Response of Conventional Seismic-resistant Tall Buildings in Tokyo During 2011 Great East Japan Earthquake

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SUMMARY:
Many tall buildings in Tokyo metropolitan area were strongly shaken during the 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 base peak horizontal accelerations were up to 150 cm/s^2, more than one-third of the one representing the major earthquake stipulated by the Japanese seismic design code. The tall buildings are of steel construction typically, concrete construction, or hybrid construction. Some of the tall buildings are instrumented with sensors and their motions recorded during the event. This paper discusses the responses of the conventional seismic-resistant tall buildings in Tokyo based on their motions recorded during the 2011 East Japan Earthquake. This kind of buildings generally has low damping ratios, and experienced both large displacement and large acceleration during the earthquake. The modal properties such as vibration period, damping ratio and participation function are identified, and the displacements of structures are obtained in terms of two different methods based on and they are compared to each other.

Keywords: 2011 Great East Japan Earthquake, structural response, seismically-resistant tall building

1. INTRODUCTION

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. Strong motions with PGA (peak ground acceleration) 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.

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, which is relatively far from the epicenter, was subjected to the ground 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. Some tall buildings were instrumented with accelerometers, so acceleration records obtained during the earthquake would be one of the best resources to study the building responses (Kasai, 2011a).

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 4 typical seismically-resistant tall buildings with relatively low damping ratios. The response-controlled buildings whose damping ratios are increased by the new technology using various types of dampers will be discussed in a companion paper of this conference (Kasai et al. 2012).

2. OVERVIEW OF BUILDINGS EXAMINED

2.1. *Three Tall Buildings Selected*

In this paper, three conventional seismically-resistant buildings are selected. All the selected buildings are higher than 60 meters, which is commonly considered as the lower limit of height of high-rise buildings in Japan. Time history analysis is compulsory for these buildings during structural design. The number of floors of the buildings ranges from 19 to 37. Note that some of the buildings considered in this paper will be anonymous, considering the request of building owners.

Table 1 shows structural types of buildings, fundamental periods of x- and y-directions, peak accelerations of top and base. Hereinafter, the top means the highest floor where the accelerometer is installed, and the base means the lowest floor in structure 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 natural frequency of structure.

Table 1. Basic information on buildings examined

No.	Type of Main 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	S	29	127.8	2.96	3.09	235	316	91	89	2.58	3.55
2	S	20	86.5	1.97	1.87	210	150	91	85	2.31	1.75
3	S	19	84.7	1.82	1.95	290	385	66	69	4.38	5.58

2.2. *Spectral Responses at Building Sites*

According to statistic data of base accelerations recorded in ten building sites (Kasai et al. 2012), 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 150 to 385 cm/s^2, and the average story drift angle (ratio of peak displacement of top to its height) is 1/316 rad. at most. Response spectra of base acceleration recorded in Tokyo area have small coefficient of variation of about 0.2 at the middle to long period range.

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 understood from Figure 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.

2.4. *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 accelerations and displacements of mode 1 to 3 are calculated by using the base acceleration recorded, and modal superposition is performed.

For all the three seismically-resistant buildings considered in this paper, the displacements from method 2 agreed well with those from method 1, and accelerations from method 2 agreed with those recorded. Thus, 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 Chapter 3, the three selected buildings will be described in detail.

3. PERFORMANCE EXAMPLES

3.1 *Building 1*

Building 1 is a seismically-resistant 29-story steel building constructed in 1989. It is a school building of Kogakuin University, located in Shinjuku ward of central Tokyo. Figure 1 shows the elevation and plan of the building as well as locations of sensors instrumented (Hisada, 2011).

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 ratio of accelerations of top to base is named as "acceleration amplification ratio". The acceleration amplification ratios for building 1 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).

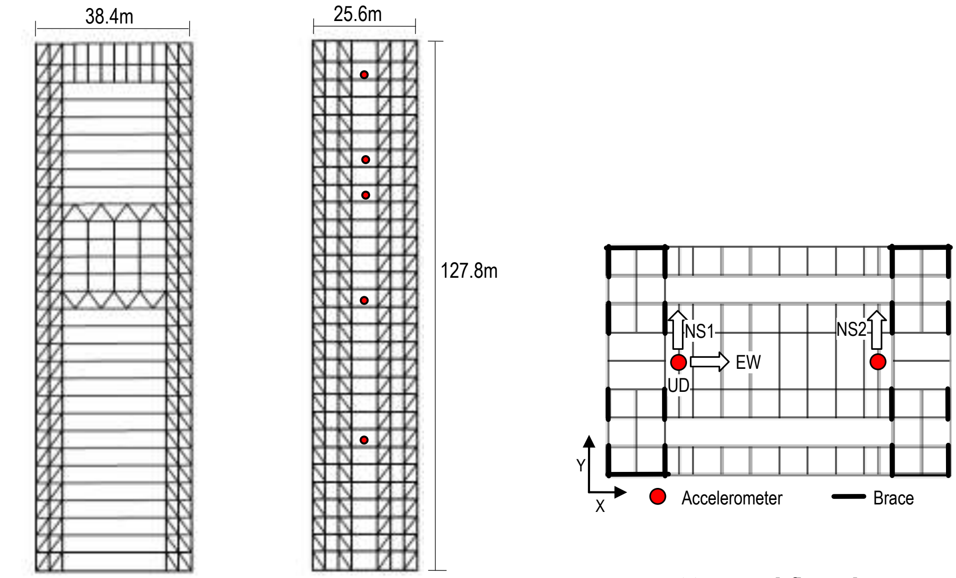
Figure 2a shows y-direction pseudo-acceleration response spectra S_{pa} of Building 1 (*building acceleration spectrum*, solid line) and a component at building top floor *(component acceleration spectrum*, broken line) due to the y-direction accelerations recorded at the building base and top floor, respectively. Damping ratios are set to 2% and 3%, considering responses of building and non-structural component such as ceiling (Kasai 2011b), respectively. Similarly, Figure 2b 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 Figure 2b indicates the component may move 400cm, if it is flexible and resonate with the building's 1st mode.

Figure 3a 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. 3a). 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. 3a), but its low frequency contents excited the first mode and caused resonated response.

Figures 3b 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. Figure 3c 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.



(a) X-direction frame (b) Y-direction frame (c) Typical floor plan

Figure 1. 29-story building (Building 1).

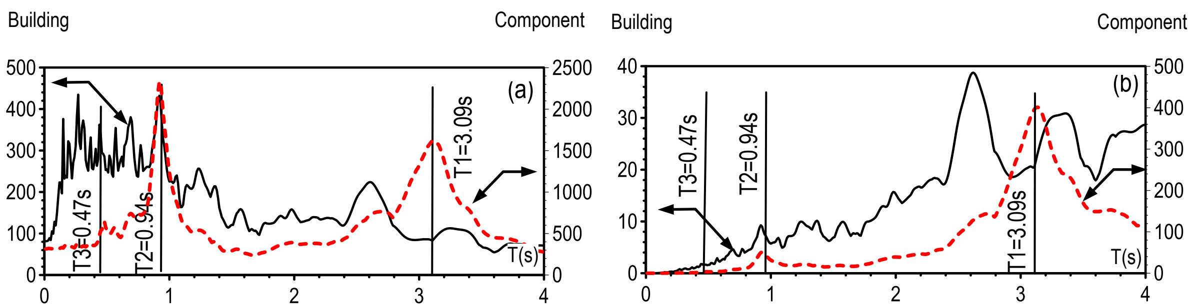


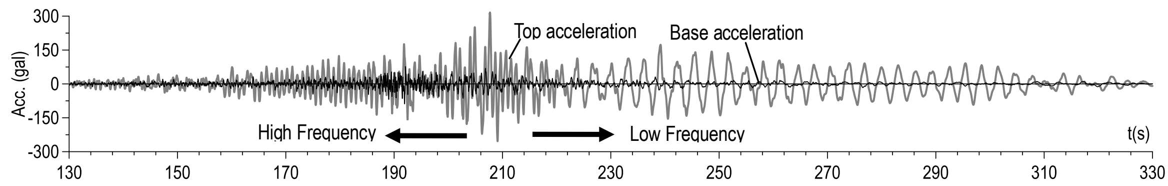
Figure 2. Response spectra of building (solid lines) and component at top floor (broken lines) of Building 1.

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:

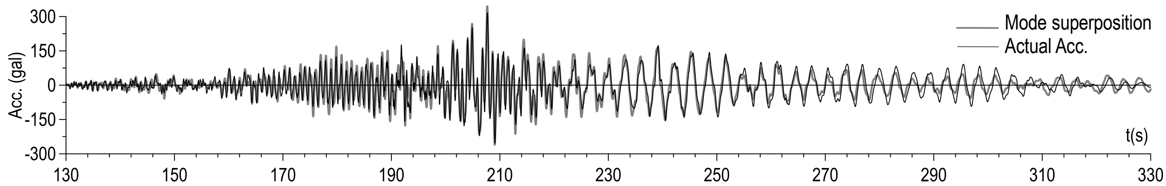
Figure 4a shows the acceleration of each mode at the top floor. As mentioned earlier, it is dominated by the 2nd, 1st, and 3rd modes in the order of weight for the first 90 seconds. For the later 110 seconds, the 1st mode response increases and become dominant, with slight contribution from the 2nd mode. As Figure 5b 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. 4c), the 1st mode dominates throughout the entire duration.

3.2 *Buildings 2, 3 and 4*

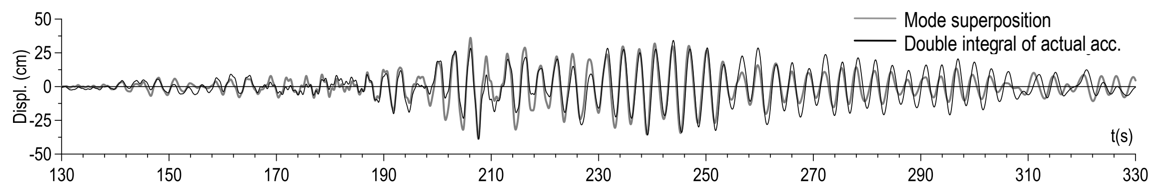
Building 2 is a 20-story steel building constructed in 1994, and served as a government building. The accelerometers are instrumented in the 1st floor and 19th floor, respectively. As indicated in Table 1, the peak accelerations in x- and y-directions were 91 and 85 cm/s^2 at the base, and 210 and 150 cm/s^2 at the top floor, respectively. The acceleration amplification ratios are 2.31 and 1.75, respectively. The average drift angle (*i.e.*, top floor displacement divided by the height) is 1/671 rad., and the structure remained elastic. The vibration periods for the first three modes are 1.98s, 0.68s, and 0.38s for x-direction, and 1.86s, 0.62s, and 0.36s for y-direction, respectively. Likewise, damping ratios are 1.9%, 4.1%, and 3.9% for x-direction, and 2.3%, 5.5%, and 3.2% for y-direction, respectively.



(a) Accelerations recorded at top and base.

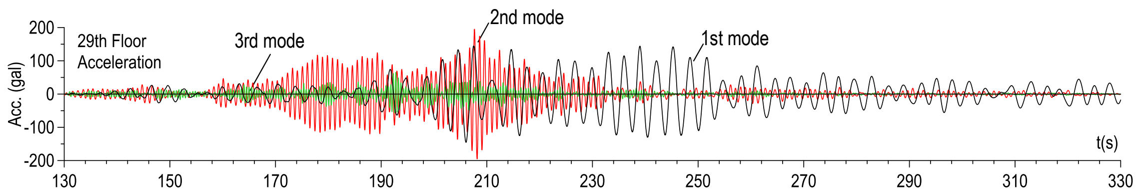


(b) Comparison: acceleration obtained by mode superposition vs. actual recorded acceleration.

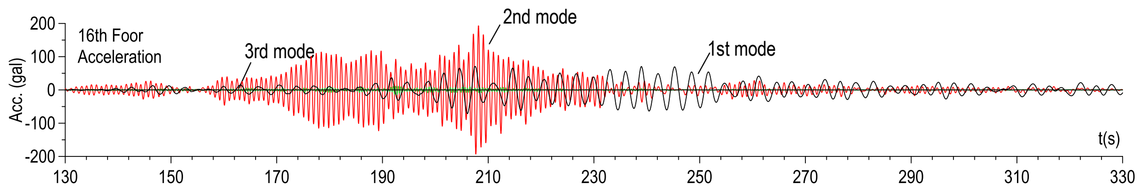


(c) Comparison: dispt. obtained by mode superposition vs. dispt. from double integral of recorded acceleration.

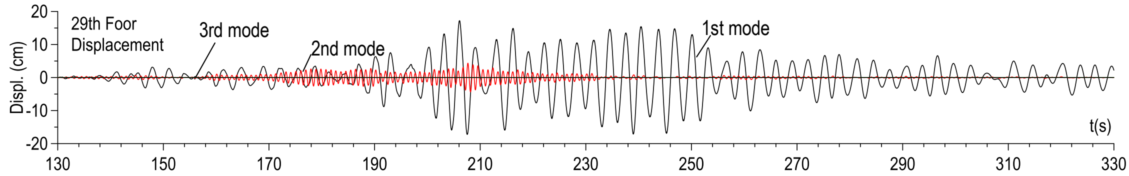
Figure 3. Acceleration records and accuracy of mode superposition method.



(a) Modal contributions to top floor acceleration.



(b) Modal contributions to 16th floor acceleration.



(c) Modal contributions to top floor displacement.

Figure 4. Contributions of the first three modes to acceleration and displacement of Building 1.

Building 3 is a 19-story steel building constructed in 1990, and also served as a government building. As shown in Table 1, the peak accelerations in x- and y-directions were 66 and 69 cm/s^2 at the base, and 290 and 385 cm/s^2 at the top floor, respectively. The acceleration amplification ratios are 4.38 and

5.58, respectively. The average drift angle is 1/316 rad. The vibration periods for the first three modes are 1.82s, 0.58s, and 0.33s for x-direction, and 1.95s, 0.66s, and 0.40s for y-direction, respectively. Likewise, damping ratios are 3.0%, 3.0%, and 5.0% for x-direction, and 2.4%, 3.1%, and 3.7% for y-direction, respectively.

For all the buildings examined, 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 Figures 3b. Such responses at top floor are shown by black lines in Figures 5 and 6 for buildings 2 and 3, respectively. In these three buildings, the acceleration (Figs. 5 and 6) 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. 5 and 6) is dominated by the 1st mode throughout the shaking.

To improve the seismic performance of such kind of seismically-resistant buildings, retrofitting of structures by adding damping devices has been widely used. Thus, the responses are compared with those of higher but possible damping ratio representing a hypothetical case of 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 5% (8% for building 4) and superposition is repeated. The results are shown by red lines in Figures 5 and 6 for buildings 2 and 3, respectively.

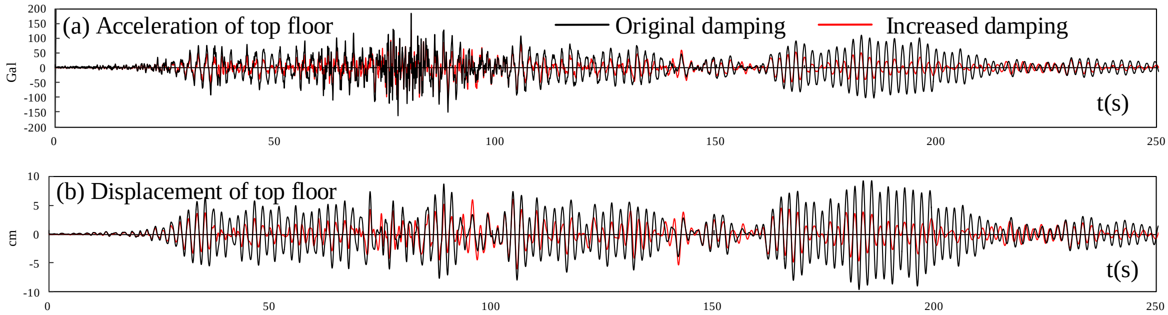


Figure 5. Response of building 2 with different damping ratios (y-dir.)

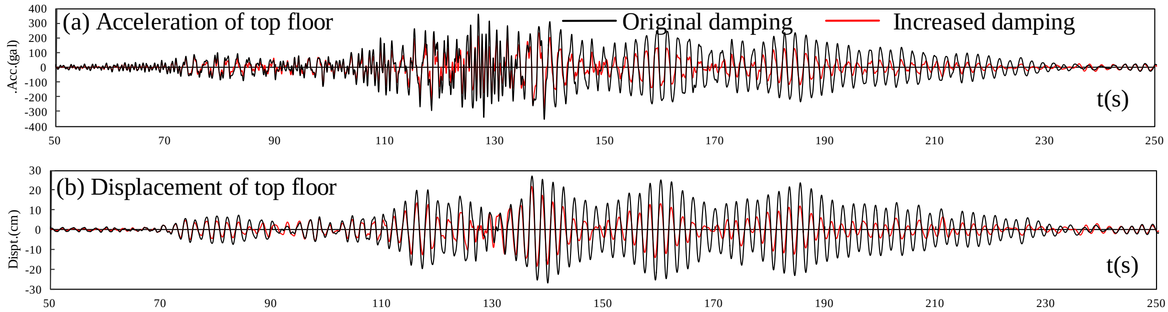


Figure 6. Response of building 3 with different damping ratios (y-dir.)

3.3 *Summary of response of seismically-resistant buildings*

In all the three buildings, their responses are considerably larger (black lines) than those with high damping (red line). The peak accelerations and displacements are about 1.19 and 1.20 times those of the high damping case. Moreover, between significant ground shakings, the responses of high damping cases 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 1.25 and 1.37 times those with high damping, respectively.

Table 1 indicates very high acceleration amplification ratios for taller buildings, in contrast to the low spectral accelerations for the long vibration period. This is due to contribution of higher modes, and, in case of seismic-resistant buildings, the damping ratios is much small, large acceleration response was observed in such kind of tall buildings. As a matter of fact, in structural design of buildings, design criteria for displacements are usually set, but that for accelerations have rarely been explicitly stated to-date. In spite, it has been recognized that the excessive acceleration can cause large economic loss due to the damage on the non-structural components and facilities. Obviously, the effect of acceleration amplification in structure should be taken in structural design more seriously.

4. DISCUSSIONS

4.1. *Effect of long-period earthquake*

The soil base of Tokyo plains is composed of soft sedimentary layers, and the specific long period components of earthquake wave may be amplified by these soft layers, and subsequently cause undesirable resonance of long period structures such as tall buildings and base-isolated buildings.

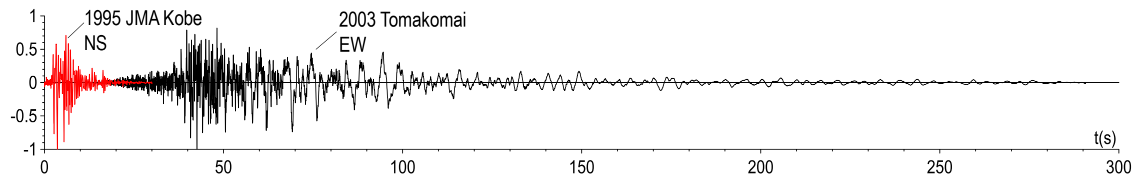


Figure 7. Normalized acceleration time histories of Tomakomai (black) and JMA Kobe (red) records.

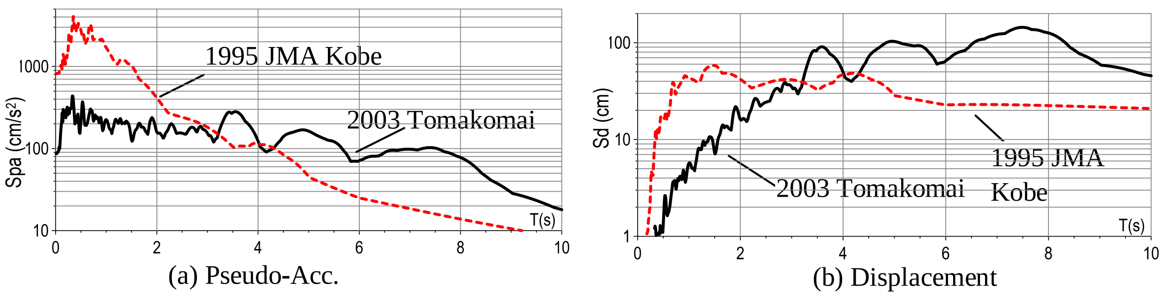


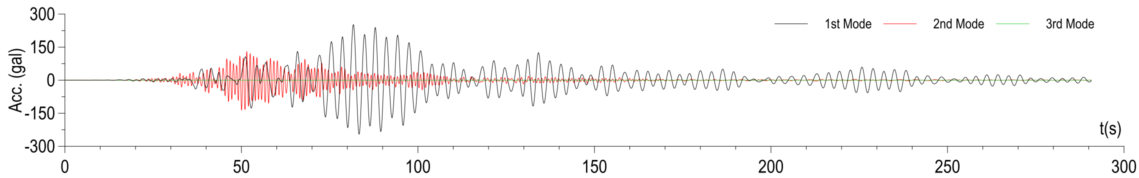
Figure 8. Pseudo-acc. spectra and displacement spectra for JMA Kobe and Tomakomai records (damping ratio = 2%).

The supposed velocity spectrum of the long-period earthquake at damping ratio of 5% is 80 to 120 cm/s, 3 to 4 times than recorded in Tokyo during the 2011 Great East Japan Earthquake. The Japanese government is striving to find social and technical solutions to the long period earthquakes.

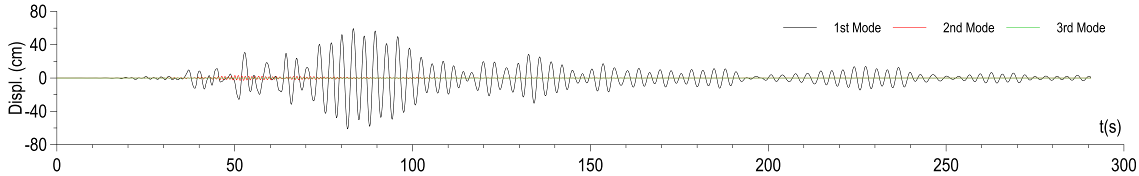
The accurately time history analyses superposing the modal response are utilized here for examining the effects of the long period earthquake. A well-known EW-direction ground motion recorded in Tomakomai during the 2003 Tokachioki Earthquake is used (Figure 7).

From Figure 8a where damping ratio is 2%, the S_{pa} exceeds the PGA (72.9 cm/s^2) in a wide period range up to 9s, suggesting significant higher mode contributions to the building accelerations. The S_d on the other hand shows mostly the 1st mode contribution to the building displacement.

These are in conformity with the peak responses summarized in Table 2: In case of Building 1 having low damping, high acceleration amplification ratio of 3.65 is obtained, and acceleration history contains multiple mode contributions. Compared with the values from the 2011 Great East Japan Earthquake, the average drift angle is 1/212 and 1.65 times, although PGA is smaller and 0.82 times. Figure 9 shows the contribution of first three modes to the acceleration and displacement of top story of Building 1.

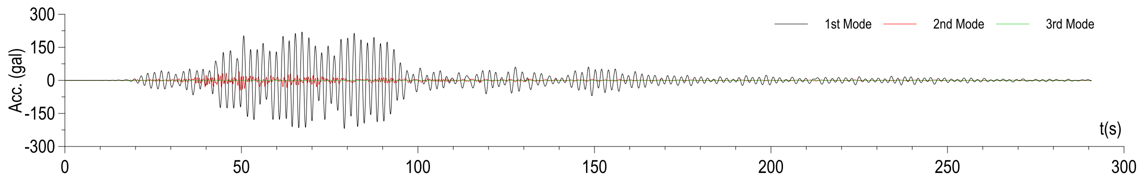


(a) Modal contribution to acceleration

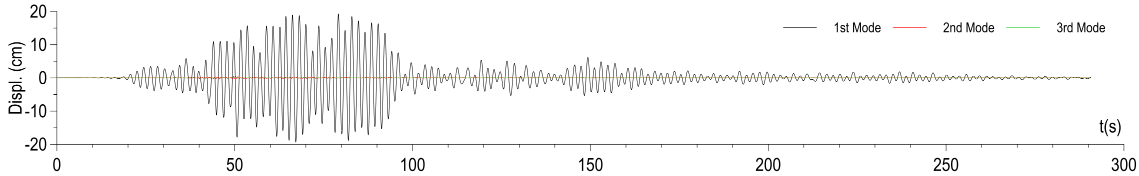


(b) Modal contribution to displacement

Figure 9. Contributions of the first three modes to acceleration and displacement of Building 1

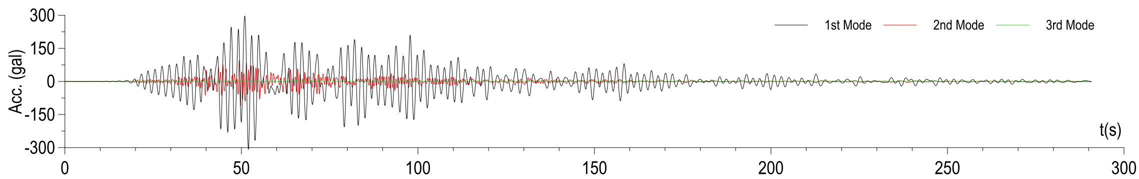


(a) Modal contribution to acceleration (unit: cm/s/s)

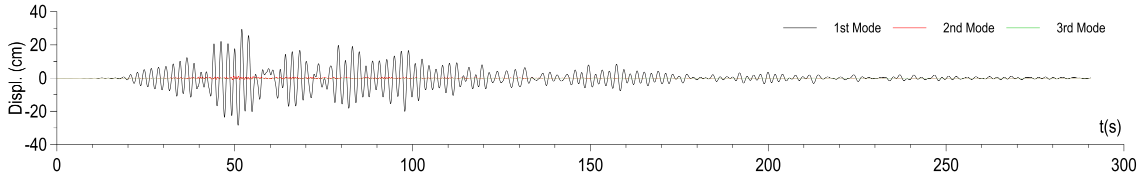


(b) Modal contribution to displacement (unit: cm)

Figure 10. Contributions of the first three modes to acceleration and displacement of Building 2



(a) Modal contribution to top acceleration (unit: cm/s/s)



(b) Modal contribution to top displacement (unit: cm)

Figure 11. Contributions of the first three modes to acceleration and displacement of Building 3

Significant responses are also found from other two buildings. Figure 10 and Figure 11 show the modal contribution to top acceleration and displacement for building 2 and building 3, respectively. Based on these, it is important to realize the possible large accelerations during the long period earthquake due to excitation of multiple modes, in addition to commonly concerned large displacement due to the 1st mode excitation. In addition, the long period earthquake has long duration, which causes long vibration of structures. Conventional seismically resistant buildings has low damping ratios, the decay of structural vibration takes much long time. In such cases, the fatigue of structural members become serious and should be pay much attention.

4.2. *Effect of near-field earthquake*

Near-field earthquake is another major concern in Tokyo. It has a short duration but strong intensity, and the pulse-like ground motion destroyed numerous infrastructures and buildings during the 1995 Great Hanshin Earthquake. Near-field earthquake has shorter duration compared above mentioned long-period earthquake, but generally causes large intensity of ground motions in building sites. One of the three strongest ground motions recorded during the 1995 quake, the NS component of JMA Kobe record (PGA 821 cm/s^2) is used here. As Figure 7 shows, its duration is much shorter than the long-period motion.

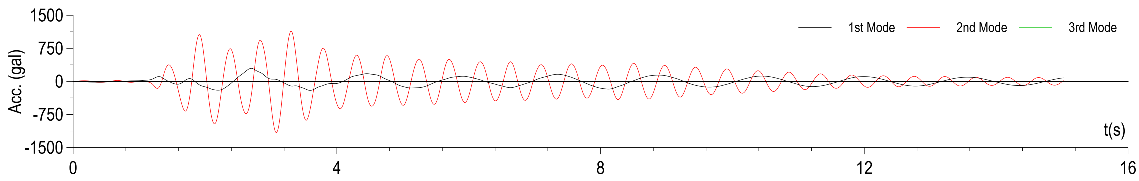
In Figure 8a, S_{pa} of JMA Kobe record is compared with that of Tomakomai record, where much higher S_{pa} at short period suggests strong higher mode contribution to the building acceleration. The trend of S_{pa} makes S_d the largest at the period of about 1.5s, and smaller at longer periods. These suggest the average drift angle, similar to S_d divided by effective height of the building, decreases for a taller building.

These match well with the peak responses summarized in Table 2: Building 1 shows high acceleration of 1,532 cm/s^2 at top floor, and time history (Figure 12) indicates that the contribution of the 2nd mode is by far the largest. Similar trend is observed from other two tall buildings (Figures 13 and 14), particularly Building 3. As Figure 14a shows, contribution of 2nd mode to top acceleration is very large, especially in the first 5 seconds; in the last 10 seconds, the higher mode decay very fast, and the first mode is dominant.

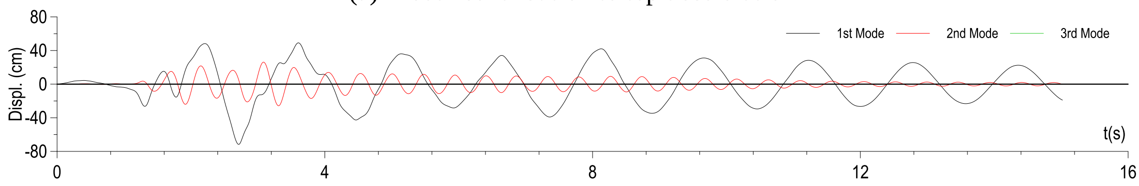
Table 2. Response of buildings subjected to JMA Kobe and Tomakomai ground motions.

	JMA Kobe NS (1995)			Tomakomai EW (2003)		
Bldg.	Dispt. (cm)	Avg. Drift Angle	Acc. (cm/s2)	Dispt. (cm)	Avg. Drift Angle	Acc. (cm/s2)
No.1	72.7	1/176	1532	60.4	1/212	266
No.2	69.4	1/125	1292	19.3	1/448	237
No.3	114.4	1/74	2913	27.7	1/306	303

Note: Avg. Drift Angle = Top Floor Displacement/Height

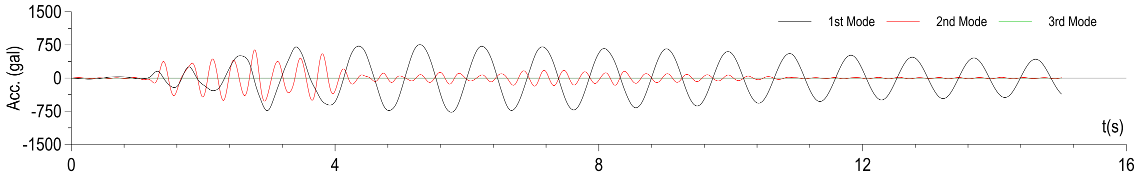


(a) Modal contribution to top acceleration

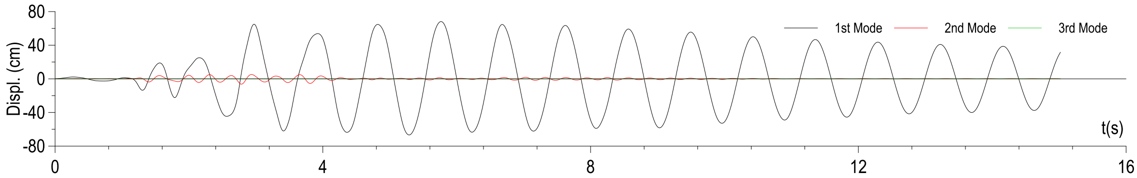


(b) Modal contribution to top displacement

Figure 12. Contributions of the first three modes to acceleration and displacement of Building 1

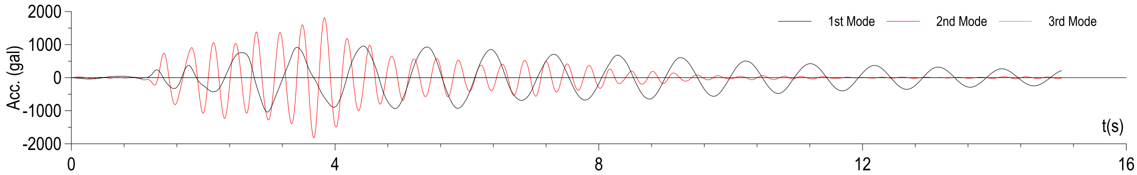


(a) Modal contribution to top acceleration (unit: cm/s/s)

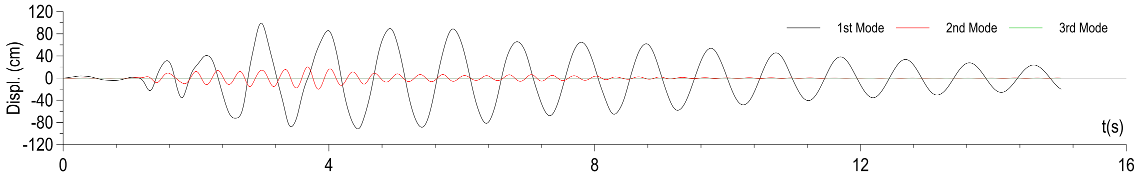


(b) Modal contribution to top displacement (unit: cm)

Figure 13. Contributions of the first three modes to acceleration and displacement of Building 2



(a) Modal contribution to top acceleration (unit: cm/s/s)



(b) Modal contribution to top displacement (unit: cm)

Figure 14. Contributions of the first three modes to acceleration and displacement of Building 3

In summary, the near-field motion did not excite the 1st mode of the tall buildings, since it has mainly short to medium long period contents. This resulted in relatively small story drift, but large accelerations of higher modes. The responses could be more significant without the supplemental damping provided by the velocity-dependent dampers used.

5. CONCLUSION

Responses of the conventional seismic-resistant 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. Conventional seismic resistant tall buildings experienced large acceleration due to high order modes' contribution. 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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