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SEISMIC RESPONSE EVALUATION OF 30 STORY HIGH-RISE RC BUILDING SUBJECTED TO 2011 TOHOKU EARTHQUAKE AND OTHERS

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ABSTRACT

The study compares the analysis results from various stiffnesses, building weights, damping models and damping ratios of analytical models by normal design procedure with actual natural periods and seismic records of existing buildings. Accordingly, the correlation between analysis results and seismic records is evaluated by examining the effect of setting values of damping models and damping ratios on response values.

Furthermore, the report considers the relation between the damage estimation through analysis results of seismic response of the 2011 Tohoku Earthquake and the actual damage on existing buildings.

Keywords: High-rise RC building, Seismic record, Analytical model, Viscous damping

INTRODUCTION

The Tohoku Earthquake in March 2011 did not cause significant damage to the frames of high-rise reinforced concrete (RC) buildings, although moving and overturning of indoor furniture, as well as damage on partition wall finishes and other non-structural members have been reported(Watanabe, 2012). In seismic design of high-rise buildings, the focus is mainly on the maximum response values of story deformation angle and story shear force. In general, the various parameters of the seismic response analysis model used for design have been established to estimate these response values during major earthquakes.

For indoor response evaluation intended for moving and overturning of indoor furniture, the essential factors are the maximum response values of acceleration and velocity. In order to correctly evaluate these, estimation of the damping in higher modes is essential. However, few studies have been conducted on comparing the effects of damping estimation on the response results of analytical models commonly used in design.

In this paper, we will examine the effects on responses of the setup value used in the damping model, as well as the effects of building load estimation and frame stiffness used in the high-rise RC building's seismic response analysis model, by comparing actual measurements or observed records on the existing building and the analysis results. Furthermore, we will investigate the connection between the actual building's damaged condition and the indoor response evaluation based on results from the seismic response analysis using the seismic motion of the Tohoku Earthquake.

DESCRIPTION OF THE SEISMIC OBSERVATION RECORDS

The building under seismic observation(Inai et al., 2008) is a 1-story basement, 30-story RC building of 95.9 m in height, with a span of 38.4 m on the x-direction (longitudinal direction) and 32.4 m on the y-direction (lateral direction). The foundation consists of cast-in-place concrete piles, with their end at -52 m below ground level.

Seismic observation was conducted using seismographs (accelerometers) installed at 3 points : the basement floor level, 15th story floor level and rooftop floor level, with measurements taken at the x

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and y directions to match the building axes. Figure 1 shows the ground plan, cross-section view and the positions of the installed seismographs for the building.

Microtremor measurements were conducted on the building at the time of its completion. The measured natural periods, which were estimated from these microtremor records, are given in Table 1.

Since seismic observations for this building have been carried out since April 1999, strong motion records obtained from the Tohoku Earthquake on 11th March 2011 are also available. From the numerous records observed thus far, we selected 11 ground motion waves containing relatively large maximum accelerations. Descriptions of the selected seismic records are shown in Table 2.

ANALYTICAL MODEL CONFIGURATION

Equivalent Flexural Shear Model

The equivalent flexural shear lumped mass model was used for seismic response analysis. The equivalent flexural shear model is a 31-lumped mass model in which flexural springs represent the flexural deformation of the entire building caused by column axial deformations and shear springs represent the shear deformation at each story. Flexural springs are considered to be elastic while the nonlinear behavior of each story is represented by tri-linear curves in the shear springs. The TAKEDA model is used for the hysteretic characteristics of the shear spring. Input of the ground motion is set at the basement floor level, while the foundation level is fixed.

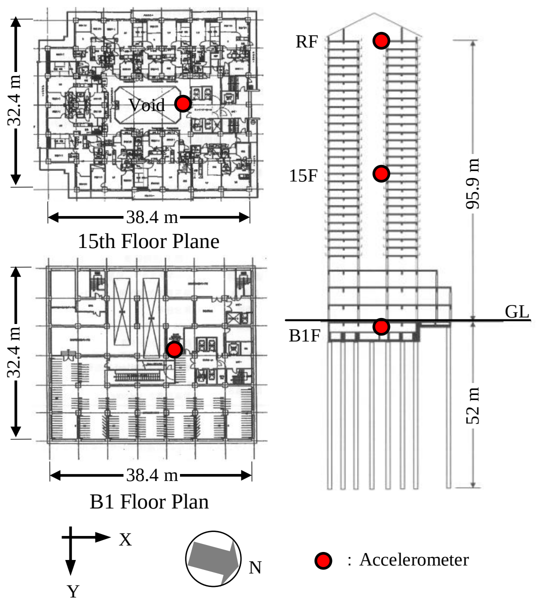


Figure 1. Thirty-story high-rise RC building

Table 1. Measured natural periods

Direction	X direction			Y direction		
Natural periods (sec)	1st mode	2nd mode	3rd mode	1st mode	2nd mode	3rd mode
	1.36	0.44	0.24	1.47	0.45	0.24

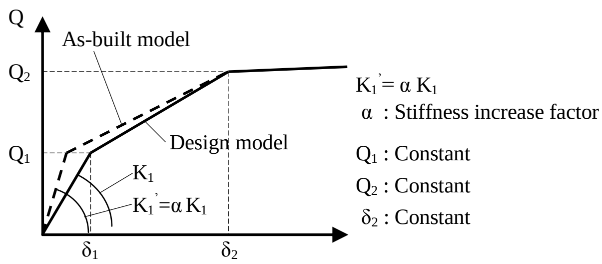


Figure 2. Consideration method of the stiffness increase factor on the skeleton curve

Table 2. Description of selected seismic records

No.		1	2	3	4	5	6	7	8	9	10	11
Occurrence date		2002/ 7/13	2003/ 5/26	2004/ 10/23	2004/ 10/23	2005/ 4/11	2005/ 7/23	2005/ 8/16	2005/ 10/16	2007/ 3/25	2007/ 7/16	2011/ 3/11
Occurrence time		21:45	18:25	17:56	18:34	7:22	16:34	11:46	16:05	9:41	10:13	14:46
Seismic center		South of Ibaraki pref.	Off Miyagi pref.	Niigata pref. Chuetsu	Niigata pref. Chuetsu	North-east of Chiba pref.	North-west of Chiba pref.	Off Miyagi pref.	South of Ibaraki pref.	Off Noto peninsula	Off Niigata pref. Chuetsu	Off Sanriku
Magnitude (M)		4.8	7.0	6.8	6.5	6.1	6.0	7.2	5.1	6.9	6.8	9.0
Epicentral distance(km)		38	448	188	188	128	38	352	65	288	250	417
B1F Amax. (cm/s^2)	X	9.6	9.5	12.2	12.7	6.7	42.8	13.5	13.1	2.0	12.1	71.3
	Y	7.7	10.9	17.2	16.1	10.5	63.9	16.4	15.6	1.8	9.6	91.3
B1F Vmax. (cm/s)	X	0.7	1.6	2.9	2.4	1.4	7.9	2.4	1.6	1.2	3.9	18.4
	Y	0.9	1.8	3.6	2.8	2.0	15.8	3.0	1.9	1.3	2.7	22.8

Design Model and As-built Model

In the model used during the building design, the specified concrete strength Fc, which is the lower limit for concrete strength, is used to conservatively estimate the member strengths and deformations. Moreover, the live load for the building weight is calculated under a fully loaded condition in order to conservatively estimate earthquake loads.

The model configured with the above conditions is defined as the design model (SD model). For the design model stiffness, the modulus of elasticity calculated from the specified concrete strength Fc, while 100% seismic live load is considered for the building weight.

In general, the concrete strength of the actual building is higher than Fc, and the modulus of elasticity is larger as well. Hence, member stiffnesses of the actual building are expected to be larger than those found in the design model.

For the analysis, a model with the stiffness increase factor set at 1.20 (SM1) and another model set at 1.10 (SM2) was configured as the as-built model (SM model), to take into account the larger modulus of elasticity resulting from the increase in concrete strength.

In considering the stiffness increase factor, the stiffness increase factor was used in the first stiffness ratio, while the story shear force of the first break point Q_1, second break point Q_2, and the deformation of the second break point δ_2 are identical to the design model. Figure 2 shows how the skeleton curve for the equivalent flexural shear model, considering the stiffness increase factor, is determined.

The analytical model chart is given in Table 3. Note that the building weight, used for comparison with natural periods obtained from microtremor observations, includes a load reduction factor of 0.94 in order to simulate the live load at building completion.

Comparison between Natural Periods of Analytical Model and Microtremor Measurement Results

Table 4 presents the comparison between natural periods from each analytical model and the measured natural periods based on microtremor measurements.

The fundamental natural period of the design model (SD model) is about 16% longer than the measured fundamental natural period. This is because in the actual building, the member stiffnesses are higher relative to the design model and the building weight is smaller. Looking at the fundamental natural period of the as-built model, which takes into account the stiffness increase and weight reduction in the actual building, the period of the SM1 model is longer by about 5%, while the SM2 model is longer by about 9% compared to the measured fundamental natural period. These show better agreement compared to the design model. However, because the measured natural periods are estimated values from minute amplitudes in the microtremor measurements, the measured natural periods tend to be shorter than the natural periods of the frame due to the stiffness contribution of non-structural walls and such.

Table 3. Analytical model chart

Name of models		Stiffness increase factor	Load reduction factor
Design	SD	1.00	1.00
As-built	SM1	1.20	1.00 (0.94*)
	SM2	1.10	1.00 (0.94*)

* It's used for comparison with natural periods obtained from microtremor observations.
※The stiffness increase factor and the load reduction factor is the ratio of the design model.

Table 4. Comparison of natural periods

Model	X dir. natural period (sec)			Y dir. natural period (sec)		
	1st mode	2nd mode	3rd mode	1st mode	2nd mode	3rd mode
Measured	1.36	0.44	0.24	1.47	0.45	0.24
SD	1.58	0.56	0.32	1.70	0.59	0.33
SM1	1.43	0.50	0.28	1.55	0.53	0.30
SM2	1.48	0.52	0.29	1.60	0.55	0.31

INTERNAL VISCOUS DAMPING CONFIGURATION

Internal Viscous Damping

In seismic response analysis models used for design, an instantaneous stiffness proportional type of internal viscous damping is generally used. However, the damping model used for analysis and the value set for the damping ratio uses the conventional values of the design phase and may not always reflect the real damping property of the actual building. In this study, we performed analyses using fluctuating damping ratios and damping models, and by comparing the results with seismic observation records taken from the actual building, we studied the setup value for internal viscous damping.

For the damping models, we selected instantaneous stiffness proportional damping, Rayleigh damping and mode-differentiated damping, while the damping ratio for each model were fluctuated for the investigation. The chart of the selected damping models and damping ratios is given on Table 5.

Table 5. Damping models and damping ratios

Damping model	case1	case2
Instantaneous stiffness proportional damping	h_1 =1%	h_1 =3%
Rayleigh damping	h_1 = h_2 =1%	h_1 = h_2 =3%
Mode-differentiated damping	All modes h =1%	h_1 =1%, Others h =2%

Model type S - 1 : Instantaneous stiffness proportional case1
Damping model ↲ ↳ Case of Damping Ratio

S : Instantaneous stiffness proportional, R : Rayleigh, M : Mode-differentiated

Verification of Internal Viscous Damping by Comparing Analysis Results and Seismic Observation Results

Setting the equivalent flexural shear model as the analytical model, as well as setting the stiffness increase factor to the one that is considered closer to the increase factor of the actual building's modulus of elasticity, the as-built model SM2 is used.

The seismic motion for use in this study is set to 10 wave records, excluding the Tohoku Earthquake, which are selected from the observed wave records taken at the building’s basement level (Table 2). To take into account the effect on hysteretic characteristics of the seismic motions experienced in the past, the 10 observed wave records are consecutively used as input seismic motion.

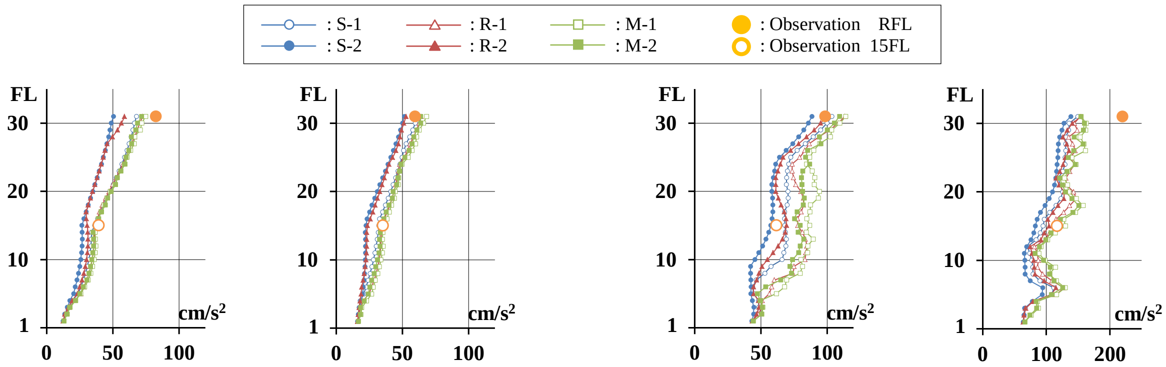
The maximum response acceleration from the analysis results and from observations are compared in Figs. 3 and 4 for the observed wave record nos. 4 and 6, both of which had relatively large response acceleration among the investigated wave records. For seismic motion no. 4, the difference in responses due to the damping model is small and the agreement between analytical values and observed values are good for both x and y directions. Furthermore, the damping ratio setting for case 1 showed good agreement. For seismic motion no. 6, although the agreement between analytical values and observed values are relatively good in the x direction, the observed values at the rooftop floor are underestimated in the y direction for all models.

The maximum response acceleration from analysis results and observation records (15th floor and rooftop floor) are compared for each damping model and presented in Figs. 5 and 6 for the 10 investigated wave records. At large accelerations, instantaneous stiffness proportional damping tends to underestimate response acceleration compared to damping models using Rayleigh and mode-differentiated damping. Moreover, for the damping ratio setting, the agreement with observed values tends to be better for case 1 compared to case 2.

RESPONSE ESTIMATION FROM THE 2011 TOHOKU EARTHQUAKE

Analytical Model

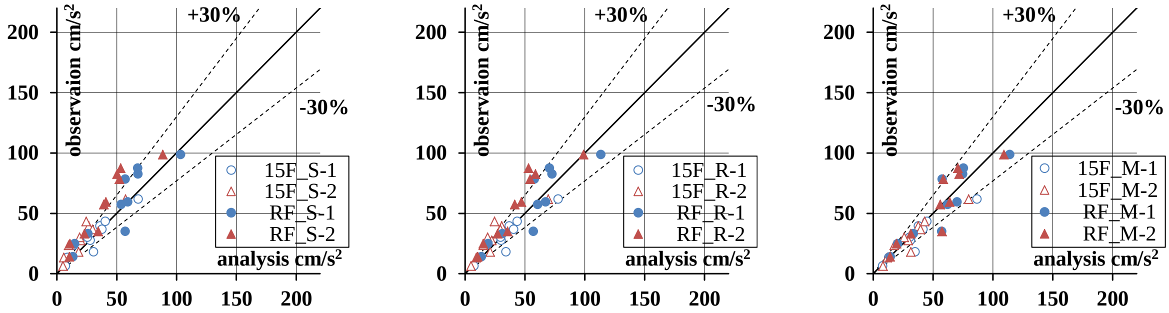
The equivalent flexural shear model is used as the analytical model for the investigation, while the stiffness and weight is set to the as-built model SM2. The damping model and damping ratio set up in previous section is used for the internal viscous damping, and the agreement between response acceleration for each model and observation records are discussed.



(a) X direction (b) Y direction (a) X direction (b) Y direction

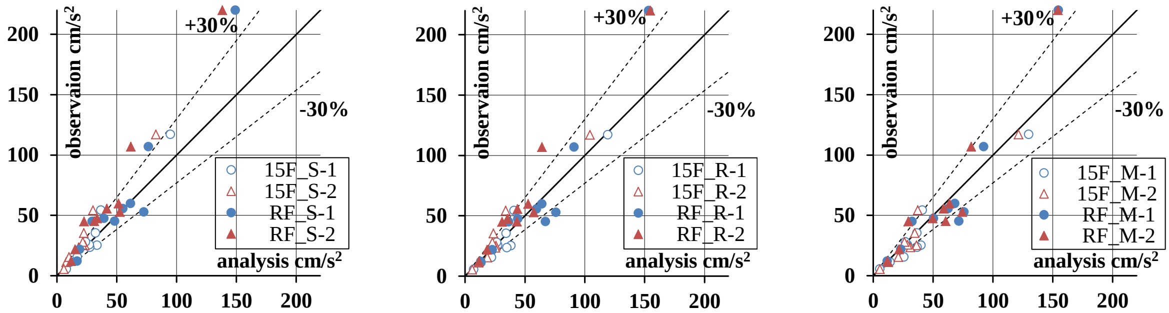
Figure 3. Maximum response acceleration (observed wave record No.4)

Figure 4. Maximum response acceleration (observed wave record No.6)



(a) Stiffness proportional (b) Rayleigh (c) Mode-differentiated

Figure 5. Comparison of maximum response acceleration from analysis results and observation records (X direction)



(a) Stiffness proportional (b) Rayleigh (c) Mode-differentiated

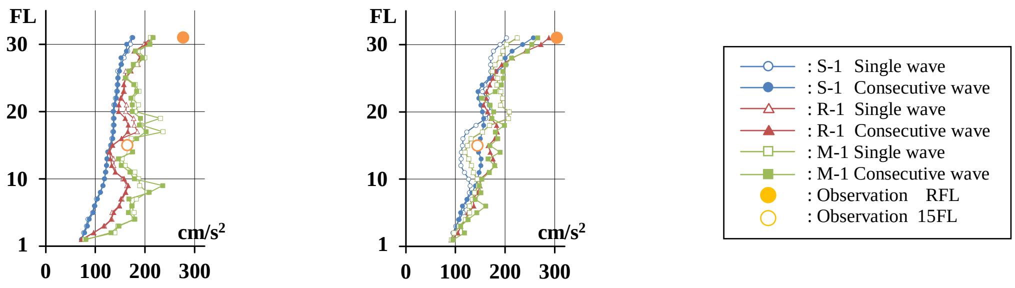
Figure 6. Comparison of maximum response acceleration from analysis results and observation records (Y direction)

Input Seismic Motion

The 2011 Tohoku Earthquake (observed wave record no. 11) is used as the seismic motion for the investigation. To investigate the effect on hysteretic characteristics of the seismic motions experienced in the past, the input is set to (1) the observed wave record nos. 1 - 11 given in Table 2 applied consecutively (consecutive wave record) and (2) no. 11 only (single wave record).

Comparison of Seismic Observation Results and Response Estimation

The maximum response acceleration values due to consecutive and single wave records are compared in Fig. 7 for the case 1 damping model. For the x direction, the difference in responses due to consecutive and single wave records is small. For the y direction, there are differences exhibited in the responses due to consecutive and single wave records. The response acceleration at the rooftop floor tends to be large when using the consecutive wave record, and the agreement with observation results is good. Next, the differences in the x and y direction are discussed. For seismic motion no. 6 shown in Table 2, relatively large acceleration values are recorded in the y direction (see Fig. 4). In the x direction, the maximum shear responses in the analysis are at the level of crack point Q_1 or less on the skeleton curve, or even slightly going over in some cases; while in the y direction, they greatly exceeded Q_1 in nearly all the stories, reaching to a level of about 30 - 40% of yield point Q_2. Such effects are manifested in the hysteretic characteristics under the consecutive wave record, and may be causing the difference in response values compared to the single wave record.



(a) X direction (b) Y direction

Figure 7. Comparison of maximum response acceleration due to consecutive and single wave records

The comparison of maximum responses due to different damping models are shown in Figs. 8 and 9 for the consecutive wave record. The observation results are also shown together in the figure for the maximum response acceleration.

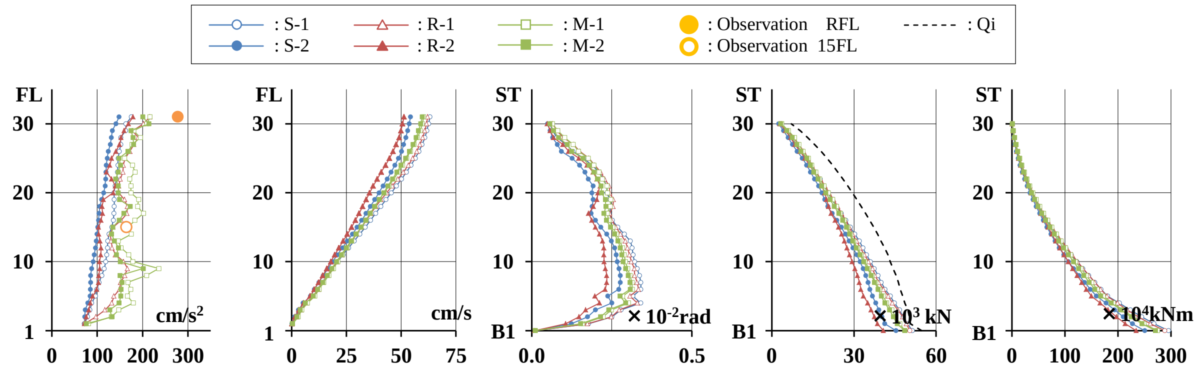
For the response acceleration, the rooftop floor response values in the x direction are smaller than the observation results for all damping models. At the 15th floor, response values for S-1, S-2 and R-2 damping models are smaller than the observation result although response values for R-1, M-1 and M-2 show comparatively good agreement. In the y direction, the S-2 damping model response values at both the 15th and rooftop floor are small compared to observation results. For the other damping models, the 15th floor response values show relatively good agreement with the observation result. However, R-1 shows good agreement with observation result in the response values at the rooftop floor, while all the models except R-1 are a little smaller.

For all the other response values, the difference in results due to different damping models are relatively small compared to those found in acceleration. The maximum story deformation angle responses grow large at the lower story levels in the x direction and at the middle story levels in the y direction, with a peak of about 1/300 to 1/270. The maximum story shear responses do not exceed the design shear force (Qi), so that member stresses may be considered at allowable stress level or less. These results match the survey results described in the literature(Watanabe, 2012), stating that there is no obvious damage incurred by the structure.

Estimated Indoor Damage based on Analysis Results

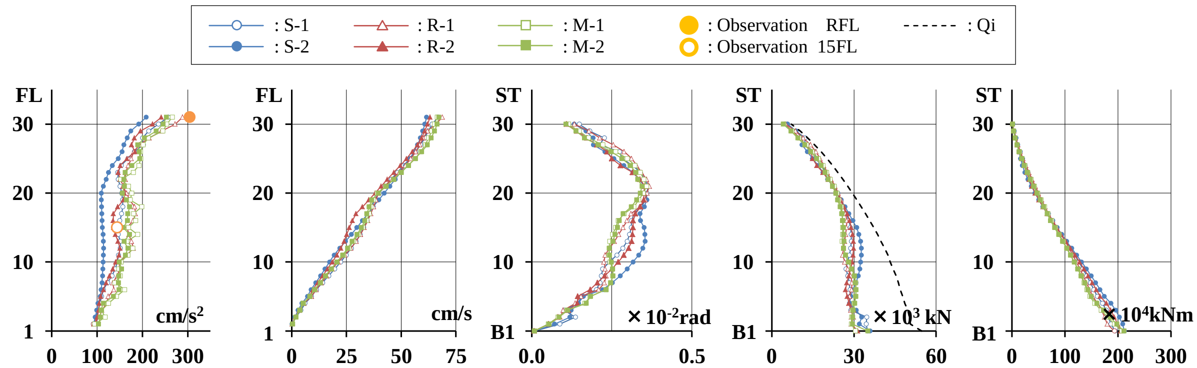
Indoor damage caused by the Tohoku Earthquake is estimated based on analysis results. The damping model used for the analysis is Rayleigh damping, case 1 (h_1 = h_2 = 1%, R-1), which gave relatively good agreement between analysis and observation results.

The damage estimation method used is similar to the one in the literature(Arai, et al., 2012), and gives estimates based on a simple estimation method used in the past[4)]. The estimation of damage from overturning furniture is carried out by comparing the maximum floor response acceleration (A_f) to the acceleration at 50% overturning rate of furniture (A_{R50}). The estimation of damage from sliding furniture is carried out by comparing the amount of furniture sliding (δ_S) to the sliding limit (δ_0). Note that A_{R50} and δ_S are calculated using the formulas given in the literature(Kaneko, 2003), while δ_0 is set to 100 cm, assuming sliding furniture equipped with casters. The various parameters for the furniture investigated are given in Table 6.



(a) Acceleration (b) Velocity (c) Story deformation angle (d) Story shear (e) Overturning moment

Figure 8. Comparison of maximum responses due to different damping models (X direction)



(a) Acceleration (b) Velocity (c) Story deformation angle (d) Story shear (e) Overturning moment

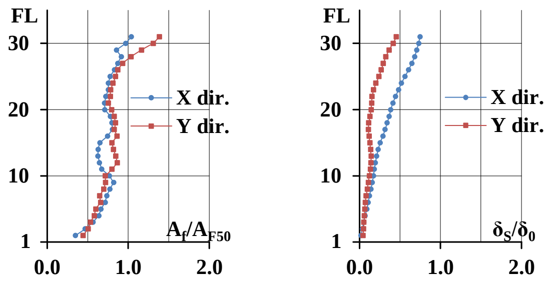
Figure 9. Comparison of maximum responses due to different damping models (Y direction)

Figure 10 shows A_f/A_{R50} and δ_S/δ_0 based on the analysis results. On the one hand, A_f/A_{R50} is around 0.8 from the middle up to the upper floors, with the upper floors exceeding 1.0 in the y direction. On the other hand, δ_S/δ_0 is comparatively small at less than 1.0 over all the stories. From the estimates, damage from overturning is expected to occur from the middle up to the upper floors. Furthermore, looking at the maximum story deformation angles shown in Figures 8(c) and 9(c), the responses grow larger from the lower to the middle floors. Hence, there is a high possibility of damage on finishings and such at these regions and their vicinity.

Table 6. Parameters of the furniture

Kind of furniture	b/h	μ
Overturning furniture	0.18	0.30
Sliding furniture	0.40	0.05

b/h : Width and height ratio of furniture
μ : Friction coefficient



(a) A_f/A_{F50} (b) δ_S/δ_0

Figure 10. Estimation of indoor damage based on analysis results

In the literature(Watanabe, 2012), cracks on interior materials at this building were reported to have occurred from 5th to 15th floor due to the 2011 Tohoku Earthquake. Also, according to a questionnaire survey conducted by Hida et al.(2011), moving and overturning furniture were reported at the top floors while cracks on interior materials were observed relatively more often at the lower floors. These phenomena match the tendencies of the indoor damage estimates based on analysis results. However, the estimated values for damage from overturning are relatively small. This may be attributed to the approximation in the furniture parameters used. Also, damage on finishings provided in the literature(AIJ, 2003) defines a damage level of 1 for a story deformation angle of 1/150 and above. In comparison, the story deformation angle in the analysis results, around 1/300 to 1/270, are smaller than the above. The method for determining damage on finishings is a topic for future studies.

CONCLUSIONS

The findings obtained from the scope of this study are as follows.

(1) The natural period of the actual building was shorter than that of the design model by around 16%. For the as-built model with a stiffness increase factor of 1.20 and a weight reduction factor of 0.94, the differences with the actual measurement values are about 5%.

(2) Comparisons were made between analysis results from fluctuating internal viscous damping and past observation records (acceleration) to investigate the effect of the damping model and the value set for the damping ratio on the response. Results showed that, although compatibility with observation records depend on the characteristics of the seismic motion, case 1 using smaller damping ratios in higher modes exhibited good agreement with observation records for all damping models.

(3) Results of indoor damage estimates based on analysis results generally matched the tendencies found in damage surveys based on questionnaires. However, since there were insufficient data on furniture parameters and damage condition, the agreement with determined values is a topic for further studies.

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