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RESPONSES OF CONTROLLED TALL BUILDINGS IN TOKYO SUBJECTED TO THE GREAT EAST JAPAN EARTHQUAKE

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ABSTRACT: Many tall buildings in Tokyo metropolitan area were strongly shaken during the Great East Japan Earthquake, March 11, 2011. Most of them are less than 40 years old, and have not experienced the shaking of such a strong level. The tall buildings less than 15 years old, constructed after the 1995 Kobe Earthquake, are typically response-controlled by using dampers, due to the increased concern about serious consequence of seismic damage in major buildings. Some of such buildings are instrumented with sensors and their motions were recorded during the event. This paper discusses the responses of the Japanese controlled tall buildings based on their motions recorded in Tokyo during the 2011 Great East Japan Earthquake. The responses are also compared with those of conventional seismic-resistant buildings.

Key Words: Great East Japan earthquake, tall buildings, response-controlled buildings, vibration period, damping ratio

INTRODUCTION

The Great East Japan Earthquake

At 14:46 on March 11, 2011, the East Japan Earthquake of magnitude 9.0 occurred off Sanriku coast of Japan. It caused tremendous tsunami hazard in the pacific coast of eastern Japan, killing more than 15,000 people, destroying and washing away cities.

The epicenter was 129km from Sendai, the largest city in the northeast of Japan. The depth of the hypocenter was 24 km. The recorded magnitude places the earthquake as the fourth largest in the world since 1900, following 1960 Chile Earthquake M9.5, 1964 Alaska Earthquake M9.2, 2004 Sumatra Earthquake M9.2, and it is the largest in Japan since modern instrumental recording began 130 years ago. The earthquake has recorded the seismic intensity 7, highest in the Japan Meteorological Agency scale, in the north of Miyagi prefecture.

There were many strong earthquake observation networks in operation under the management of research institutes, universities and companies. A large amount of data was well recorded during the earthquake, and various maps are available. For instance, the readers may refer to the distribution of peak ground acceleration (PGA) recorded during the earthquake, summarized by Earthquake Research Institute (ERI), the University of Tokyo.

K-NET Tsukidate, located in Kurihara city, Miyagi prefecture, is the only station that recorded Intensity 7 during the main shock. A maximum acceleration in the N-S direction reached almost 2699 cm/s^2, representing that the main shock caused excessively severe earthquake motions. Strong motions with PGA larger than 200 cm/s^2 were observed over a very wide area from Ibaraki to South Iwate. Tokyo is located 300 km away from the epicenter, and its stations recorded PGA of 50 to 150 cm/s^2.

The records from stations close to epicenter, such as Sendai, show two wave groups with a time interval of about 50 seconds, and those from southern station such as Ibaraki and Tokyo areas show one large wave group. Such phenomenon occurred due to difference of focal rupture process and the wave propagation to recording stations in the northern and southern portions of the fault area of 500 km long.

Building responses and records

Where ground acceleration was large, except for some areas of soft ground, the response spectrum indicates short dominant period, which was probably the main reason for relatively small seismic damage.

On the other hand, Tokyo relatively far from the epicenter was subjected to theground motion of short to long period components. Many tall buildings have been constructed for the last 40 years in Tokyo, and the shaking they experienced is much stronger than those in the past. Therefore, the response observed are believed to be the precursors for the performance of the tall buildings against the stronger shaking that will definitely occur in future.

Since some tall buildings were instrumented with accelerometers, acceleration records obtained during the earthquake would be one of the best resources to study the building responses (Kasai 2011a).

Objectives and scopes

Pursuant to these, the objective of the present paper is to clarify behavior of tall buildings in Tokyo, based on the responses recorded during the 2011 Great East Japan Earthquake. The paper will analyze acceleration records of a typical seismically-resistant building with a low damping ratio, and response-controlled buildings whose damping ratios are increased by the new technology using various types of dampers.

Virtually, most Japanese tall buildings constructed after the 1995 Great Hanshin Earthquake are response-controlled in order to protect not only the human lives but also the structural components, nonstructural components, and building functions. Therefore, effectiveness of the new technology in achieving the above-mentioned performance will be discussed.

OVERVIEW OF BUILDINGS EXAMINED

Nine tall buildings selected

We select one conventional seismically-resistant building and eight response-controlled buildings that have various types of dampers for seismic energy dissipation. All the selected buildings are over 60 meters high, commonly considered as the lower limit of height of high-rise buildings in Japan. Time history analysis is compulsory for these buildings when performing structural design. The number of floors of the buildings ranges from 19 to 54. Note that some of the buildings will be anonymous in this paper, due to the request of building owners.

In Table 1, structural types of buildings, fundamental periods of x- and y-directions, peak accelerations of top and base are given. Hereinafter, the top means the highest instrumented floor, and the base means the instrumented floor closest to the ground level. The vibration periods are obtained from the transfer function of acceleration of top to base; the frequency at the peak value of transfer function is defined as the vibration period of structure.

The peak acceleration at the base ranged from 52 to 142 cm/s^2, and their average is about 80 cm/s$^{2.}$ The peak acceleration response at top of the building ranged from 113 to 251 cm/s^2, and the average story drift angle (ratio of peak displacement of top to its height) is 1/300 rad. at most.

Table 1. Basic information on buildings examined.

No.	Type of Damper	Type of Frame	Number of Floors	Height (m)	Period (s)		Top Acc. (cm/s2)		Base Acc. (cm/s2)		Amplif. Factor	
					X	Y	X	Y	X	Y	X	Y
1	**No Damper**	**S**	**29**	**127.8**	**2.96**	**3.09**	**235**	**316**	**91**	**89**	**2.58**	**3.55**
2	Steel+Viscous	S	14	66.0	1.21	1.66	217	155	112	127	1.94	1.22
3	Viscous	S	19	79.5	1.83	1.58	142	154	75	71	1.89	2.17
4	Steel+Viscous	S	21	99.6	1.83	1.97	113	128	75	71	1.51	1.80
5	Viscous	CFT+S	37	178.0	4.96	5.21	99	145	108	92	0.92	1.58
6	Oil	CFT+S	41	186.9	3.97	4.10	118	124	53	52	2.23	2.38
7	Oil	CFT+S	42	157.3	4.78	4.31	147	152	47	68	3.13	2.24
8	Steel+Viscous	CFT+S	43	152.5	4.75	4.23	136	199	72	78	1.89	2.55
9	Oil	S	54	223.0	5.37	6.43	236	161	94	142	2.51	1.13

Note: S = Steel structure, CFT = Concrete-filled tube columns.

Spectral responses at building sites

Fig. 1 shows the response spectra of the base acceleration records of both x- and y-directions, the damping ratio of 5% is used. The nine buildings are located in Shinjuku ward, Chiyoda ward, Bunkyo ward, Minato ward, and Meguro ward of Tokyo. As can be seen from Fig. 1, response spectra have small coefficient of variation of about 0.2 at the middle to long period range.

In view of the strong randomness of earthquake motions, the intensity and characteristics of these input earthquake motions may be considered as similar ones. Consequently, it is reasonable to use the average to represent the input earthquake level in this area. In one sense, it could be considered that all the nine buildings were subjected to a common ground shaking characterized by the average spectral plots.

It should be also noted that in Tokyo and Osaka city relatively far from the epicenter, long-period earthquake motions were observed during the 2011 Great East Japan Earthquake. The spectral characteristics in Tokyo differed considerably from the well-known motion recorded during the 2004 Niigata Chuetsu Earthquake. As partly shown by Fig.1b, the spectral velocity was almost uniform for the vibration periods from 0.5s and 20s, and its magnitude exceeded even the largest spectral value due to the 2004 Niigata Chuetsu Earthquake that concentrated at the period about 7s (not shown).

Thus, unlike the responses during the 2004 quake, the responses of the tall buildings in Tokyo were dominated by not only the long period motion but also the shorter period motions during the 2011 Great East Japan Earthquake.

Amplification of acceleration

The ratio of accelerations of top to base will be named as “acceleration amplification ratio”. In Fig. 2, its value is shown with respect to each building height. In addition to the nine buildings of 14-story or higher discussed above (Table 1), two shorter response-controlled buildings with steel dampers and ten short to tall seismically-resistant buildings (Kasai 2011a) are added for comparison over wide range.

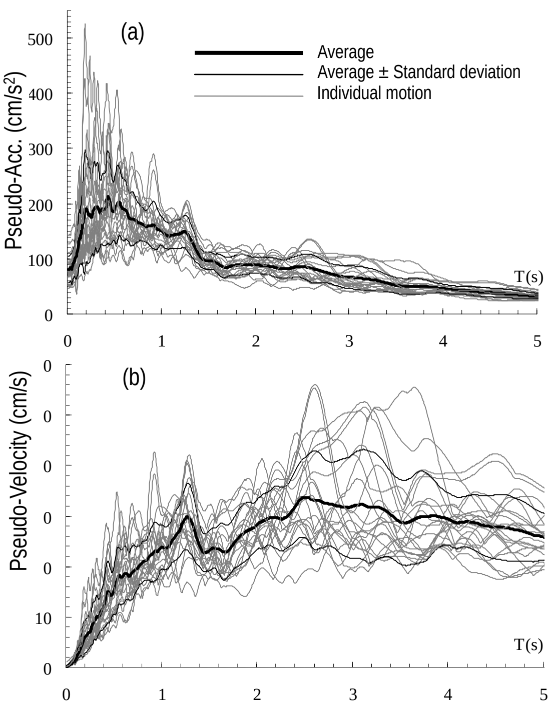
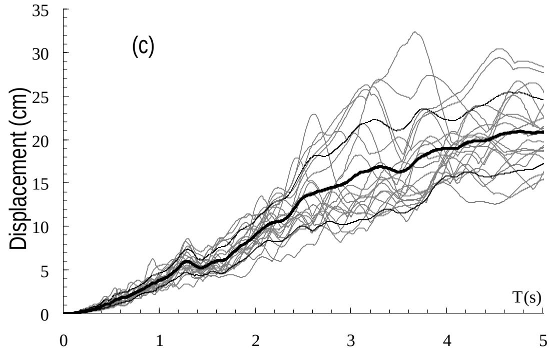



Fig. 1. Response spectra ((a) to (c)) generated from the base motions of the nine buildings (damping ratio 5%).

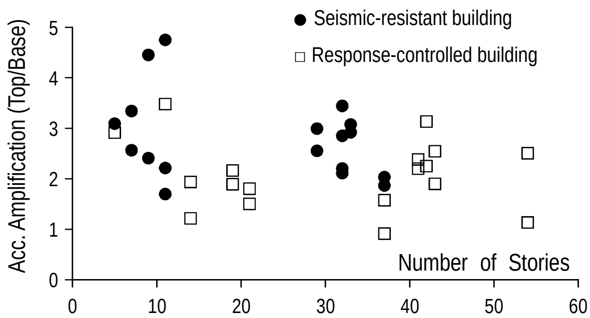


Fig. 2. Acceleration amplification factor vs. number of stories of buildings

As stated in the previous section, the response spectra for the buildings are similar, and the vibration period of the building is well correlated to its height. However, acceleration amplification ratio in Fig. 2 does not follow the trend of acceleration spectra in Fig. 1a: It is very high for taller buildings, in contrast to low spectral accelerations in Fig. 1a (Kasai, 2011a,b).

This is due to significant contribution of higher modes as will be explained in later sections, and, in case of seismic-resistant buildings, their damping ratios below the value of 5% used in the spectra of Fig. 2. Note also the shorter response-controlled buildings with steel dampers show similar trend as the seismically-resistant building, since damper was elastic or yielding very little for the level of the ground shaking in Tokyo.

In Japanese practice to-date, design criteria for response-controlled buildings have been set for displacement control, and rarely for acceleration control. In spite, excessive accelerations have been found to cause large economic loss due to the damage on non-structural components and facilities. Thus, acceleration amplification such as shown in Fig. 2 should be taken in structural design more seriously. By this reason, the following discussion will refer to both displacement and acceleration.

Calculation of displacements

By using the following two different methods, the displacements of structure are calculated from the recorded accelerations. The results are compared with each other in order to confirm their reliability (Kasai, 2011a).

Method 1 performs double integration together with hi-pass filtering in frequency domain. The cut-off frequency is typically 0.05 or 0.1Hz. Method 2 first obtains modal properties such as vibration period, damping ratio, and participation vector, by applying a basic system identification technique for the story where recording was done. Then, the time histories of acceleration and displacement of each mode are calculated by using the base acceleration recorded, and thus-obtained responses typically for modes 1 to 3 are added together. This is a so-called modal superposition analysis, and is conducted easily, without modeling numerous structural elements of the building.

For all the nine buildings considered, the displacements from method 2 agreed well with those from method 1, and accelerations from method 2 agreed with those recorded. In such cases, the modal properties obtained are considered valid, and the contribution of each individual mode to the total response of system, and the effect of damping can be furthermore examined.

Note that method 2 is based on the assumption of linear response, proportional damping, and real number mode. The agreement between the two methods suggests that the buildings had linear or slightly nonlinear behavior during the earthquake as well as moderate amount of damping. In the next sections, four selected buildings will be considered to describe in detail typical responses and modal contributions.

PERFORMANCE EXAMPLES

Building 1 (seismically-resistant conventional structure)

Building 1 is a seismically-resistant 29-story steel building constructed in 1989 (Hisada et al. 2011, 2012). It is a school building of Kogakuin University, located in Shinjuku ward of central Tokyo. The building height is 143 m, and floor plan dimension is 38.4 and 25.6 m in EW and NS directions, respectively. See Hisada et. al. (2011, 2012) for the building details and sensor locations.

As indicated in Table 1, the peak accelerations in x- and y-directions were 91 and 89 cm/s^2 at the base, and 235 and 316 cm/s^2 at the top floor, respectively. The acceleration amplification ratios are 2.58 and 3.55, respectively. The average drift angle (i.e., top floor displacement divided by the height) is 1/350 rad., and the structure remained elastic. The vibration periods for the first three modes are 2.96s, 1.00s, and 0.52s for x-direction, and 3.10s, 0.94s, and 0.47s for y-direction, respectively. Likewise, damping ratios are 1.7%, 1.8%, and 3.4% for x-direction, and 2.1%, 1.6%, and 3.4% for y-direction, respectively. Prof. Hisada of the University reported damping ratio of 0.01, based on the small amplitude vibration test which he conducted before 2011 (Hisada 2011).

Fig. 4 top shows y-direction pseudo-acceleration response spectra S_{pa} of Building 1 (*building acceleration spectrum*, solid line) and a component at building top floor *(component acceleration spectrum*, broken line) due to the y-direction accelerations recorded at the building base and top floor, respectively. Damping ratios are set to 2% and 3%, considering responses of building and non-structural component such as ceiling (Kasai 2011b), respectively. Similarly, Fig. 4 bottom shows displacement spectra S_d of Building 1 (*building displacement spectrum*, solid line) and a component (*component displacement spectrum*, broken line), respectively. The two vertical axes on two sides of each figure are in reference to responses of the Building 1 and the component, respectively.

S_{pa}'s of Building 1 at the 2nd (0.94s) and 3rd (0.47s) mode periods are large, suggesting possible higher mode contributions to the accelerations in the building. Also, S_{pa}'s of components resonant with the 1st (3.09s) and 2nd (0.94s) modes are extremely high, and their values are 1,600 and 2,400 cm/s^2, respectively. Time history analyses of such components have indicated many cycles of large accelerations.

On the other hand, S_d's of both Building 1 and component are highly dependent on the building's 1st mode period (3.09s). Note that broken line in Fig. 4 bottom indicates the component may move 400cm, if it is flexible and resonate with the building's 1st mode.

According to Hisada et al. (2011, 2012), moving of furniture, falling of ceiling and books occurred due to the high acceleration. The control of building's higher mode responses is a problem that needs to be resolved in case of a conventional seismically-resistant tall building (see also later sections*)*.

Fig. 5a compares acceleration records at top floor and base in y-direction. The earthquake duration is long, and is considered to be about 200 seconds (Fig. 5a). For the first 90 seconds of the figure, high frequency response of the top floor is apparent, as confirmed by the large number of cycles per unit time. These are caused by the high-frequency ground shaking, as shown by the base accelerations. In contrast, for the last 110 seconds, low frequency response is dominant. The ground shaking is weak (Fig. 5a), but its low frequency contents excited the first mode and caused resonated response.

Fig. 5b compares the top floor acceleration recorded with that calculated by method 2. The good agreement suggests that the mode method is effective, and the first three modes are adequate in response calculation for this case. Fig. 5c compares relative displacement of top floor obtained by double integration of the record (method 1) with that calculated by method 2. In some cycles the peak values by the both method differ a little, but the displacements agree well overall.

As is known, the contribution of each vibration mode depends on the type of response as well as the story level determining participation vector. Since the properties and responses of each vibration mode have been obtained, it is possible to discuss such contributions:

Fig. 6a shows the acceleration of each mode at the top floor. As mentioned earlier, it is dominated by the 2nd, 1st, and 3rd modes in the order of weight for the first 90 seconds. For the later 110 seconds, the 1st mode response increases and become dominant, with slight contribution from the 2nd mode. As Fig. 6b shows, for the 16th floor the 2nd mode is much more dominant, developing almost the same acceleration as top floor. As for the displacement at top floor (Fig. 6c), the 1st mode dominates throughout the entire duration.

Fig. 3. Building 1

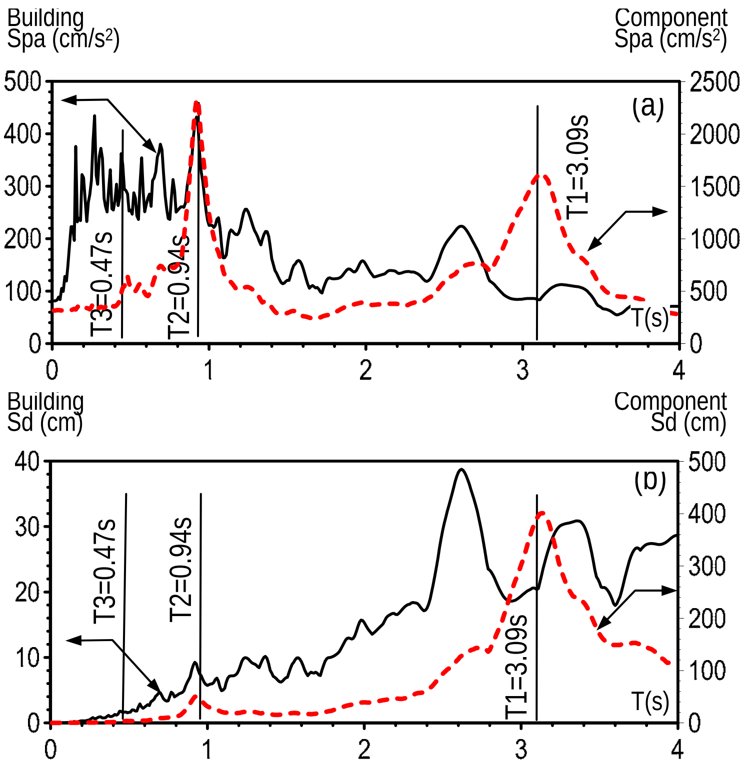
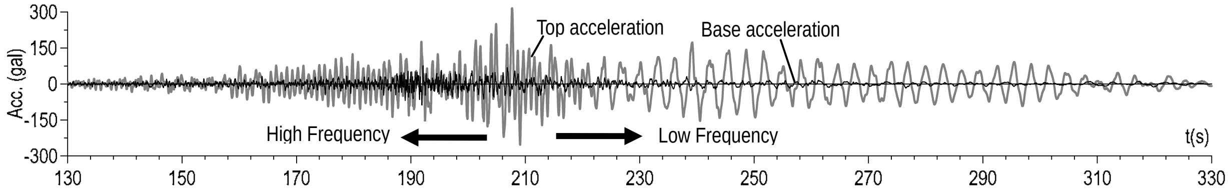
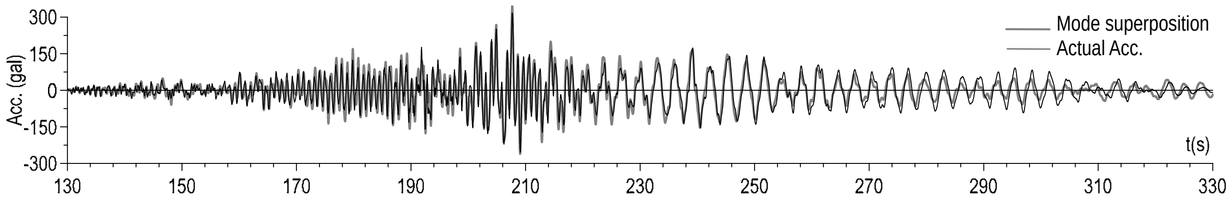


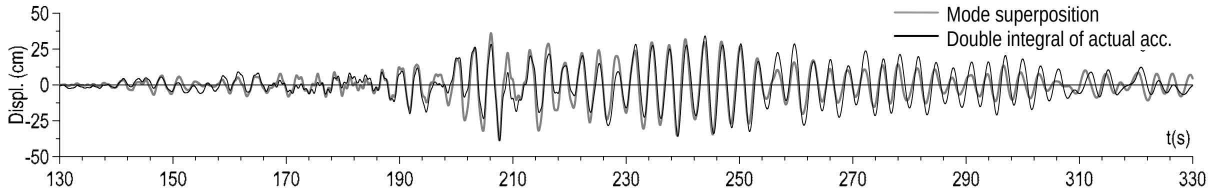
Fig. 4 Response spectra of Building 1 (solid lines) and component at top floor (broken lines).



(a) Accelerations recorded at top and base.

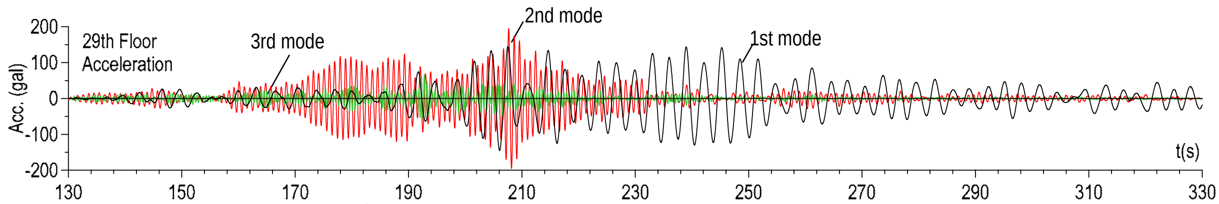


(b) Comparison: acceleration obtained by mode superposition vs. actual recorded acceleration.

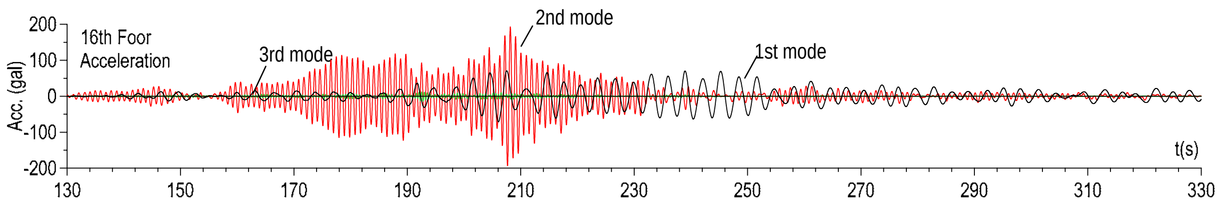


(c) Comparison: dispt. obtained by mode superposition vs. double integral of recorded acceleration.

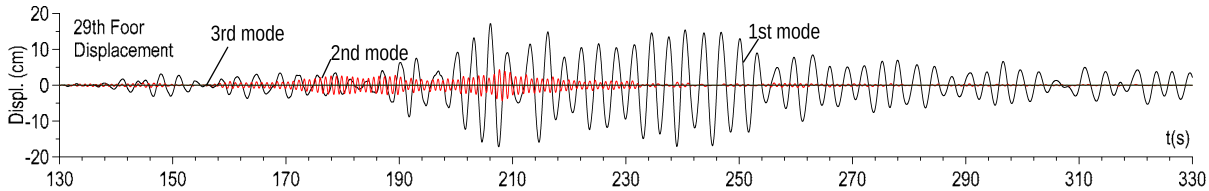
Fig. 5. Records and accuracy of mode superposition method.



(a) Modal contributions to top floor acceleration.



(b) Modal contributions to 16th floor acceleration.



(c) Modal contributions to top floor displacement.

Fig. 6. Contributions of the first three modes to acceleration and displacement of Building 1.

Buildings 4, 6, and 9 (response-controlled structures)

Building 4 is a 21-story government office building (Koyoma and Kashima 2011, Kasai 2011b). It consists of a steel frame and 336 low yield point steel (wall) dampers and 284 viscous (wall) dampers (Fig. 7a). For seismic energy dissipation, the steel damper utilizes yielding of steel material, and viscous damper utilizes flow resistance of the polymer liquid with high viscosity (Kasai et al. 2009). Note that a contrasting case of using only steel dampers lead to large accelerations like the seismically-resistant building in the previous section, since the damper remained elastic for the level of shaking in Tokyo (Kasai et al. 2011b). Building 4 had been designed to avoid such a situation, expecting that viscous damper would dissipate energy from a small earthquake, and steel damper, the most economical among all types, would dissipate considerable amount of energy at a large quake, respectively.

Building 6 is a 41-story office building (Kasai et al. 2009, 2011b). It consists of a frame using concrete-filled tube columns and steel beams, and 688 oil dampers (Fig. 7b). For energy dissipation, the damper utilizes flow resistance of oil with low viscosity. The valve placed at orifice is shaped to produce the force linearly proportional to velocity, but a relief mechanism to limit the force is provided, making the hysteresis to switch from an elliptical shape to a rectangle shape. Most likely, the relief did not occur for the level of shaking.

Building 9 is a 54-story office steel building constructed in 1979. It was retrofitted in 2009 (Maseki et al. 2011) by attaching 288 oil dampers (Fig. 7c). 12 dampers per floor were attached to middle 24 stories of the building. The oil damper is similar to those used for Building 6, except that its relief mechanism is modified to reduce forces near peak responses. This aims to reduce the axial force of the column transmitting the damper force, and consequently uplift force of foundation. Most likely, however, the relief did not occur for the level of shaking.

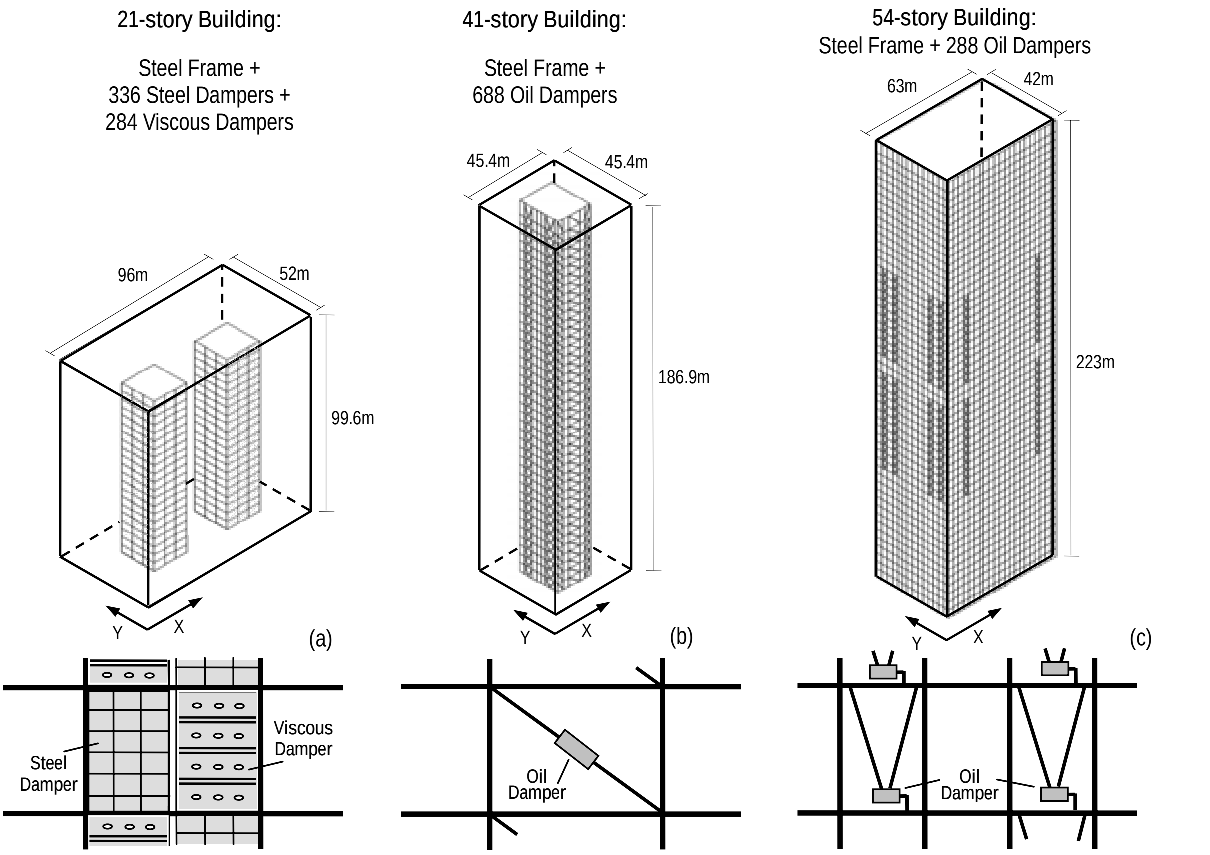


Fig. 7. Response-controlled buildings and dampers considered (Buildings 4, 6, 9 from left).

As indicated in Table 1, average of acceleration amplification ratios of Buildings 4, 6, and 9 is less than 2, well below those of Building 1, and they remained elastic. Modal properties are obtained from method 2, and estimated 1st mode damping ratios are about 4%, and those of the 2nd and 3rd modes are almost equal or larger. The 1st mode vibration periods are indicated in Table 1, and those up to the 3rd mode will be mentioned later.

For all the three buildings, their accelerations and displacements are obtained from superposition up to the 3rd mode, and accuracies are confirmed as in to be even better than those shown in Figs. 5b and c shown earlier. Such responses at top floor are shown by black lines in Figures 8, 9, and 10 for buildings 4, 6, and 9, respectively.

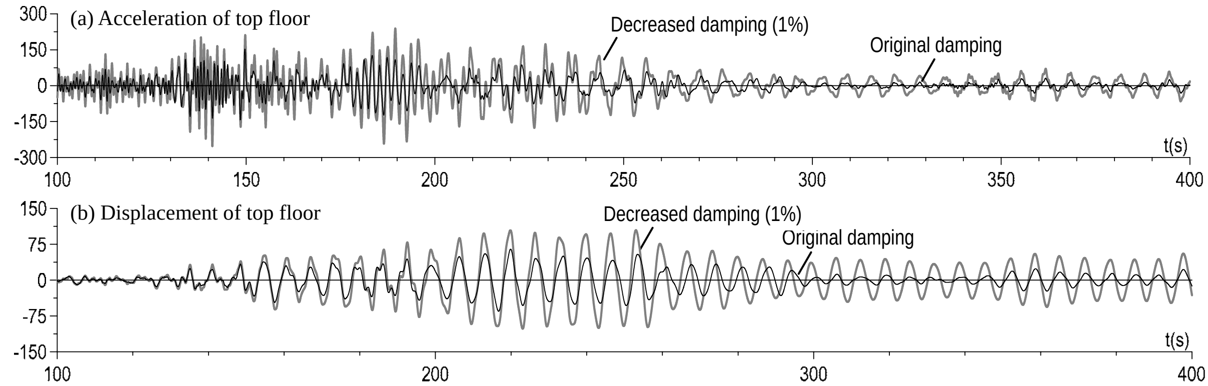


Fig. 8. Building 4 with different damping ratios (y-dir.).

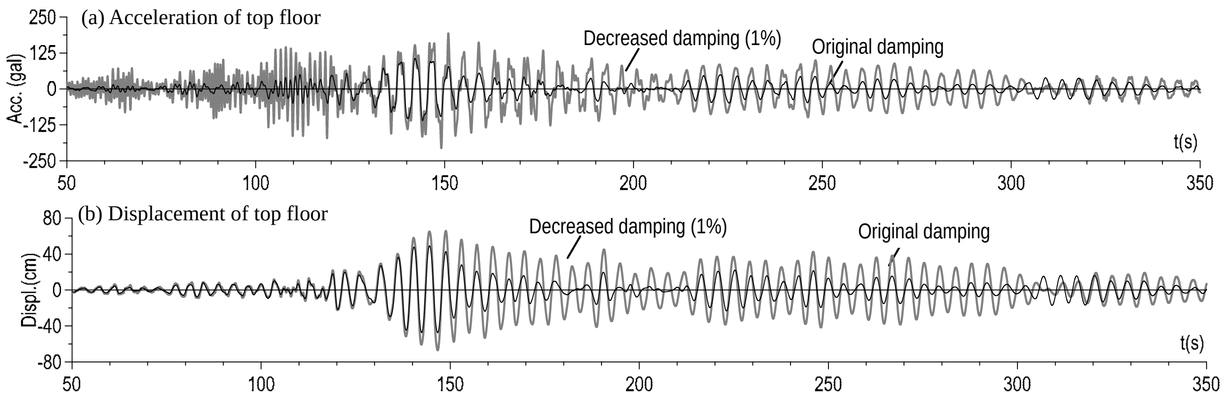


Fig. 9. Building 6 with different damping ratios (x-dir.).

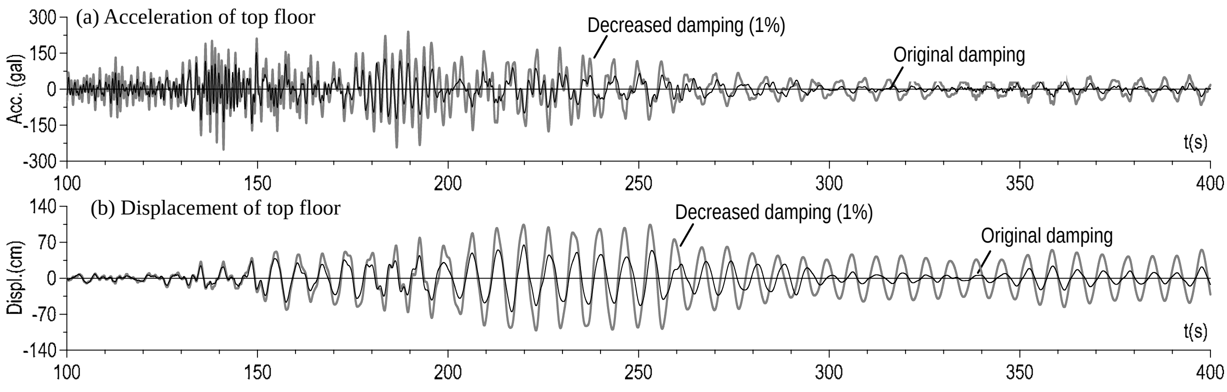


Fig. 10. Building 9 with different damping ratios (y-dir.).

In these three buildings, the acceleration (Figs. 8 to 10) is dominated by the 2nd and 3rd modes for about 100 seconds, and by the 1st mode for later 200 seconds. Whereas, the displacement (Figs. 8 to 10) is dominated by the 1st mode throughout the shaking.

This trend is like that of seismically-resistant Building 1, but the amplitudes are believed to be smaller due to the supplemental damping. Thus, the responses are compared with those of lower but possible damping ratio representing a hypothetical case of not using the dampers. The modal period is unchanged, assuming small stiffness of the damper. The 1st to 3rd mode damping ratios are uniformly set to 1% and superposition is repeated. The results are shown by gray lines in Figs. 8, 9, and 10 for Buildings 4, 6, and 9, respectively.

In all the three buildings, their responses are considerably smaller (black lines) than those with low damping (gray line). The peak accelerations and displacements are about 0.5 and 0.7 times those of the low damping case. Moreover, between significant ground shakings, the responses decay much faster, and number of large cycles is reduced considerably. These help reducing damage and fatigue of structural and non-structural component as well as fear or discomfort of the occupants. In order to quantify such an effect, root mean square of the acceleration and displacement at top are calculated, and their values appear to be about 0.4 and 0.5 times those with low damping, respectively.

COMMENTS ON ACCELERATION AND NONSTRUCTURAL DAMAGE

Inertia forces against structural and non-structural components including equipment and building content are produced by accelerations in the building. Large accelerations typically developed at upper stories cause falling, overturning, shifting, crashing, rapture, and excessive vibration of a variety of non-structural components.

As a matter of fact, economic loss due to damage of non-structural components is much more than that of structural damage. Falling of ceilings and other components may also cause death of occupants. Such failures due to the 2011 Great East Japan Earthquake were enormous.

Figure 11 shows component acceleration spectra for the top floors of the four buildings 1, 4, 6, and 9. Damping ratio of the component is assumed to be 3%. The value attached to "original damping" is the first mode damping ratio. For Building 1 (Fig. 11a) that is seismically resistant, the broken line is based on the recorded top floor acceleration of the original building having low damping ratios as mentioned earlier, and solid line shows a case where the building damping ratios of the first three modes are increased to 4%. In contrast, for Buildings 4, 6, and 9 (Fig. 11b-d) that are response-controlled, the solid line is based on recorded top floor acceleration of the original building (Figs. 8 to 10), and the broken line shows when the first three modal damping ratios of the building are reduced to 1%. These examine a merit of increasing building modal damping ratios for protecting the acceleration-sensitive components.

According to Fig. 11, the past belief that short-period components are safer in a tall building is incorrect. They are as vulnerable as the long-period components due to multiple resonance peaks created by different modes of the building. The peaks are extremely high, even greater than 2,000 cm/s^2 (≈2G). Thus, the resonant acceleration of the components may be greater than 8G at a so-called major quake 4 times or stronger. The problem may become more serious when damage and softening of components cause period shifting from one resonance peak to others. Note that three peaks for each building are shown in Fig. 11, since the first three modes were identified. But more peaks may emerge in an actual low damping case.

As a rule of thumb, facilities may overturn when floor acceleration exceeds 0.3G, and ceiling whose vibration period typically ranges from 0.3s to 1s may fall when its acceleration response exceeds 1G. These indicate the needs for an immediate attention to component responses at a major quake that will occur in Tokyo. Fig. 11 also clearly indicates that even moderately increasing the building damping ratio by 3% or so would reduce the component acceleration considerably.

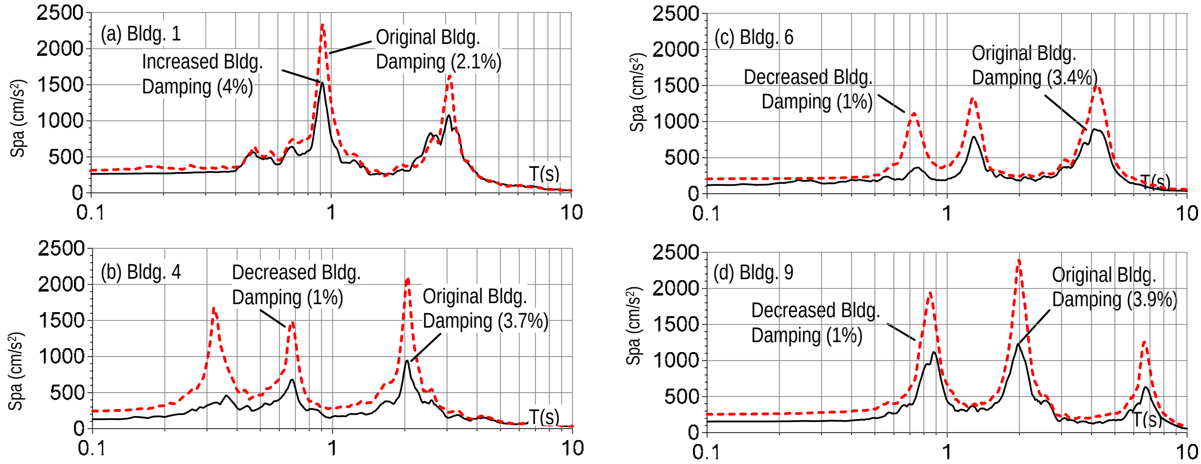


Figure 11. Component response spectra (component damping ratio = 3%).

CONCLUSION

Responses of the tall buildings in Tokyo during the 2011 Great East Japan Earthquake are discussed based on the strong motions recorded. By successfully analyzing contributions of multiple vibration modes, various shaking phenomena in the tall building that had not been experienced are clarified.

Based on this approach, the merit of damping technology for occupants and contents in the tall building is explained. Immediate attention must be given to the acceleration-induced hazard in tall buildings, considering much stronger shaking likely to occur in the near future.

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