SYSTEM IDENTIFICATION AND RESPONSE ANALYSIS OF RC HIGH-RISE BUILDINGS UNDER SUCCESSIVE EARTHQUAKES

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ABSTRACT

System identification was performed to two RC high-rise buildings located in Tokyo, Japan, one was 37 stories and the other was 32 stories. Response analysis was conducted to the 37-story building for which there was structural data available. The strong motion data of the buildings that recorded the successive earthquakes: before, during, and after the mainshock of the 2011 Off the Pacific Coast of Tohoku Earthquake were used as input in system identification to obtain dynamic parameters. The parameters as results of system identification were then used as input data in response analysis to observe the behavior of the MDOF model under some earthquake motions and to determine the adequate damping type that governs the behavior. Health monitoring was conducted as a combination of system identification and response analysis to monitor the more specific part of the building. The dynamic parameters of the buildings after the mainshock were changed compared to that before the mainshock, the frequency decreased to 24%, the period increased to 32%, and the damping factor increased to 50%. The adequate damping types of the MDOF model before the mainshock were modal damping and Rayleigh damping, while that during the mainshock were proportional damping to initial stiffness and proportional damping to nonlinear stiffness. The structural members were in a good condition before the mainshock, but the stiffness degradation occurred when they were struck by the mainshock, the story shear force exceeded its cracking capacity but still below the yielding capacity, with maximum ductility 0.4, it happened to almost the entire structure except the 4 top stories. The decreases of frequency on the upper and lower part of the building were 15% and 18% respectively that indicates the stiffness degradation on the lower part was higher than that on the upper part.

Keywords: Reinforced Concrete, High-rise, Earthquake, System Identification, Response Analysis.

1. INTRODUCTION

The existence of high-rise buildings nowadays becomes a common sight in many cities, both in developed and developing countries. Various reasons stand behind its construction: the increasing demand of housing unit, limited land in urban areas, or by the reason of community prestige due to the existence of a highrise building considered to reflect economic and technological development of a region. Reinforced concrete (RC), as a popular building material is widely used due to some advantages such as low construction cost, simple treatment, and easily shaped for aesthetic purposes.

In earthquake prone areas, it was necessary to ensure the safety of highrise building structures against the damage due to earthquakes, although rare of highrise buildings were damaged when earthquake struck, until the Mexico earthquake in 1985. That earthquake destroyed many highrise buildings because of its characteristic of a long period which resonated with high-rise buildings that had the same characteristic of period. One of the methods to ensure the structural safety

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is to identify the dynamic parameters and monitor the behavior of building, to find out some tendency of the decreasing quality of structure that will lead to the fatal failure anytime when earthquake strikes.

The big earthquake of magnitude M9.0 struck Japan on March 11, 2011 at 14:46 JST, the epicenter was at off-shore of Sanriku (38.103ºN, 142.860ºE), in 24 Km depth. The magnitude of the main shock was the largest in Japan up to the present (2011). According to the report of The Headquarter for Earthquake Research Promotion (2011), this event had a maximum seismic intensity 7 JMA observed in Kurihara City, Miyagi prefecture. The maximum aftershock was a M7.7 earthquake that occurred at 15:15 on March 11, as of April 11. There were more than 60 aftershocks of M6.0 or over. On April 7, there was a M7.1 earthquake with a seismic intensity 6 Upper observed in Miyagi prefecture. The aftershock area extends approximately 500km in an N-S direction. There is fear that large aftershocks will occur from now on, and there is a possibility that the area will be hit by strong shaking and high tsunami. The severe damages on buildings were mostly caused by the tsunami, thus concentrated along the eastern coast of Honshu Island (pacific coast), causing collapse or dragged away of buildings. The earthquake itself did not significantly affect structural damages of new buildings (designed by the building code after 1981) or retrofitted buildings. Although only minor structural damages were found on some buildings, the successive earthquakes after the mainshock had led to fears of increasing levels of damage.

The K-Tower and the S-Tower are the target RC high-rise buildings in this study, both located in Tokyo, Japan. The K-Tower is 119 m height which has 37 stories, three strong motion sensors were planted in this building, located on the first floor (01F), middle floor (18F) and top floor (37F). The S-Tower is 111.99 m height which has 32 floors, two strong motion sensors were planted in the building i.e. on the first floor (01F) and the top floor (32F), one sensor planted underground.

The strong motion data records of the K-Tower to be analyzed in this study are 127, while that of the S-Tower are 60. Those buildings have regular shapes in vertical as well as in horizontal, the appearance and general plan view of the buildings can be seen briefly in Figure 1 and 2.

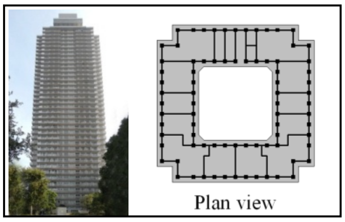


Figure 1. The K-Tower

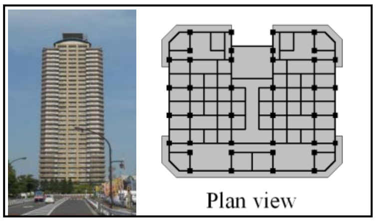


Figure 2. The S-Tower

2. OBJECTIVES AND METHOD

The strong motion data of the Target RC high-rise buildings that recorded the successive earthquakes: before, during and after the mainshock of the 2011 Off the Pacific Coast of Tohoku Earthquake were used as input data in system identification to obtain dynamic parameters of the RC high-rise buildings in each earthquake event. The dynamic parameters as results of system identification were then used as input data in response analysis to observe the behavior of the structural model in the multy degree of freedom system (MDOF) and to determine the adequate damping type of the structural model under some earthquake motions.

The health monitoring conducted as combination of system identification and response analysis to monitor the condition of the more specific part of the building. The results of those analysis stages were studied to draw some conclusions. Analysis of the strong motion data was performed by using the *ViewWave* software (developed by DR. T. Kashima, BRI), while the response analysis of the MDOF model was performed by using the *MDOF OS* software (developed by DR. T. Saito, BRI).

3. SYSTEM IDENTIFICATION

The strong motion data records of the K-Tower and the S-Tower are analyzed in this chapter. The dynamic parameters that will be obtained are frequency and period which are identified by Fourier spectral ratio, and damping factor which is calculated by using half power method. The scatters of the modal frequency of the K-Tower and S-Tower are shown in Figure 3 and 4 respectively, the red dashed line represent the mainshock of the 2011 Off the Pacific Coast of Tohoku earthquake.

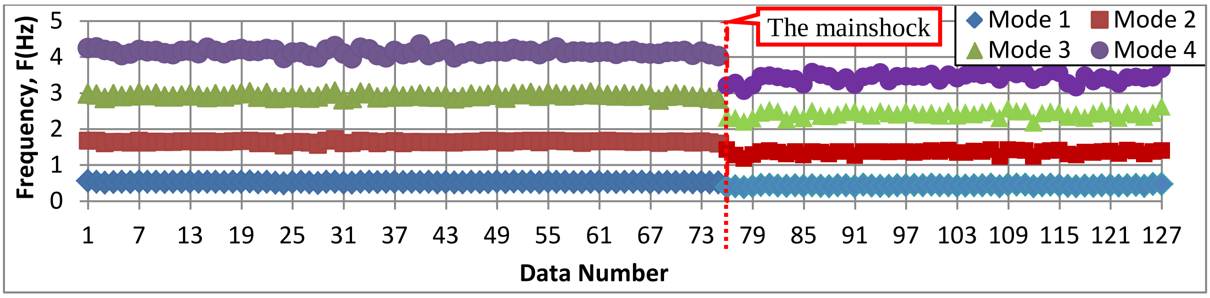


Figure 3. Frequency of the K-Tower

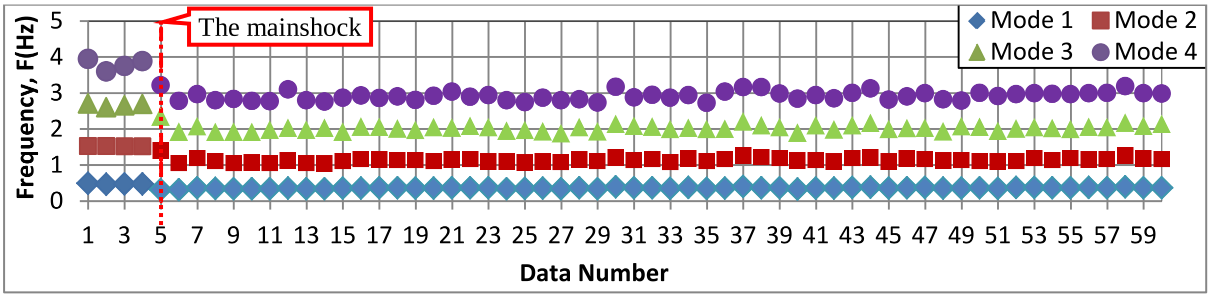


Figure 4. Frequency of the S-Tower

All of modal dynamic parameters of the K-Tower and S-Tower were analyzed up to the 4th mode and the average value before and after the mainshock were calculated, the shift of those parameters before and after the mainshock can be seen in Table 1.

Table 1. The average of dynamic parameters before and after the mainshock

Building	Mode	Before the mainshock			After the mainshock			Shift (%)		
		F(Hz)	T(sec)	h(%)	F(Hz)	T(sec)	h(%)	F	T	h
K-Tower	1	0.528	1.894	2.96	0.432	2.318	4.45	-18.15	22.34	50.03
	2	1.643	0.609	1.48	1.355	0.739	2.21	-17.49	21.36	49.95
	3	2.931	0.341	1.26	2.417	0.414	1.13	-17.53	21.39	-10.36
	4	4.138	0.242	0.64	3.422	0.293	0.64	-17.31	21.04	-0.18
S-Tower	1	0.475	2.105	4.444	0.359	2.792	5.121	-24.47	32.63	15.24
	2	1.519	0.658	1.271	1.139	0.880	2.849	-25.02	33.73	124.11
	3	2.661	0.376	0.434	2.028	0.494	1.073	-23.79	31.39	147.22
	4	3.793	0.264	0.407	2.921	0.343	0.754	-23.00	29.93	85.48

4. RESPONSE ANALYSIS

Response analysis was conducted to the K-Tower for which there was structural data available. Two strong motion data will be selected among the successive earthquakes, one is the strong motion data before the mainshock that occurred on July 16, 2007 (data no.03) which caused the maximum acceleration on the top floor 50.9 gal, the other is the mainshock of the 2011 Off the Pacific Coast of Tohoku earthquake (data no.76) which caused the maximum acceleration on the top floor 198.3 gal, occurred on March 11, 2011. The period as an analysis result from the data records is used to compare the periods that obtained from the response analysis of the MDOF model. There are 4 kinds of analysis result, each depends on the damping type assumed in the analysis, including: Proportional damping to initial stiffness, Rayleigh damping, modal damping, and proportional damping to nonlinear stiffness. The comparisons of periods before and during the mainshock are shown in Table 2.

Table 2. Comparison of period (unit: second)

Event	Mode	Record	Prop. damping to init. stiffness (h1)		Rayleigh damping (h2)		Modal Damping (h3)		Prop. damping to NL Stiffness (h4)	
Before the mainshock	1	1.974	1.974	100%	1.974	100%	1.974	100%	1.974	100%
	2	0.624	0.745	119%	0.741	119%	0.741	119%	0.745	119%
	3	0.350	0.460	131%	0.451	129%	0.451	129%	0.460	131%
	4	0.238	0.325	137%	0.325	137%	0.325	136%	0.325	136%
During the mainshock	1	2.540	2.825	111%	3.020	119%	3.020	119%	3.062	121%
	2	0.702	0.694	99%	0.752	107%	0.752	107%	0.694	99%
	3	0.432	0.476	110%	0.476	110%	0.476	110%	0.476	110%
	4	0.312	0.353	113%	0.337	108%	0.353	113%	0.353	113%

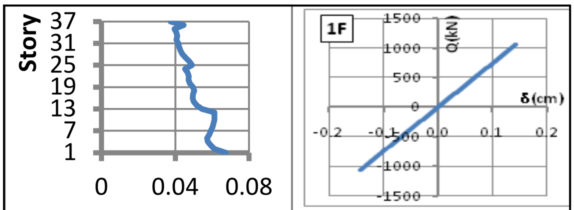
In Table 2, before the mainshock, all of the damping types give the same similarity in the 1st and 2nd mode, but in the 3rd mode the Rayleigh and modal damping give the closest period, while in the 4th mode it is closed by the modal damping and the proportional damping to nonlinear stiffness. During the mainshock, the proportional damping to initial stiffness gives the closest period to that of the data record in the 1st mode, in the 2nd mode the proportional damping to initial stiffness and to nonlinear stiffness give the same similarity, in the 3rd mode all of the damping types give the same similarity, while in the 4th mode the Rayleigh damping gives the closest value.

The comparisons of maximum acceleration before and during the mainshock are shown in Table 3. Before the mainshock, the Rayleigh and modal damping give the closest maximum acceleration to the data record on the 18th floor, while the proportional damping give the closest maximum acceleration on the 37th floor. During the mainshock, the proportional damping to nonlinear stiffness give the closest maximum acceleration to the data record on the 18th floor, while on the 37th floor the Rayleigh and proportional damping to nonlinear stiffness give the same similarity.

Table 3. Comparison of maximum acceleration (unit: gal)

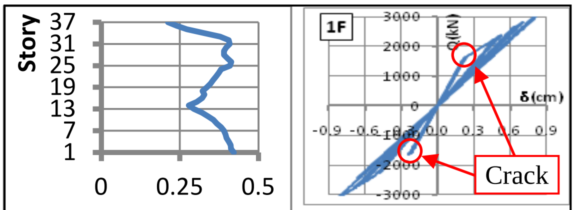
Event	Story	Record	Prop. damping to init. stiffness (h1)		Rayleigh damping (h2)		Modal damping (h3)		Prop. damping to NL stiffness (h4)	
Before the mainshock	18	23.94	22.78	95%	**23.59**	99%	**23.59**	99%	22.78	95%
	37	50.92	**40.20**	79%	39.42	77%	39.42	77%	**40.20**	79%
During the mainshock	18	140.69	98.22	70%	162.16	115%	170.92	121%	**125.06**	89%
	37	198.27	156.56	79%	**226.60**	114%	229.12	116%	**170.96**	86%

The structural performance under the strong motion before and during the mainshock are represented in maximum ductility and hysteresis curve as shown in Figure 5 and 6.



a. Max. ductility b. Hysteresis curve

Figure 5. The performance before the mainshock



a. Max. ductility b. Hysteresis curve

Figure 6. The performance during the mainshock

5. HEALTH MONITORING

In the K-Tower, the strong motion sensors installed on the first floor (01F), middle floor (18F) and top floor (37F), thus identification of suspected damage can only monitor the two parts of the building, which is the lower part (1st floor to 18th floor) and upper part (18th floor to 37th floor), while the details of each floor cannot be done due to limited number of installed sensors.

The method is to observe the changes of frequency along the occurrence time of earthquakes (2007 to 2011) and try to find out which part has been changed or dominantly changed by the earthquake events. By applying Fourier spectral ratio to both the lower part (18F/01F) and the upper part (37F/18F), the frequency and period of those parts can be obtained. Illustration of the method represent in Figure 7. The scatters of frequency are shown in Figure 8, and the values are shown in Table 4.

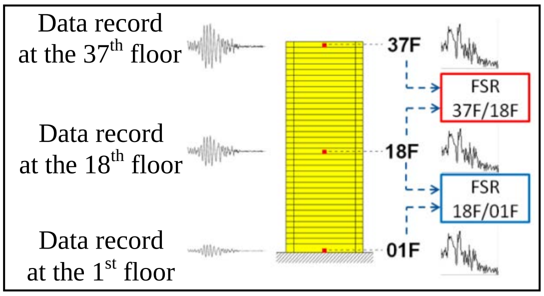


Figure 7. Health monitoring

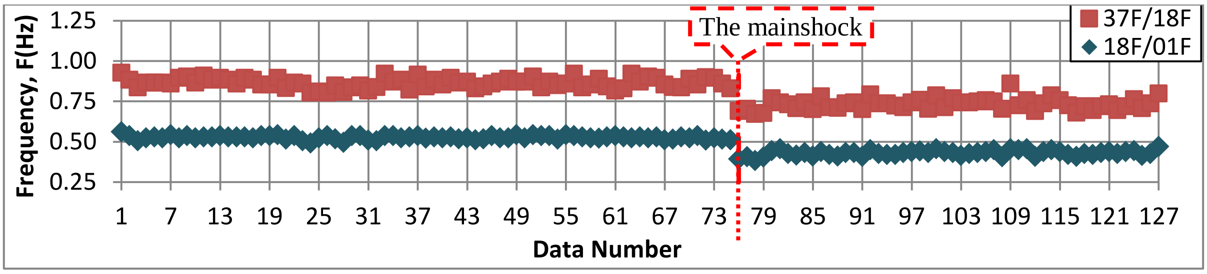


Figure 8. The scatters of frequency in 37F/18F and 18F/01F

Table 4. Frequency of the K-Tower

Position	Before the Mainshock			After the Mainshock			Shift
	Min	Max	Average	Min	Max	Average	(%)
37F/01F	0.493	0.562	0.528	0.383	0.476	0.432	-18.15
37F/18F	0.806	0.928	0.866	0.673	0.861	0.735	-15.17
18F/01F	0.493	0.562	0.528	0.383	0.470	0.431	-18.38

Frequency shift before and during the mainshock is represented for the whole part (37F/01F), the lower, and the upper part as in Figure 9. Mode shape in the 1st and the 2nd mode can be seen in Figure 10. Data no.03 represents the strong motion before the mainshock, while data no.76 represents the mainshock.

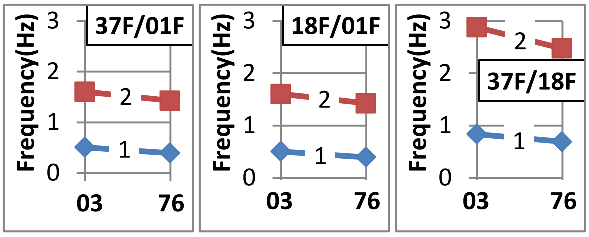


Figure 9. The shift of frequency

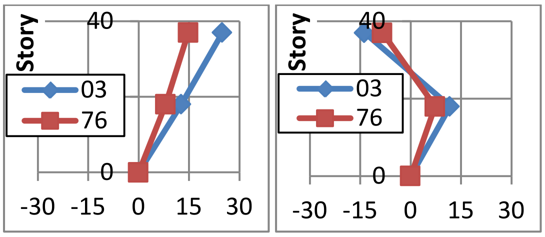


Figure 10. Mode shape

To evaluate the building condition according to the change of vibration characteristics i.e. frequency and mode shape, several cases were arranged to simulate different damage scenarios of structure and tried to find the similar characteristics between the monitored building and the simulated structural model. The frequency shift of the K-Tower is similar to that of the structural model in damage scenarios of case 4 (damage on the top and bottom) and case 5 (damage on the entire structure) where the frequency obtained in the smaller earthquake reduces when the structure is struck by the bigger earthquake. The mode shape of the K-Tower is similar to the graph of mode shape of structural model in damage scenario case 5, although there was a different tendency in the 2nd mode because of the damage level in the simulation was much higher than that in the observed building.

6. CONCLUSION

The natural frequency of the RC high-rise buildings after the mainshock was decreased, comparing to that before the mainshock. The frequency shift of the K-Tower reached 18%, while that of the

S-Tower was 24%. The natural period of the K-Tower observed before the mainshock was 1.9 sec and increased after the mainshock to 2.3 sec, while that of the S-Tower observed before the mainshock was 2.1 sec and increased after the mainshock to 2.8 sec. The damping factor of the K-Tower before the mainshock was 2.9% and increased after the mainshock to 4.4%, while that of the S-Tower before the mainshock was 4.4% and increased after the mainshock to 5.1%. The decrease of frequency indicates the stiffness degradation that could be caused by the structural damage.

Modal damping and Rayleigh damping in the MDOF model before the mainshock, gave the closest parameters to the data record from the real building, but during the mainshock, the closest parameters was given by the proportional damping to initial stiffness and to nonlinear stiffness. The strong motion data before the mainshock did not affect the structural members, the maximum ductility was 0.06, hence no stiffness degradation occurred, but when it was struck by the mainshock, almost the entire story exceeded the cracking capacity except the 4 top stories, the maximum ductility was 0.42. Thus, the stiffness degradation was caused although that did not reach the yielding capacity, hence the structure reached the inelastic condition and supposed there was a minor damage occurred.

The shift of frequency on the upper and the lower parts were 15% and 18% respectively. By conforming the vibration characteristics (frequency shift and mode shape) of the RC high-rise building with the damage simulation of the MDOF model under the El-Centro earthquake 1940, the similar characteristics were found to the simulation of case 5 when the structure damage was on the entire story. The maximum ductility of the MDOF model under the El-Centro was 1.41 which occurred on the 34th story.

7. ACTION PLAN

The method in this study can be applied to other RC high-rise buildings to monitor the condition of the structure when experiencing successive earthquakes. The analysis only used the strong motion data on the building in horizontal direction by neglecting vertical direction and the data of the underground. It is recommended to consider the effect of vertical ground motion and soil-structure interaction to get more satisfactory results. The identification of the vibration characteristics to recognize the mode shape was only performed up to the 2nd mode, because the strong motion sensors were installed only on the 3 locations. It is recommended to install more strong motion sensors to obtain the vibration characteristics in the higher modes. The same method can be tried to be applied to other types of structures to identify the behavior of the different types of high-rise buildings.

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