. The results of the seismic evaluation in x-direction after seismic retrofit are shown in Table 1. In the seismic evaluation, the second level screening procedure [5] for evaluating the ultimate shear strength and ductility of the vertical member has been adopted. The seismic demand index I_{S0} is 0.7 (U=1.25, Z=0.9) in this government building.

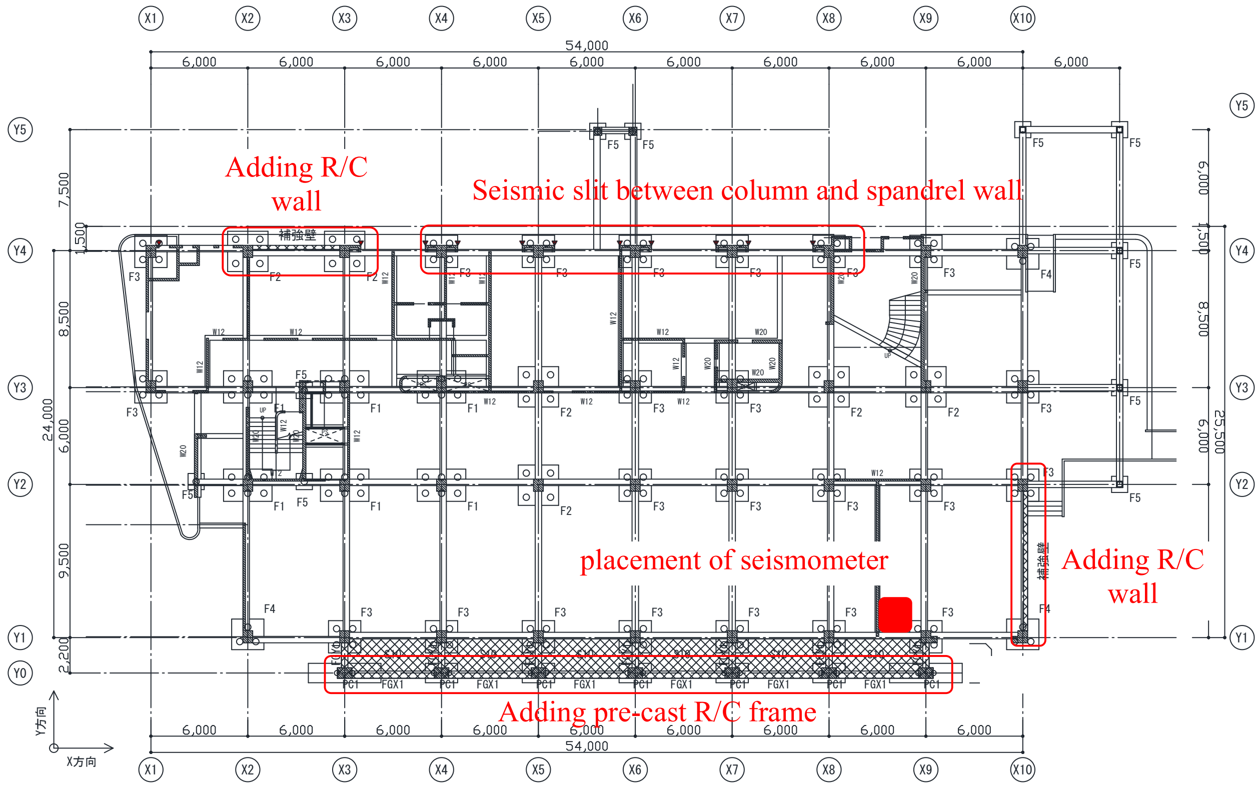


Fig. 1 – first floor and foundation stractural plan

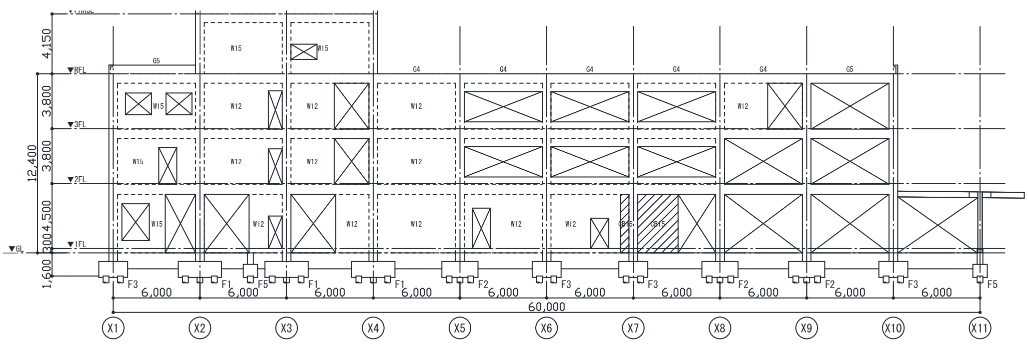


Fig. 2 – x-direction (east to west) Y3 frame structural elevation

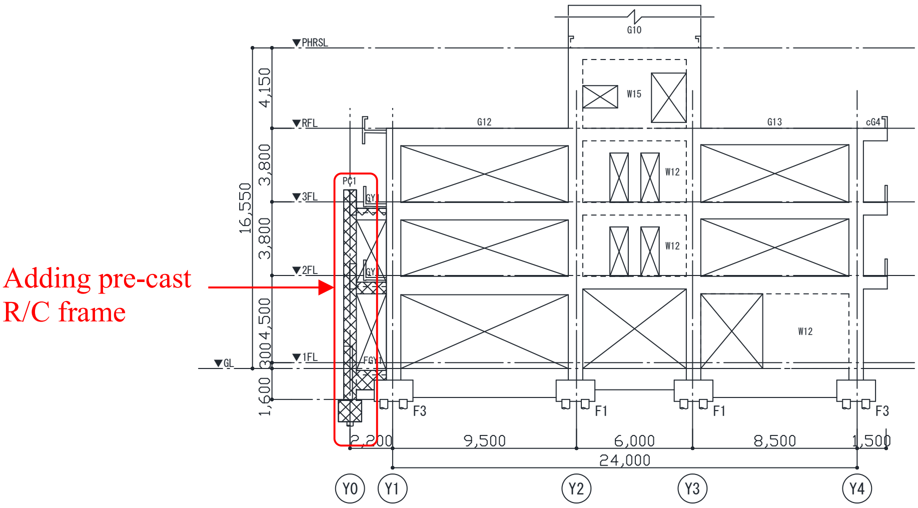


Fig. 3 – y-direction (south to north) X4 frame structural elevation

Table 1–Seismic capacity evaluation result when retrofitted in x-direction (EW)

floor	ΣW_i	$1/A_i$	W_i/A	C	F	E_0	S_D	T	I_S	$C_{TU}S_D$
3	15952	0.71	12.3	1.34	1.0	0.95	1.00	0.98	0.92	0.95
2	31943	0.85	12.7	0.91	1.0	0.77	1.00	0.98	0.76	0.77
1	50805	1.00	12.7	0.75	1.0	0.75	1.00	0.98	0.73	0.75

The concept of each Index of seismic evaluation [5] is as follows;

The seismic Index of structure, $I_S = E_0 \times S_D \times T$

Where,

$E_0=(1/A_i) \times C \times F$=Basic seismic index of structure

A_i=Story shear force distribution factor, S_D=Irregularity index, T=Time index

$C=Q_u/\Sigma W_i$=Strength index, Q_u=Ultimate lateral load-carrying capacity of the vertical members

ΣW_i=The weight of the building (kN), A=Area of floor (m^2), F=Ductility index

The seismic demand index of structure, $I_{S0}=Z \times G \times U \times E_s$

Where,

Z=0.9= Zone index, G=1.0=Ground index, U=1.25=Usage index

E_s=0.6=Basic seismic demand index

3. Earthquake damage investigation

3.1 Damage situation of the building above ground

Since the Kumamoto Earthquake occurred on April 16th, 2016, damage investigations of this building have been carried out several times. Photo 2 taken on May 2017 shows damage to the spandrel wall of the north Y4 frame, and Photo 3 shows cracks in the seismic retrofit precast R/C outer frame of the south frame. Photo 4 taken in February 2018 shows damage to wall girder above the opening of the wall after removing the finish in the building. Photo 5 taken in two years after the earthquake shows the damage of precast R/C pile and buckled steel pipe pile confirmed when foundation excavated.

3.2 Damage assesment of structure

In each floor of the building, the damage class (0 I II III IV V) at the first floor was determined according to the damage classification [6] and shown in Fig. 4. Damage class I is defined as some cracks within 0.2mm width observed, and damage class IV is defined as many heavy cracks which is larger than 2mm width observed and reinforcing bars exposed due to spalling of the covering concrete. Here, the damage classified after removing the finishing. The results of detailed damage assessment in the x-direction of the 1st Floor judged as severe damage (seismic residual capacity R equal to 57.8%,) and the result of the y-direction of the 1st Floor judged as minor damage (seismic residual capacity R equal to 82.7%). In addition, the original structure befor retrofit damaged more than the seismic retrofited structure.

3.3 Damage assessment of the foundation

Most of the damage of the foundation concentrated at piles. The foundation tilted greatly toward to the north direction due to the failure of the pile (damaged at the pile head and/or the pile intermediate part).

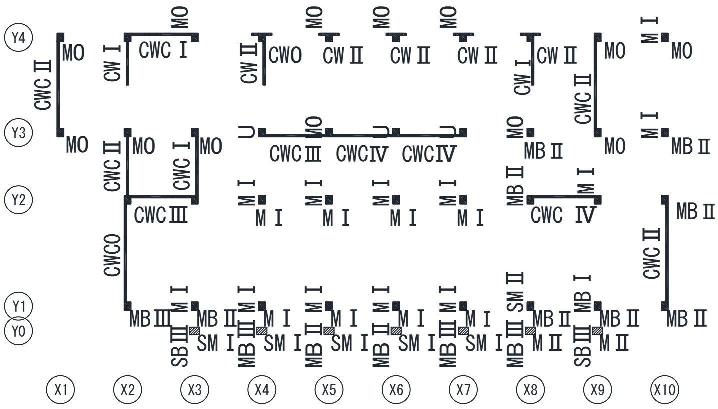
The maximum relative subsidence of the foundation was 194mm (X1-Y4 footing). The maximum inclination angle in the north-south direction was 0.87% between Y2-Y3 in X10 frame, and the maximum inclination angle in the east-west direction was 0.70% between X1-X2 in Y3 frame. As a result of these, the damage class to the pile foundation classified as a severe damage [6].

Photo 2 – Tile damaged at north Y4 frame spandrel wall

Photo 3 – Crack of connection beam between adding precast frame and existing R/C frame

Photo 4 – Shear failure of wall girder above the opening of wall at Y3 frame of x-direction

Photo 5 – Failure of precast pretensioned concrete pile and buckling of steel pipe pile



[Notes of failure mode]
S;shear column
SM; flexulural shear column
M; flexural column
SB;beam failure at shear column
MB; beam failure at flexural column
W; wall without column
CW;wall with one side column
CWC;wall with both side columns
U;unknown, 0,I,II,III,IV,V;damage class
I(<0.2mm crack width), II(0.2-1mm)
III(1-2mm), IV(>2mm), V(Buckling)

Fig. 4–Damage level at the 1st story

4. Static push over analysis

4.1 Structural model of the building above ground

The static push over analysis model is a three-dimensional model. Each floor is modeled as a rigid floor, and the pile position is a pinned support, and the columns, beams, and walls are modeled as beam elements. The modeling was based on the structural drawings and seismic retrofit report. The structural calculation software "BUS-6 Ver.1.0.9.2" (KOZO SYSTEM INC.) was used for analysis. The concrete compressive strength used for analysis was based on the seismic diagnosis report. In the stiffness and strength of beams, slab effective width and slab bars within one meter was considered following to the Japanese design guideline. Push over analysis was conducted maintaining the shear strength of the shear failure member.

4.2 Results of the static push over analysis

The relationship between story shear force and drift angle of x and y-direction is shown in Fig. 5. Further, although the push over analysis was carried out by maintaining the shear strength of the shear failure member, in the figure, the subtracted story shear force is shown. The story deformation angle at 1/250 is also shown because the ultimate lateral load-carrying capacity (Q_u) was determined at 1/250 for shear failure.

In Table 2, the ultimate lateral load-carrying capacity (Q_u) compared with the demand strength was shown.

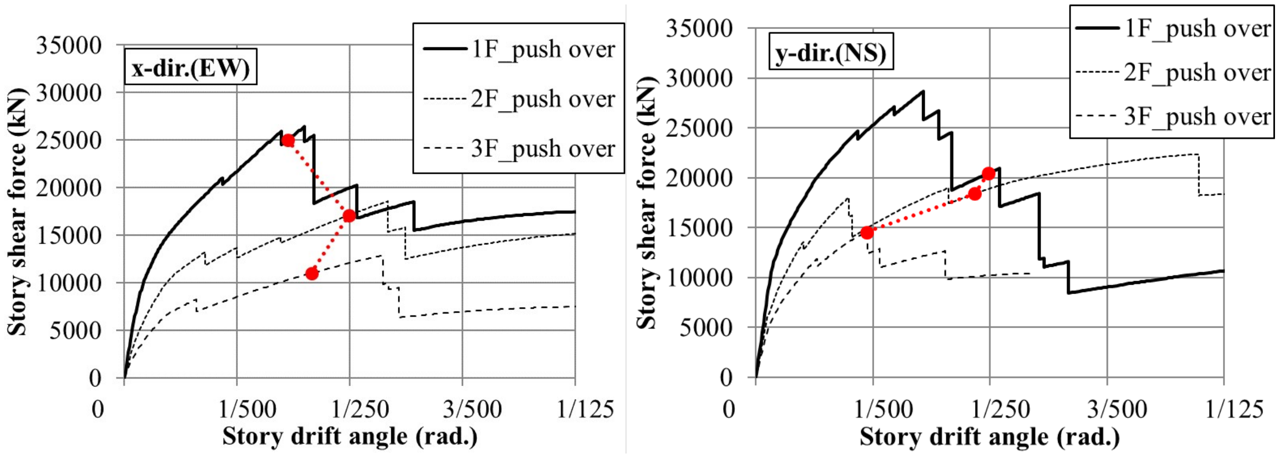


Fig. 5 – Story shear force and drift angle of x and y-direction

Table 2 –Ultimate lateral load-carrying capacity in x-direction

Story	Demand story shear strength				Ultimate story shear strength	Capacity	Relative angle (rad.)
	Q_{ud} (kN)	F_{es}	D_S	Q_{un} (kN)	Q_u (kN)	Q_u/Q_{un}	
3rd	20357.7	1.000	0.50	10178.8	11842.0	1.16	1/300
2nd	33553.1	1.000	0.50	16776.5	18474.6	1.10	1/249
1st	45050.5	1.000	0.50	22525.3	24752.6	1.09	1/369

$Q_{ud} = A_i \times C_0 \times Z \times SW$
$Q_{un} = F_{es} \times D_S \times Q_{ud}$
Where,
A_i=1.0(1st story) = the story shear force distribution factor of an earthquake
Z=0.9=the zone factor
C_0 =1.0=the basic shear force coefficient
F_{es}=1.0=the irregularity coefficient about stiffness and eccentricity
D_S=0.3~0.55=demand factor determined by the failure type of members from 0.3 for ductile frame with high ductility to 0.55 for wall frame with low ductility at 0.05 intervals.

5. Seismic response analysis

5.1 Modeling

The response analysis model is a one-dimensional multi-mass model, using software "SNAP Ver.7.0.0.9 (KOZO SYSTEM INC.). The specifications of the analysis model are shown in Table 3, and the analysis model is shown in Fig. 6. The natural period of each model is shown in Table 4 and matched to the microtremor observation.

The story spring characteristics was modeled as a tri-linear model based on the push over analysis, and Takeda model [7] (gamma; 0.4, delta; 0.7) for their hysteresis loop was applied. The internal viscous damping was 3% and proportional to instantaneous stiffness. From the microtremor observation and building mass, the sway spring was evaluated as 2,860 kN/mm, and a linear elastic dashpot 97.7 kN/(mm/s) arranged in parallel. The rocking spring was evaluated as stiff enough not to rotate.

"FX" model has fixed support at the ground level and "SR" model has the sway rocking spring between the foundation and the soil. The story spring of FX10 was modeled based on the push over analysis and those of FX15 has 1.5 times stiffer and stronger spring than FX10. And "V07" of SR15V07 has the inline spring with sway spring, and it yields at 0.7 times total weight of the building.

5.2 Input ground motion

Input ground motion for the response analysis is shown in Table 5. Observed records from two stations in two days and analysis records by response analysis considering ground surface amplification using ground boring data and engineering bedrock (GL-192m) observed record were input as seismic ground motion.

The relationship between the acceleration and displacement response spectrum using 5% damping of the motion recorded on April 14th are shown in Fig. 7.

Table 3 – Specifications of analytical models

Floor	Story height (mm)	Height (mm)	Mass (ton)
PH	4200	17120	174
RF	3830	12920	1452
3rd FL	3815	9090	1593
2nd FL	5275	5275	1886
1st FL	-	0	2574

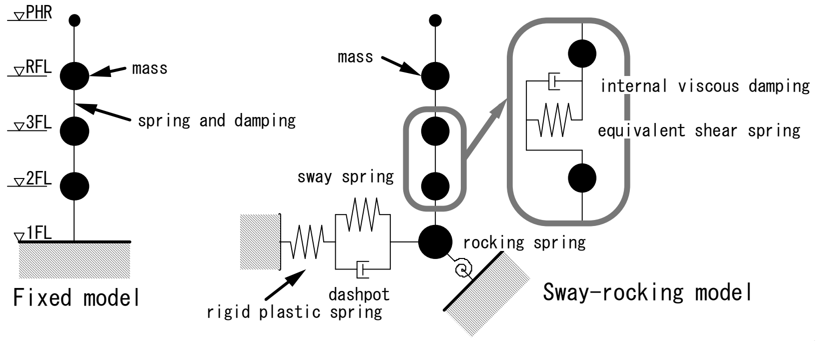


Fig. 6 –analysis model (“FX” and “SR”)

Table 4 – natural period of each model (unit; second)

	x-direction				y-direction			
mode	FX10	FX15	SR15	SR15V07	FX10	FX15	SR15	SR15V07
1st	0.229	0.187	0.359	0.360	0.213	0.165	0.352	0.353
2nd	0.094	0.076	0.117	0.117	0.089	0.065	0.101	0.101
3rd	0.062	0.050	0.067	0.067	0.059	0.043	0.058	0.058

Table 5 –Input motion maximum acceleration of EW(x-dir.) and NS(y-dir.)

Input motion	Month/Date	EW max. acc.(cm/sec^2)	NS max. acc. (cm/sec^2)	Time(second)
Observed record	KMMH160414	694.8	570.0	60
	KMMH160416	1156.7	651.8	60
	MIYA0414	731.8	631.5	60
	MIYA0416	825.4	775.5	60
Analysis record	GL1_4/14	640.1	509.1	60
	GL1_4/16	876.8	619.7	60
	GL4_4/14	825.2	626.8	60
	GL4_4/16	1182.0	780.8	60

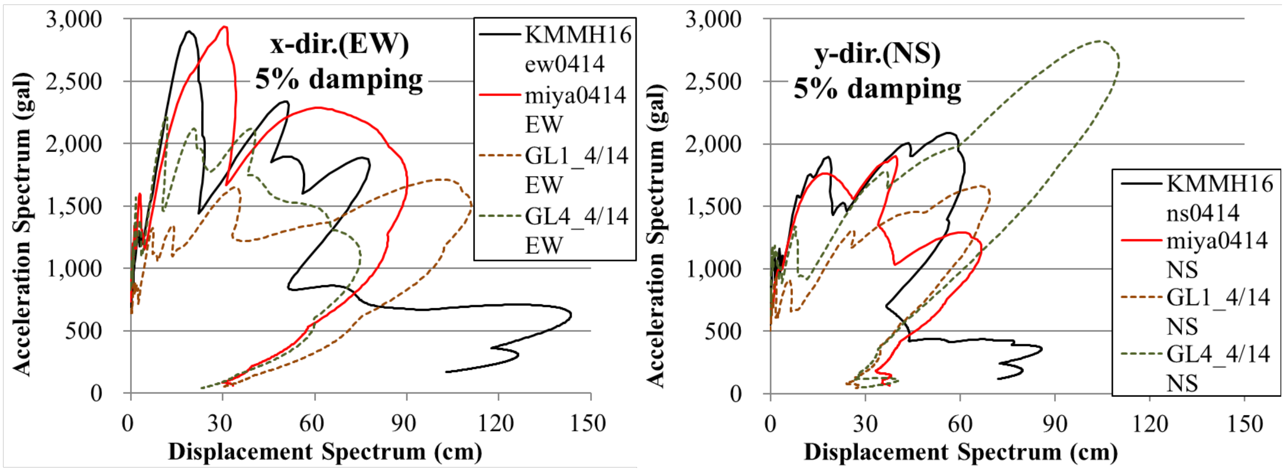


Fig. 7–Spectrum of Input motion (recorded on 4/14) using 5% damping

5.3 Results of response analysis

The maximum response drift angle of each story by the input motion of the 1st floor observed record in x-direction (EW) are shown in Fig.8 comparing FX10 to FX15. The 1st story drift angle of FX10 exceeded 1/20 though the expectation angle from the damage was 1/250 due to the residual shear crack width and damage level shown in Photo 2, Photo 4, and Fig. 4, but that of FX15 was 1/200 which could explain the damage. That is, structural evaluation by Japanese design standard gives the safer results for the expectation of damage, and in order to expect the real response, in this analysis, almost 1.5 times stiffness and strength of story shear force drift angle relationship from the static push over analysis would be reasonable in consideration of the influence by the force distribution, by the actual higher material strength than the design use, and by the slab effective width and rebar adding to the beams.

The maximum response drift angle in y-direction are shown in Fig.9. The same results as x-direction also concluded from the response of y-direction. The maximum drift angle of FX15 responded about 1/400 which damage level showed in y-direction in Photo 3 and Fig. 4.

The maximum response drift angle of each story by the input motion of the ground analysis record in x-direction (EW) are shown in Fig. 10 comparing SR15 to SR15V07. The 1st story drift angle of SR15 exceeded 1/25 though the expectation angle from the damage was 1/250, and that of SR15V07 was 1/250 which conformed to the damage.

The maximum response deformation of each floor and the basement in x-direction (EW) are shown in Fig. 11 comparing SR15 to SR15V07. The maximum response deformation on the foundation SR15V70 was 500mm which showed reasonable deformation to the pile damage.

The time history acceleration is shown in Fig. 12 comparing the 1st floor analytical response to the observed record on 4/16. The maximum analytical 1st floor response acceleration in SR15V07 was consistent with the motion recorded on 4/16. The relationship between the acceleration and displacement response spectrum using 5% damping of the motion recorded on 4/16 and the 1st floor analytical motion of SR15 and SR15V07 are shown in Fig. 13. The response spectrum of 1st floor analytical motion in SR15V07 was also consistent with the motion recorded on the 1st floor in the building on 4/16.

Consequently, in order to explain the all damages and observed records at the same time, in this study, it was necessary to establish the stiffer and stronger story shear force drift angle relationship than the expectation following design standard or guidelines, and to assume nonlinear characteristic of the soil and the piles.

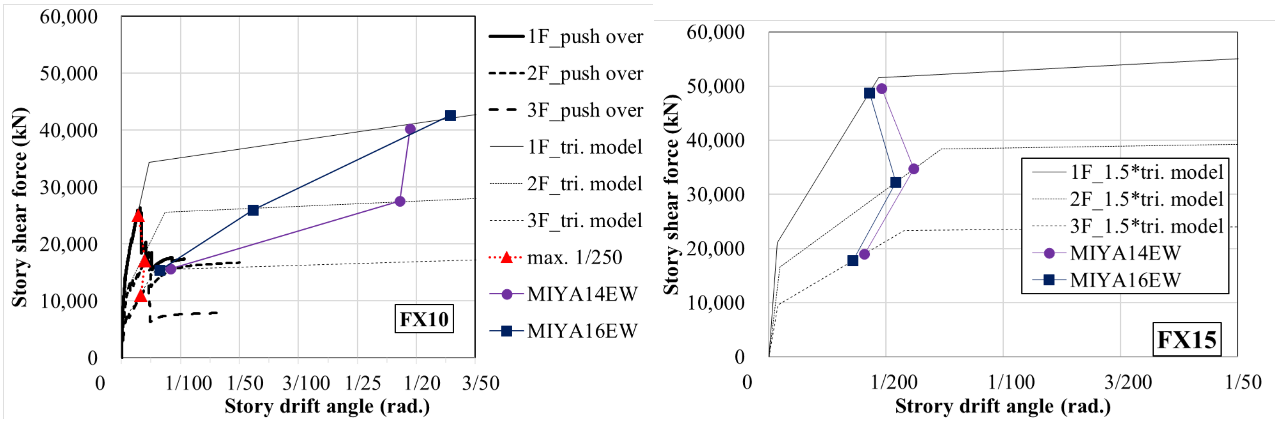


Fig. 8– Response shear and drift angle in each story of FX10 and FX15 in x-direction (EW)

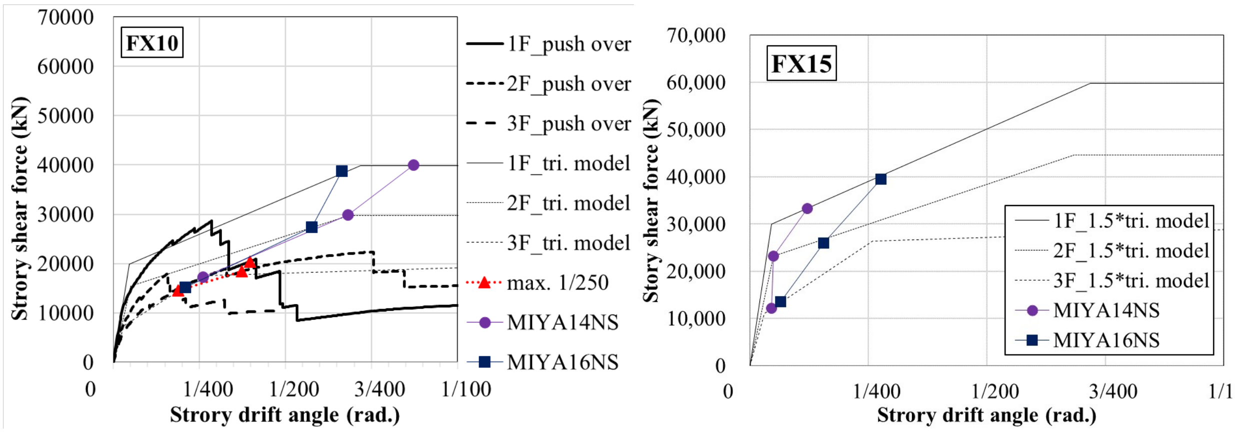


Fig. 9– Response shear and drift angle in each story of FX10 and FX15 in y-direction (NS)

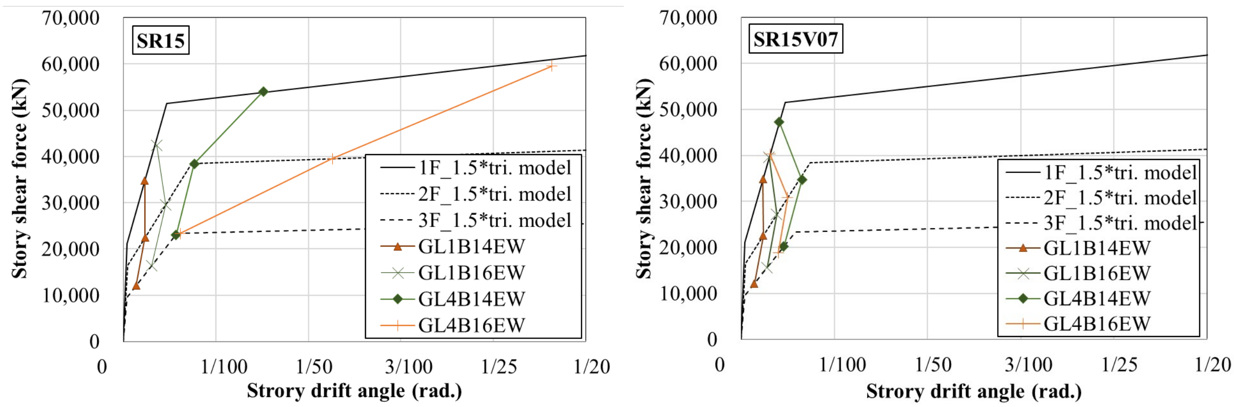


Fig. 10 – Response shear and drift angle in each story of SR15 and SR15V07 in x-direction (EW)

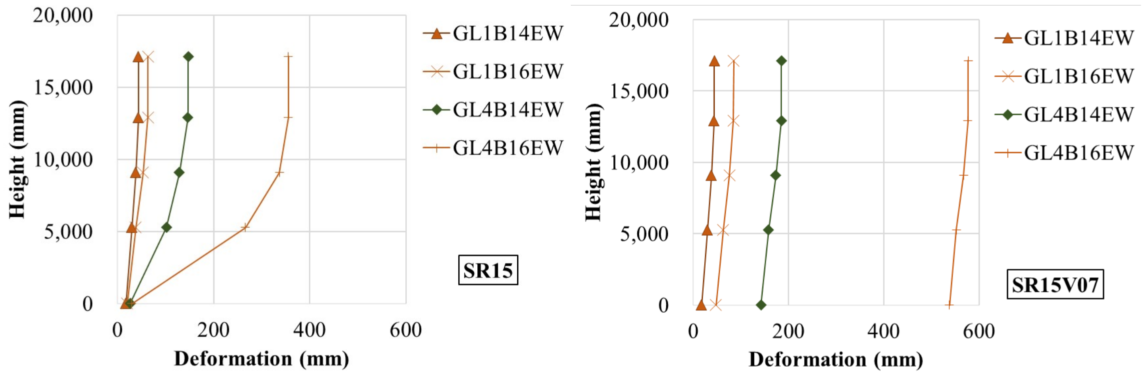


Fig. 11 – Response deformation of SR15 and SR15V07 in x-direction (east to west)

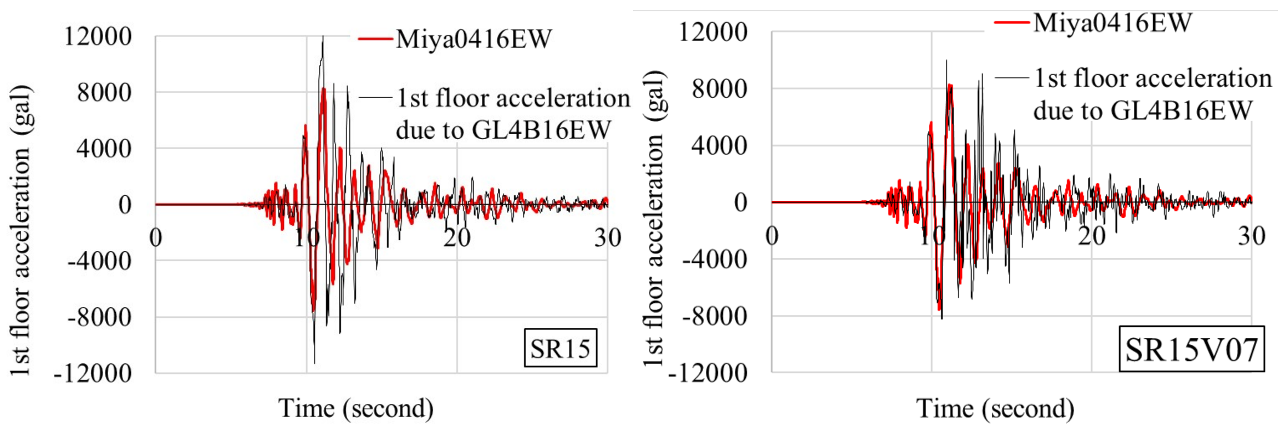


Fig. 12 – 1st floor Response and observed record of SR15 and SR15V07 in x-direction (east to west)

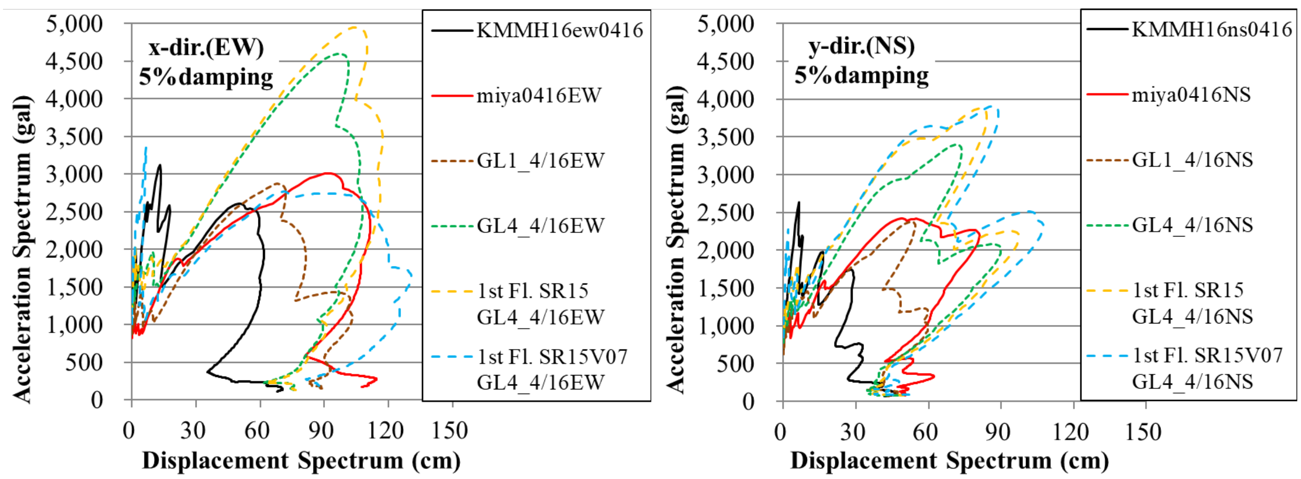


Fig. 13 – Spectrum of input motion(recorded on 4/16) and 1st floor response using 5% damping

6. Conclusion

A government building which was constructed in 1980 and retrofitted in 2001 was damaged by the 2016 Kumamoto earthquake, and making it difficult to continue to use. For this reason, a detailed damage survey of this building and the foundations were conducted. In this paper, the damage situation, the damage assessment, and the seismic evaluation in 2012 were showed and compared with the seismic response (e.g. the maximum drift angle of the building above, the maximum deformation of the piles, and the 1st floor response acceleration) by the response analysis.

As a result, the damage of the building and piles and the motion recorded on the 1st floor could be explained by the results from the analytical model "SR15V07" inputting the surface acceleration considering ground surface amplification and sway rocking spring.

Story stiffness and shear strength in each story of "SR15V07" are 1.5 times stiffer and stronger than the assumption by the push over analysis followed Japanese building standard, and the sway spring model of "SR15V07" was assumed to slide by the inertia force of 0.7 times total weight of the building.

That is, assuming the static loading distribution and considering the slab effective of one-meter width to evaluate ultimate flexural moment of beam, concrete material compression test results, and rebar strength of 1.1 times yield strength followed Japanese design standard underestimated the actual rigidity and strength of the building. The maximum acceleration, envelope, and response spectrum of seismic observation records could be roughly estimated from the response of the first floor. The relative horizontal displacement of the building and the ground due to the damage of the pile head could be roughly estimated by adding a sliding behavior to the sway spring.

7. Acknowledgements

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