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Mehmet Celebi
Earthquake Science Center, USGS, celebi.talas57@gmail.com

Izuru Okawa
Building Research Institute, Tsukuba, Japan

Toshidate Kashima
Building Research Institute, Tsukuba, Japan

Shin Koyama
Building Research Institute, Tsukuba, Japan

Masanori Iiba
Building Research Institute, Tsukuba, Japan

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Response of a tall building far from the epicenter of the 11 March 2011 M 9.0 Great East Japan earthquake and aftershocks

Mehmet Çelebi,[1]* Izuru Okawa,[2] Toshidate Kashima,[2] Shin Koyama[2] and Masanori Iiba[2]

[1]*Earthquake Science Center, USGS, Menlo Park, California 94025, USA*
[2]*Building Research Institute, Tsukuba, Japan*

SUMMARY

The 11 March 2011 M 9.0 Great East Japan earthquake generated significant long-duration shaking that propagated hundreds of kilometers from the epicenter and affected urban areas throughout much of Honshu. Recorded responses of a tall building at 770 km from the epicenter of the mainshock and other related or unrelated events show how structures sensitive to long-period motions can be affected by distant sources. Even when the largest peak input motions to the building is about 3% *g*, the strong-shaking duration was about 140 s. The 300- to 1000-s prolonged responses of the building are primarily due to a combination of site resonance (e.g. structural fundamental frequency ~0.15 Hz and site frequency ~0.13–0.17 Hz) and low damping (~1–2%) of the structure. Response modification technologies can improve the response of the building during future earthquakes. The need-to-consider risks to such built environments from distant sources are emphasized. Copyright © 2012 John Wiley & Sons, Ltd.

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KEY WORDS: long-period structure; resonance; low damping; site frequency; recorded response; Tohoku earthquake

1. INTRODUCTION

The 11 March 2011 M 9.0 Great East Japan earthquake (also known as the Tohoku event of 2011) occurred at 05:46:23 UTC (local time 14:46:23) offshore from the east coast of Honshu, Japan (38.322°N, 142.369°E) at a depth of 32 km (http://earthquake.usgs.gov/earthquakes/eqinthenews/2011/usc0001xgp/, last accessed July 15, 2011). Through 15 July 2011, the period included in this study, there were four large aftershocks with magnitudes ranging from 7.0 to 7.7, the largest of which occurred about 30 min after the mainshock (Table 1). The earthquake caused a major disaster in Japan and affected economies throughout the world. It generated one of the most significant tsunamis, a tsunami that left its mark by destroying the four-unit Fukushima nuclear power plant and by causing the largest percentage of the 15 776 fatalities[1] associated with this event. Furthermore, it caused widespread destruction and damage of major port and other facilities on a wide portion of the north–east coast of the main island of Honshu (Japan). It is widely reported that material loss may reach $300B.

Aside from the disaster, however, copious data on the earthquake were collected, which present new opportunities for research and learning opportunities on all aspects of earthquake science and earthquake engineering. One of the more significant characteristics of the earthquake from an engineering perspective is the long-duration strong shaking over large distances that affected the built environment. In particular, tall buildings and long-period structures, such as long-span suspension and cable-stayed bridges, located at hundreds of kilometers distance from the epicenter were strongly affected.

*Correspondence to: Mehmet Çelebi, Earthquake Science Center, USGS, Menlo Park, CA 94025, USA
E-mail: celebi.talas57@gmail.com

[1]From The Japanese National Police Agency (2011) (http://www.npa.go.jp/archive/keibi/biki/higaijokyo.pdf) as confirming 15 597 deaths, 5694 injured and 4980 people missing across 18 prefectures, as well as over 125 000 buildings damaged or destroyed.

Table 1. Events and particulars of records from the building (http://smo.kenken.go.jp/, last accessed 15 July 2011).

Event	Time	Name of event and epicenter coordinates	M (JMA)	Dist (km)	Largest peak acc. (gals) 1st Fl./52nd Fl.
1	201103111446	Off Sanriku (mainshock) 38°06′11″N, 142°51′36″E	9.0	769	34.3/130
2	201103111515	Off Ibaraki prefecture 36°06′29″N, 141°15′53″E	7.7	555	9.2/120
3	201103120359	N Nagano prefecture 36°59′06″N, 138°35′48″E	6.7	387	1/7
4	201103152231	E Shizuoka prefecture 35°18′29″N, 138°42′47″E	6.4	309	1/6
5	201104072332	Off Miyagi prefecture 38°12′11″N, 141°55′11″E	7.1	704	2/8
6	201104111716	Hama-dori, Fukushima prefecture 36°56′42″N, 140°40′18″E	7.0	539	1.5/8
7	201107051918	N Wakayama prefecture 33°59′24″N, 135°13′59″E	5.5	74	5/7
8	201107100957	Off Sanriku 38°01′54″N, 143°30′24″E	7.3	816	1.5/13

Long-period responses of structural systems at large distances have been observed for many earthquakes, and in particular for tall buildings. One of the earliest observations in the USA was during the M 7.3 Kern County earthquake of 7 July 1952, which shook many taller buildings in Los Angeles and vicinity, about 100–150 km away from the epicenter (http://earthquake.usgs.gov/earthquakes/states/events/1952_07_21.php, last accessed July 15, 2011; Hodgson, 1964). The 28 March 1970 M 7.1 Gediz earthquake in inland western Turkey damaged several buildings at a car-manufacturing factory in Bursa, 135 km northwest from the epicenter (Tezcan and Ipek, 1973). One of the most dramatic examples of long-distance effects of earthquakes is from the 19 September 1985, Michoacan, Mexico, M 8.0 earthquake during which, at approximately 400 km from the coastal epicenter, Mexico City suffered more destruction and fatalities than the epicentral area due to amplification and resonance (mostly around 2 s) of the lakebed areas of Mexico City (Anderson *et al*., 1986, Çelebi *et al*., 1987). To the best knowledge of the authors, however, there are no publicly available records of the responses of tall structures from these past earthquakes. However, records obtained from numerous instrumented tall buildings during the Great East Japan earthquake of 11 March 2011 offer a rare opportunity to study and understand how structures characterized by predominantly long-period responses behave during medium to large events originating at long distances. Such effects have consequences not only for large metropolitan areas in Japan but also in other parts of the world, including the USA (e.g. Los Angeles area from Southern California earthquakes, Chicago from NMSZ and the Seattle (WA) area from large Cascadia subduction zone earthquakes). For example, the recent M 5.8 Virginia earthquake of 23 August 2011 was felt in 21 states of Eastern and Central USA, which include large cities such as New York and Chicago (http://earthquake.usgs.gov/earthquakes/eqinthenews/2011/se082311a/#summary, 15 July 2011).

In this paper, the purpose is to present and study the unprecedented records from one particular tall building (hereinafter referred to as 'the building') located 769 km from the epicenter of the mainshock, as well as those from several aftershocks of the 11 March 2011 Great East Japan earthquake. The building response records have long durations and reflect the consistent long-duration strong-shaking characteristics of the hundreds of surface and downhole (mainshock and aftershock) free-field records publicly released by KNET and KIKNET.[2] In addition, the building records also exhibit many distinct structural and site-related characteristics that certainly contributed to prolonged shaking that was most likely unbearable to occupants of the building. Records from KIKNET station OSKH02, the free-field station closest to the building, will be used in this paper to infer and confirm site characteristics

[2]KNET and KIKNET are free-field networks (www.k-net.bosai.go.jp and www.kik.bosai.go.jp/)

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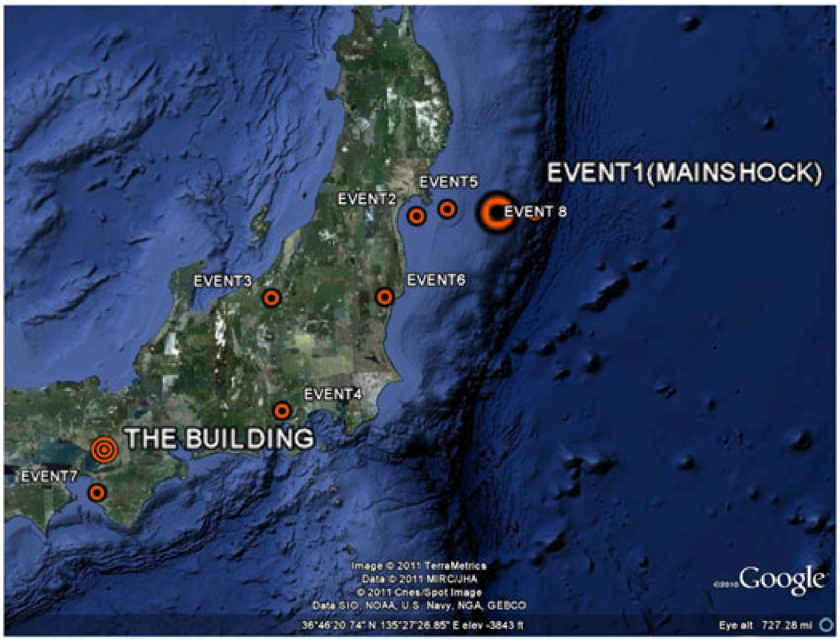

Figure 1. Google Earth map showing the relative locations of the building, the epicenters of the mainshock (large circle) and other events from which data are referred to in this paper.

computed from geotechnical logs. Tectonics, seismology and structural damage reconnaissance observations related to the Great East Japan earthquake mainshock are outside the scope of this paper.

1.1. Recorded building responses and analyses

The earthquakes studied in this paper and the peak motions of the records from these events are summarized in Table 1. A map showing the locations of the epicenters with respect to the building is shown in Figure 1. All of the events occurred at shallow depth (<40 km). The large epicentral distances are again noted.

1.2. Duration of strong shaking

An accepted indicator of strong-shaking duration is the interval between the 5% and 95% levels of the cumulative sum of squared acceleration values (Trifunac and Brady, 1975). From the cumulative sums of acceleration for the downhole components of the KIKNET station OSKH02 (Figure 2), the duration of strong shaking for the mainshock in the vicinity of the building is determined to be about 140 s.

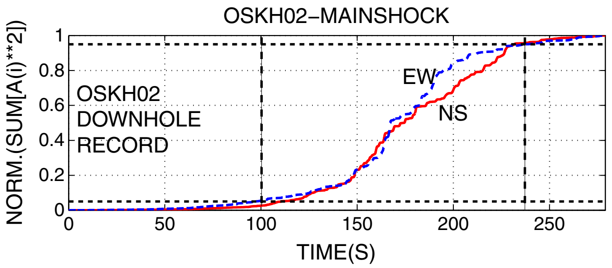

Figure 2. Normalized cumulative sum of squares of acceleration time-histories at the borehole of KIKNET station OSKH02 station indicates strong-shaking duration as 130–140 s.

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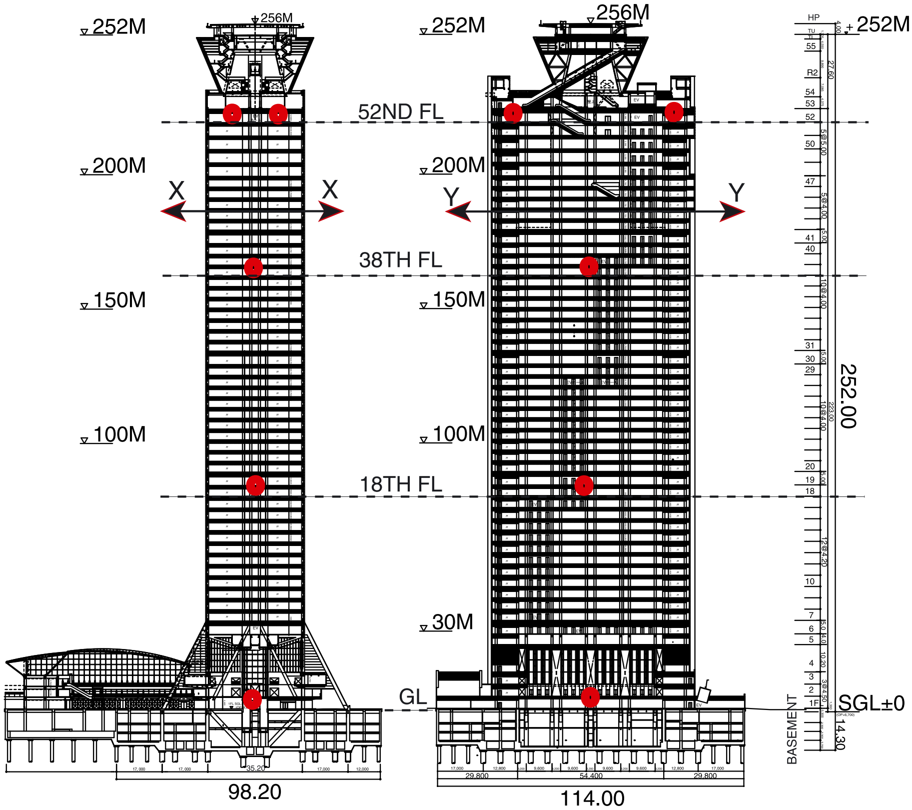

Figure 3. Vertical sections of the building showing major dimensions and locations of tri-axial accelerometers on the 52nd, 38th, 18th and ground level (first floor). *X* and *Y* denote principal axes of the building (Figure 5).

1.3. The building, its foundation and instruments

The 256-m tall building (55 stories plus 3-story basement) is located on a reclaimed island near Osaka, Japan. The vertically irregular building has a steel-moment frame and a rigid truss beam every 10 stories. There are no shear walls around the several elevator shafts that would add to the lateral stiffness of the building. The building is founded on piles that are approximately 60–70 m long and that are essentially end-bearing at approximately 63 m below ground surface and rest on a diluvial gravel layer. The pile designs include friction in the upper alluvial clay layers of the subsurface. The construction of the building was completed in 1995 and therefore was designed according to pre-1995 codes (before the M 6.9 Kobe earthquake of 16 January 1995).

Vertical sections of the building with general dimensions and locations of tri-axial accelerometers[3] are shown in Figure 3. Principal axes of the building are identified as *X* (229° clockwise from N) and *Y* (319° clockwise from N; Figure 5). A plan view of the building at the 52nd floor is shown in Figure 4, and a schematic showing approximate locations and orientations of the accelerometers on the 52nd, 38th, 18th and ground level (also the first floor) are displayed in Figure 5. During the design/analyses process, the fundamental frequencies for the two principal horizontal directions (*X* and *Y*) and torsion were computed to be 0.17 Hz, 0.18 Hz and 0.27 Hz, respectively.

1.4. The building site

The prolonged responses of the building to shaking inputs suggest the possibility of resonance due to soil–structure interaction (SSI). SSI was not considered during the design/analysis phase of the building.

[3]The instruments were installed by Building Research Institute, (http://smo.kenken.go.jp/).

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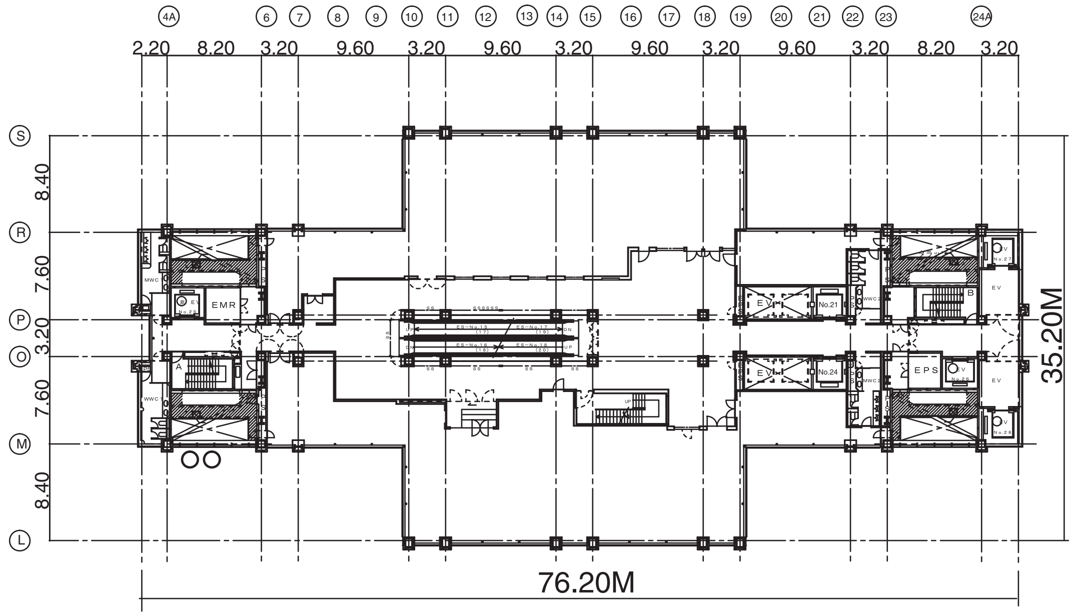

Figure 4. Typical plan view (the figure shows the 52nd floor).

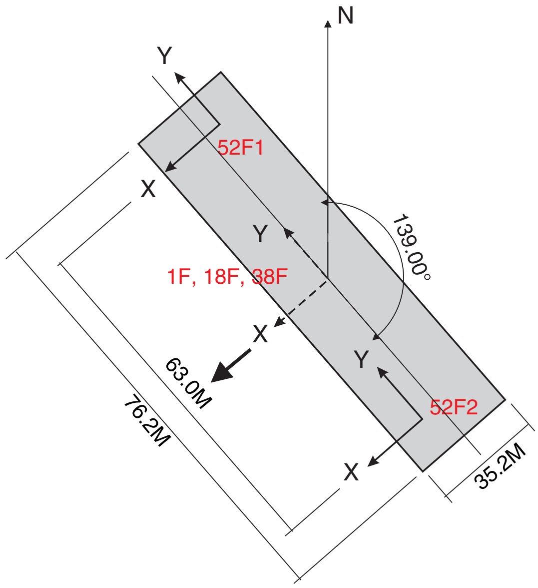

Figure 5. Principal axes of the building in plan view showing general locations of the sensors on the 52nd, 38th, 18th and ground levels. Note that at the 52nd floor, there are two sensor locations: 1 denoted as the north location and 2 as the south location.

To assess whether SSI may have played an important role in the response requires knowing the fundamental site response characteristics. Detailed studies of the Osaka basin are presented in Yamada and Horike (2007), Sekiguchi *et al*. (2007) and Iwaki and Iwata (2008), but the results of these studies likely do not accurately characterize the local site transfer function. Instead, we computed the site transfer functions using software developed by C. Mueller (personal communication, 1997), which is based on Haskell's shear wave propagation method (Haskell, 1953, 1960). In this method, the transfer function is computed using linear propagation of vertically incident horizontally polarized shear waves and has, as input, data related to the layered media (number of layers, depth of each layer, corresponding V_s, damping and density), desired depth of computation of transfer function, computation frequency (df), half space substratum shear wave velocity and density. Damping (ξ) in the software is introduced via the quality factor (Q), a term used by geophysicists that is related to damping by $\xi = 1/(2Q)$.

The parameters used in computing the site transfer functions are Profiles A, B and C as shown in Figure 6. Profile A is an approximation based on the geotechnical data for free-field KIKNET station OSKH02 that is near (~2.5 km) the building. In this profile, the upper and softer layers have been ignored. By way of comparison with the transfer functions computed for Profiles B and C, which underlie the building, it is concluded that the upper layers do not significantly alter the computed fundamental frequency of the site of this building. Q values used in calculating the transfer functions range between 25 and 60 for shear wave velocities between 200 and 600 m/s—having been approximately interpolated to vary linearly within these bounds.

As seen in Figure 6, the site fundamental frequency of the site is computed to be in the range of 0.13–0.17 Hz due to the dominant characteristics of layers 3 and 4 (typically of the area of the site of the building and KIKNET OSKH02 strong-motion station as described in Figure 6).

2. ANALYSES OF MAINSHOCK RESPONSE RECORDS

2.1. Record from nearby KIKNET station OSKH02

The computed site frequency is corroborated by spectral ratio of surface and downhole records obtained at the KIKNET station OSKH02. Figure 7 shows the mainshock acceleration time histories at the surface and downhole (elevations at 6.68 m and −2001 m, respectively) for this station, the corresponding amplitude spectra, and the spectral ratios of the amplitude spectra at the surface with respect to the downhole. The spectral ratios indicate that the first mode of the site is in the range 0.13–0.17 Hz (5.9–7.7 s). This is an important characteristic site parameter that will be referred to later in the paper.

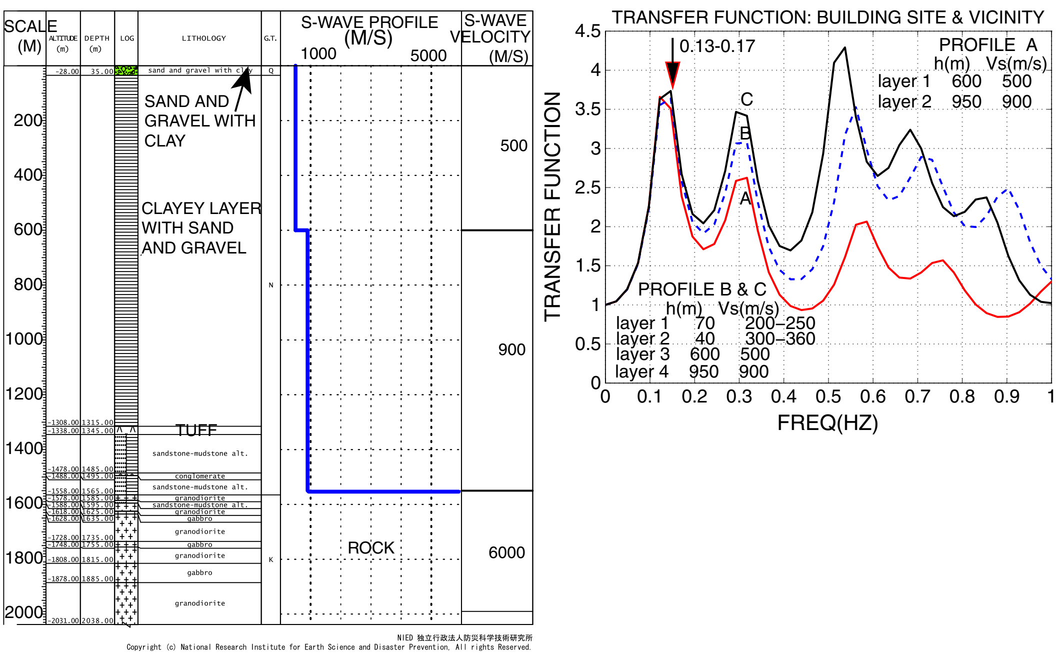

Figure 6. (Left) Depth versus V_s profile of OSKH02 KIKNET site (modified from NIED, 2011: www.kik.bosai.go.jp/, last accessed 16 September 2011). (Right) Transfer functions computed for Profile A (near the OSKH02 strong-motion site) and Profiles B and C below the building. The depth of the softer upper two layers below the building does not significantly change the position of the peaks in the transfer function, particularly for the fundamental mode of the site.

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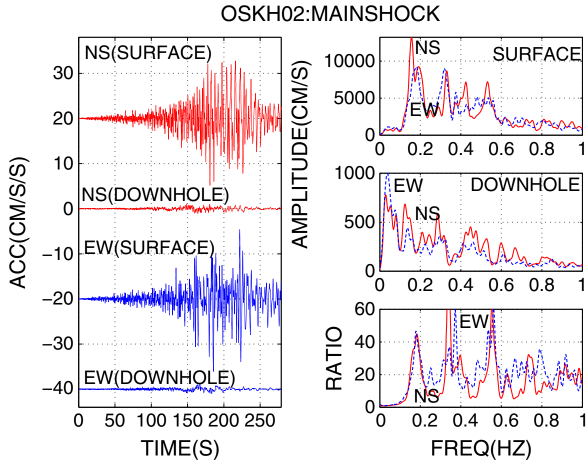

Figure 7. Mainshock acceleration time histories at the surface and downhole of KIKNET station (lower panel). Note the relative amplitudes of motions and spectra at the surface with respect to the OSKH02 (top two panels), corresponding amplitude spectra (next two panels), and spectral ratios downhole, as reflected in the scaling.

2.2. *Mainshock records from the building*

The most significant response records of the building are those from the mainshock at an epicentral distance of 769 km. Figure 8 shows a plot displaying the unprecented 1000-s-long records of responses from different levels of the building in the *X*-direction and *Y*-direction. To the best knowledge of the authors, such long-duration response records have not previously been obtained, even though there likely have been many buildings that experienced such shaking. The long durations of repetitious cycles in the responses suggest that the building is in resonance and also that damping is quite low. Beating, particularly in the *Y*-direction, is also clearly observed.

Figure 9 compares accelerations and displacements at the 52nd floor. It is noted that, except for a scaling factor, the envelopes of the accelerations and displacements are quite similar. Approximately 100 gals of acceleration has translated into approximately 100 cm of displacements. For a 52-story building, as for most tall buildings, these levels of motion are not expected to cause problems. Average drift ratios are presented later in the paper to support this assertion.

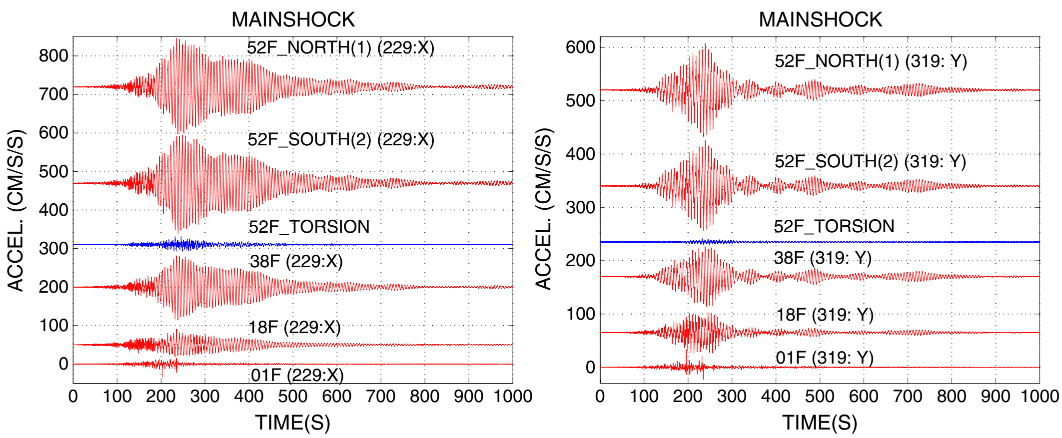

Figure 8. Recorded mainshock acceleration responses at the ground level (01* floor), 18th, 38th and 52nd floors of the building. Torsional time-history is the differential parallel horizontal records at locations 1 (north) and 2 (south) of the 52nd floor (*as used in database but is interchangeably referred as the first floor also. It is also the ground level).

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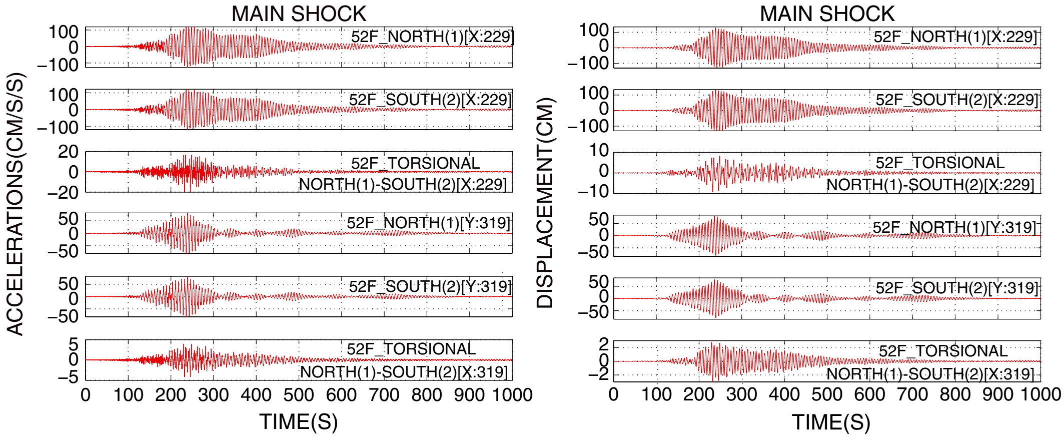

Figure 9. Comparison of accelerations and displacements at the 52nd floor. Torsional effects are plotted from difference of two parallel channels at locations 1 and 2 on that floor.

Figure 10 shows amplitude spectra of mainshock accelerations at the 52nd floor. Figure 11 shows spectral ratios of amplitude spectra of accelerations at the 52nd, 38th and 18th floors with respect to those from the first floor. In all cases, clear, narrow-band peaks in frequency are indicative of low-damping percentages. Identified frequencies (periods) are 0.152 Hz, 0.489 Hz and 0.905 Hz (6.58 s, 2.06 s and 1.11 s) for the first three modes in the *X*-direction; 0.145 Hz, 0.426 Hz and 0.725 Hz (6.90 s, 2.34 s and 1.38 s) for the first three modes in the *Y*-direction; and 0.214 Hz and 0.580 Hz (4.69 s and 1.72 s) for the first and second torsional modes. Torsional frequencies are identified from amplitude spectra of differences between two parallel accelerations in the *X*-direction and *Y*-direction, respectively. It is noted that the torsional and translational frequencies are not close to each other. The lack of additional sensors on floors other than the 52nd precludes comparing the torsional frequency with those from other floors.

Figure 12 shows system identification (SID) analysis results applied to the mainshock records. In SID analysis, a model is estimated using appropriate pairs of recorded acceleration responses as single-input, single-output (SISO). The auto-regressive extra input (ARX) model based on least squares method is used in this analysis. The reader is referred to Ljung (1987) and MATLAB Users Guide (1988 and newer versions) for detailed formulations of the ARX and other SID methods. Some of the key frequencies for two modes in the *X*-direction and three modes in the *Y*-direction, as well as associated modal damping percentages (ξ), are identified by the SISO SID method. First-floor accelerations are used as input and 52nd-floor accelerations as output. The recorded and computed accelerations at the 52nd floor and their corresponding amplitude spectra match well. The damping percentages extracted from SID analyses are quite low (1.2–1.6% for the fundamental modes) and is

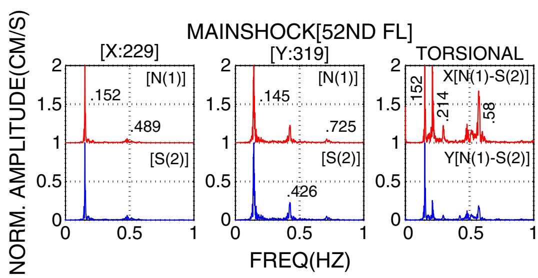

Figure 10. Amplitude spectra of accelerations at the 52nd floor displays the frequencies in the principal axes of the building. Torsional frequencies are also identified from the difference (N–S) between two parallel accelerations at the north (1) and south(2) end locations and in the *X*-direction and *Y*-direction, respectively. North (1) and south (2) denotes locations at the 52nd floor (Figure 5). The third mode (~0.9 Hz) in *X*-direction is not identified from amplitude spectrum.

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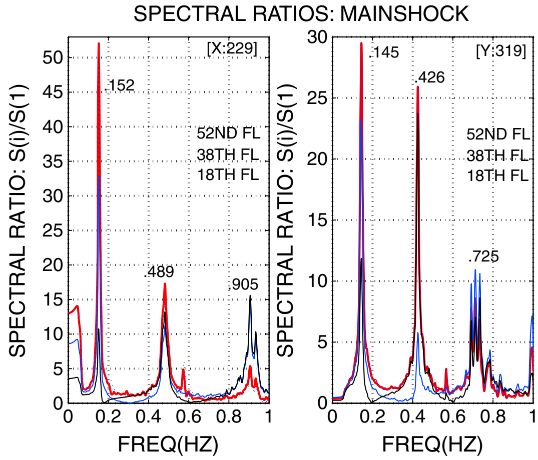

Figure 11. Spectral ratios of amplitude spectra at the 52nd, 38th and 18th floors with respect to that at the first floor. Note that the third mode in the X-direction is identified from the ratios. Different colors (red, black and blue) are used only to distinguish lines corresponding to different floors in descending order.

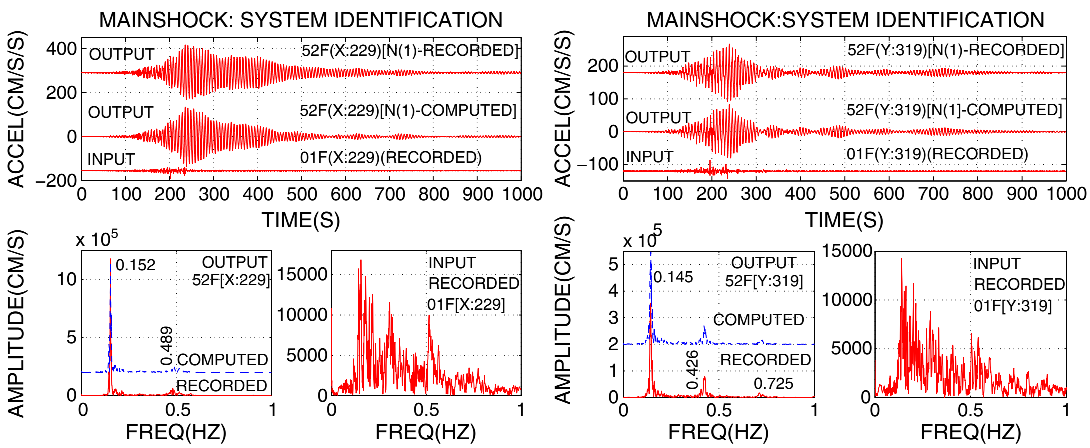

Figure 12. System identification applied to mainshock records. First-floor accelerations are used as input and 52nd-floor accelerations as output. The computed 52nd-floor accelerations match well those that were recorded.

herewith asserted to be one of the main causes for the prolonged shaking (including beating phenomenon) of the building. The results from analyses of response records from the mainshock and two selected aftershocks using both spectral analyses and SID methods, as well as those determined during design/analyses process, are all summarized in Table 2 and discussed later in the paper.

An important observation is that the fundamental frequencies (periods) in the X-direction and Y-direction of the building are similar to the site frequency (period) of 0.13–0.17 Hz (5.88–7.69 s), which was discussed earlier. From this, we conclude that there was SSI between the building and the subsurface below the foundation. Resonance caused by SSI and enhanced by low structural critical damping percentages (ξ) resulted in the prolonged responses. These motions involve more than 100 cycles with peak input at the first floor of the order of 34 gals (~3% g) or less. This will be further discussed later in the paper.

The results also bring forth an interesting observation. Taking 0.152 and 0.145 as the fundamental frequencies in the X-direction and Y-direction, respectively, the square of the circular frequency is $w^2 = (2\pi f)^2 = 0.83–0.91$ (~1); hence, the displacement amplitudes are similar to those of accelerations (but with different units). The similarity in amplitudes was also observed earlier in Figure 9.

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Table 2. Summary of frequencies (periods) determined by spectral analyses and system identification techniques applied to the mainshock and two aftershocks (event 2 that occurred 30 min following the mainshock and Event 8 that occurred on July 10, 2011). Critical damping percentages are identified by system identification only.

Orientation	*X* (229)			*Y* (319)			Torsion	
Modes	1	2	3	1	2	3	1	2
Analyses during design								
Frequency (Hz)	0.1887			0.1724			0.2703	
(T (s))	(5.3)			(5.8)			(3.7)	
Spectral analyses								
Mainshock (event 1)								
Frequency (Hz)	0.152	0.489	0.905	0.145	0.426	0.725	0.213	0.58
(T (s))	(6.58)	(2.06)	(1.11)	(6.90)	(2.34)	(1.38)	(4.69)	(1.72)
System identification								
Mainshock (event 1)								
Frequency (Hz)	0.1524	0.4887	N/A	0.1447	0.4264	0.7250		
(T (s))	(6.56)	(2.05)		(6.91)	(2.35)	(1.38)		
Damping (ξ)	0.012	0.020		0.016	0.001	0.020		
Aftershock (event 2)								
Frequency (Hz)	0.1552	0.4791	N/A	0.1430	0.4241	0.7154		
(T (s))	(6.44)	(2.08)		(6.99)	(2.36)	(1.40)		
Damping (ξ)	0.010	0.016		0.016	0.004	0.008		
Aftershock (event 8)								
Frequency (Hz)	0.1535	0.4864	N/A	0.1497	0.4427	0.7787		
(T (s))	(6.51)	(2.06)		(6.68)	(2.26)	(1.28)		
Damping (ξ)	0.011	0.013		0.033	0.024	0.041		

2.3. Drift ratios

Since sensors are not installed at any two consecutive floors of the building, only average drift ratios (D) can be computed from displacements between any two floors where accelerations are recorded. Figure 13 shows drift ratios between the 52nd and 1st floors, the 52nd and 38th floors, and the 38th and 1st floors for the X-direction and Y-direction. Maximum drift ratios in the X-direction are about 0.5%, and those in the Y-direction are about 0.2%. For the ~3% g input motion at the first-floor level, these are large drift ratios inferred to be due solely to the resonating amplified response of the building. A 1% drift ratio is the maximum limit for the design of buildings taller than 60 m in Japan for the collapse protection (level 2) motions for which zero period acceleration is much larger than 3% g (The Building Center of Japan, 2001a, 2001b). In the USA, the comparative maximum drift ratio for tall buildings for Risk Category 1 or 2 is 2% (Table 12.12, ASCE7-10, 2007).

2.4. Beating effects

In general, beating behavior is characterized by coupling between translational and torsional motions in the presence of low damping (Boroschek and Mahin, 1991; Çelebi, 1994). Cyclically stored potential

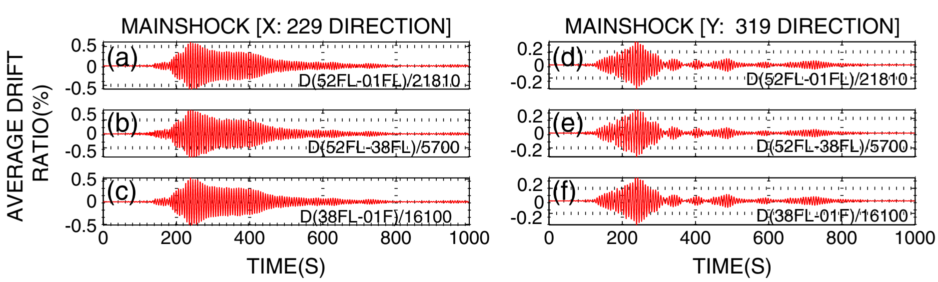

Figure 13. Average drift ratios computed from displacements between the 52nd, 38th and 1st floors. In each frame, the numbers in denominators are distances (in cm) between the designated floors.

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energy during the coupled translational and torsional deformations turns into repetitive vibrational energy. Thus, periodic repeating and resonating motions ensue as depicted in Figures 8, 9, 12 and 13. Beating becomes severe if the system is lightly damped. However, in this building the translational and torsional frequencies are neither close nor coupled. The classical formula for computing a beating period is $T_b = 2\, T_1 T_2/(\mathrm{T}_1 - \mathrm{T}_2)$ (alternatively, beating frequency, $f_b = (f_2 - f_1)/2)$, where T_b is the beating period, and T_1 and T_2 are translational and torsional frequencies, respectively. Application of this formula to the present case yields a beating period that is shorter than the beating periods observed in the response records depicted in Figures 8, 9, 12 and 13. For example, taking for T_1 the previously identified translational fundamental period(s) of the building as 6.58 s or 6.9 s in the *X*-direction and *Y*-direction, respectively, and torsional period as 4.69 s, then $T_b = 2T_1T_2/(T_1 - T2) = 2 \times 6.58 \times 4.69/(6.58 - 4.69) = 32.66$ s or $2 \times 6.9 \times 4.69/(6.9 - 4.69) = 29.29$ s. The observed beating periods range between 50 s and 150 s. This mismatch leads to the conclusion that low damping played a predominant role in generating the observed long-duration beating vibrations and inadequate dissipation of vibrational energy.

2.5. *Moving window analyses*

The results of subjecting an orthogonal pair of 52nd-floor acceleration records to moving window amplitude spectral analyses are displayed in Figure 14 for the whole 1000-s record and also for the

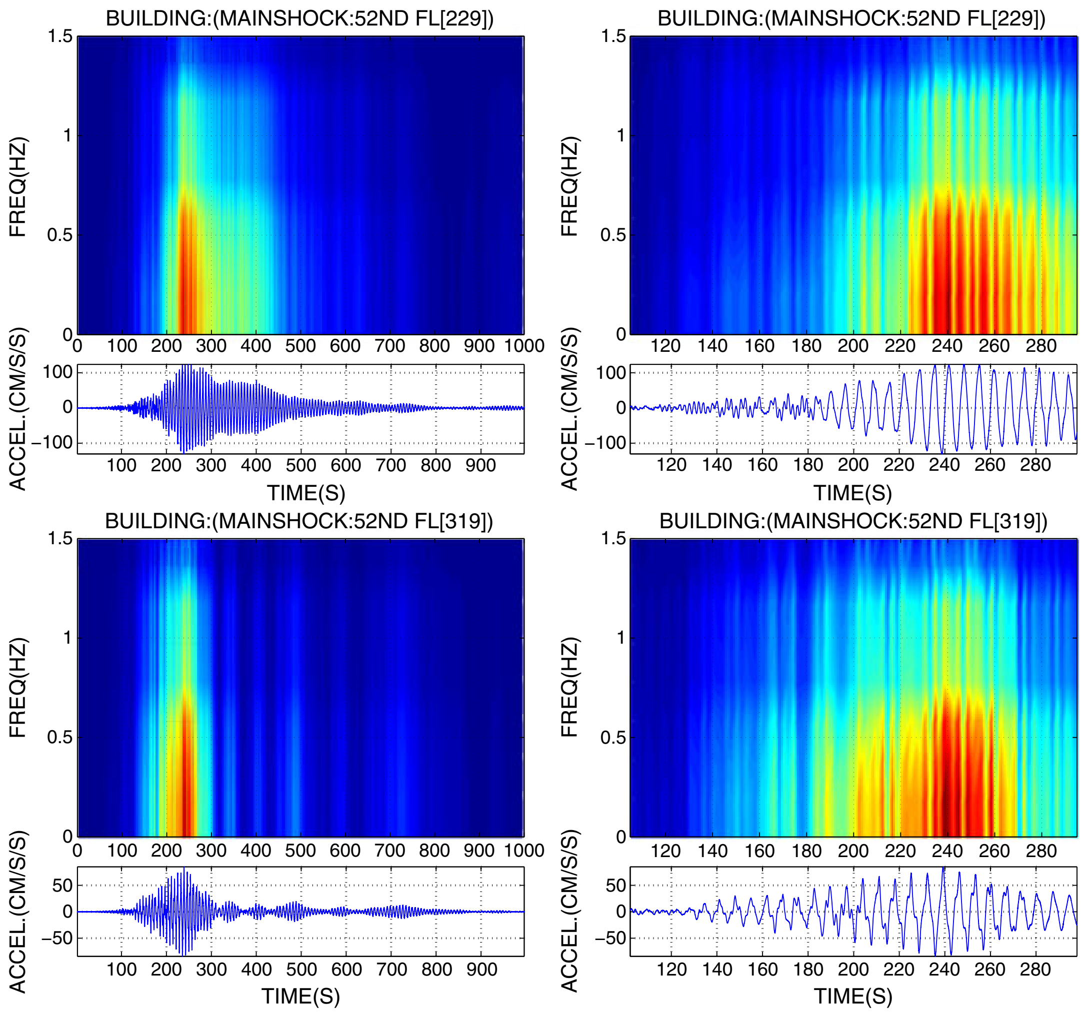

Figure 14. Moving window amplitude spectra of one pair of orthogonal accelerations from the 52nd floor. The spectra, while indicating highest energy between ~230 s and 260 s, does not indicate any significant shift in the frequencies.

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time interval 100–300 s of the record. The spectra for the shorter time history window clearly indicate that the largest shaking energy occurred approximately 230–260 s into the record. The spectra do not reveal any obvious significant shift in the predominant frequencies throughout the strong shaking—an indicator that the building likely was not damaged.

2.6. Aftershocks and independent post-mainshock events

Like the mainshock records, the recorded responses of the building have long durations for the other seven events considered (Table 1). Furthermore, all of the post-mainshock events share very similar or identical characteristics—frequency peaks, damping, long durations of strong shaking and beating periods that are considerably longer than those computed for the mainshock. As an example of these repeated characteristics of recorded responses, accelerations and displacements at the 52nd floor are presented for event 8 in Figure 15. In Figure 16, normalized amplitude spectra for all eight events, including the mainshock, are provided to demonstrate that there is no significant variation in the identified frequencies and spectral shapes. The consistency through time in the translational and torsional fundamental frequencies is an indicator that there was no damage to the building. However, even though the building appears not to have been damaged, such prolonged and repetitious shaking may affect the future health of the building by contributing to low-cycle fatigue of the structural steel members and joints of the building. The implication is that the response characteristics of the building must be changed by modifying its fundamental frequencies so that they are substantially different than the site frequency and also by increasing the damping capability of the overall structural system to readily dissipate the vibrational energy.

3. SUMMARY AND CONCLUSIONS

Significant modal frequencies and relevant critical damping percentages (ξ) were determined from unique and unprecedently very long duration (about 1000 s) response records of the subject building using spectral analyses and SID techniques. The computed values are summarized in Table 2.

The building-specific information and results presented in this paper prompt the following comments and conclusions.

1. Structural and site frequencies:

    a. As summarized in Table 2, the significant fundamental translational and torsional frequencies (periods) identified are clearly shorter (longer) than those determined and used during the design of the building.

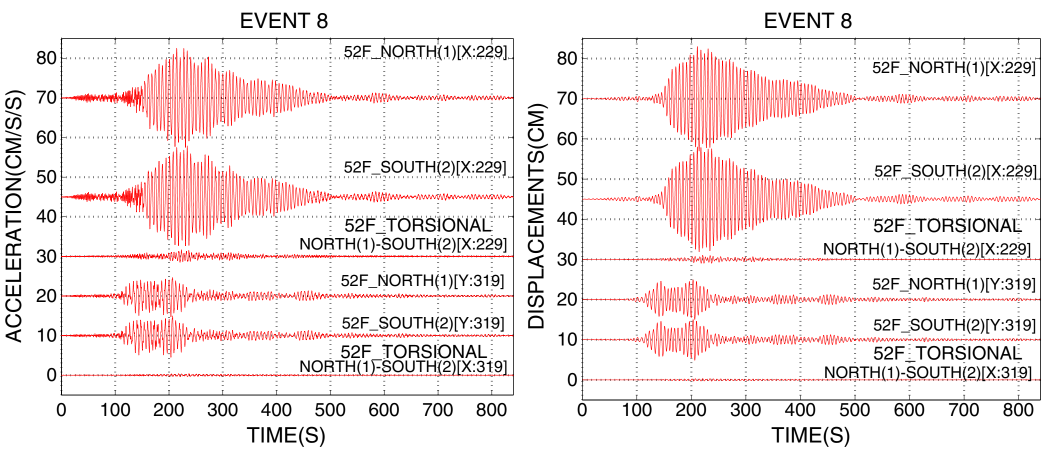

Figure 15. Comparison of accelerations and displacements at the 52nd floor for event 8. As in Figure 9, torsional effects are plotted from difference of two parallel channels on that floor.

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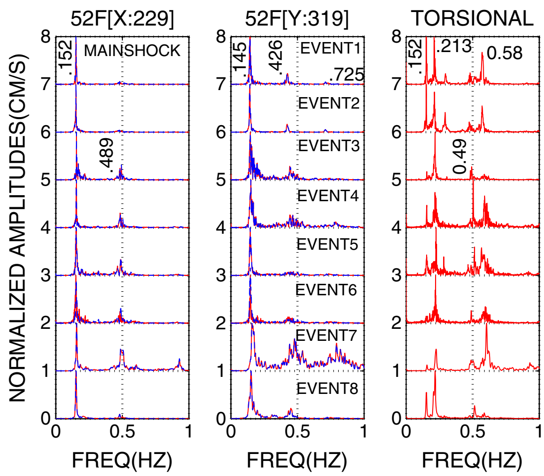

Figure 16. Normalized amplitude spectra of accelerations recorded at the 52nd floor of the building depicts consistent translational and torsional frequencies for the mainshock plus seven events that followed.

b. The fundamental site frequency (period) determined using records from a nearby KIKNET free-field station (OSKH02) and by computation of transfer functions using geotechnical logs is in the range of 0.13–0.17 Hz (5.9–7.7 s) indicate that resonance due to closeness of the site frequency to that of the building contributed to the prolonged long-duration shaking of the building. (Note that because KIKNET stations were deployed after the 1995 Kobe earthquake, such important site-related data was not available at the time of design/construction of the building).
c. SSI was not considered during design/analyses process. Therefore, the possibility that resonance might occur was not taken into account when computed structural characteristics could not be compared with (empirical) site frequency due to lack of site-related data.

2. Damping: An important characteristic of the structural system is the low critical damping percentage ($1\% < \xi < 2\%$), as determined by SID methods using the mainshock and two aftershock records. This property likely contributed to the prolonged, repetitious and resonating cyclic behavior of the building. The possibility of such low-damping percentages of the structural system was most likely not considered or estimated during the design/analyses process.
3. Drift ratios: Average drift ratios computed from relative displacements between many floors indicate that maximum average drift ratios experienced during the mainshock was between 0.5–1.0% for the X-direction and 0.2–0.4% for the Y-direction. These average drift ratios are less than the maximum 1% limit usually used in Japan for collapse protection level motions (level 2 used for buildings 60 m or taller (The Building Center of Japan, 2001a, 2001b)). However, average drift ratios are much larger than expected for an input motion with small peak acceleration in the order of only 3% g. In the USA, the comparative maximum drift ratio for tall buildings for Risk Category 1 or 2 is 2% (Table 12.12, ASCE/SEI Standard 7–10, 2007).

The following suggestions are made:

1. The risk from closer large-magnitude earthquakes that could subject the building to larger peak input motions should be assessed in light of the substantial drift ratios under the low peak input motions experienced during and following the Great East Japan earthquake of 2011.
2. Prolonged cyclic behavior of the building, even at moderate to small amplitudes with relatively acceptable elastic drift ratios, can be a cause for low-cycle fatigue. This issue is significant, and unless structural dynamics characteristic of the building are modified, low-cycle fatigue can be an important concern for the health of the building in the future.
3. Immediate remediation to improve the behavior of the building by applying response modification technologies (e.g. adding dampers at select bays and floors) in order to dissipate the vibrational energy and thus decrease the prolonged shaking and suppress the beating effect of the building.

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4. To the knowledge of the authors, no low-amplitude ambient tests of the building have been conducted. Low-amplitude ambient shaking of this and similar buildings can be very useful in identifying relevant structural characteristics for future design/analyses purposes. Such information can provide clues related to structural characteristics which in turn can be used to infer possible detrimental behavior and performances of structures during strong shaking.
5. The instrumentation in the building should be denser. Deployment of additional sensors on different floors would facilitate better correlation and identification analyses, specifically the following:
   a. Instead of using tri-axial sensors at upper levels, using uni-axial and bi-axial sensors can save the vertical sensors (at the 52nd, 38th and 18th floors) to be re-deployed as additional horizontal sensors at the 38th, 18th and 1st levels and as additional vertical sensors in the basement or first floor.
   b. Adding horizontal channels to the 38th and 18th floors will facilitate correlation of torsional behavior of the building.
   c. Adding vertical sensors in the basement will facilitate recording rocking, if it occurs during the vibration of the building.
   d. Adding additional sensors to other floors or neighboring floors will facilitate better evaluation of local drift ratios instead of average drift ratios between many floors.

Finally, the behavior and performance of this particular tall building far away from the strong-shaking source of the Great East Japan event of 2011 and large-magnitude aftershocks should serve as a reminder that, in the USA as well as in many other countries, risk to such built environments from distant sources must always be considered.

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AUTHORS' BIOGRAPHIES

Mehmet Çelebi received his BS degree in civil engineering from Middle East Technical University, Ankara, Turkey; MSc from Stanford University, Stanford, California; and PhD from McGill University, Montreal Canada. He is currently employed as a senior research engineer at United States Geological Survey in Menlo Park, California. His research interests include monitoring of structures, analyses of response data from structures, site response and assessment of existing structures.

Izuru Okawa received his BS, MSc and PhD degrees from University of Tokyo, Tokyo, Japan. He is currently employed as a senior research fellow at Building Research Institute of Japan in Tsukuba, Japan. His current research interests include site response, structural response of buildings during earthquakes and also design earthquake motions for buildings.

Toshihide Kashima graduate from Ishikawa National College of Technology, Ishikawa, Japan, and received his PhD degree from Keio University, Japan. He is currently employed as a senior research engineer at Building Research Institute in Tsukuba, Japan. His research interests include effect of surface geology on earthquake motions, seismic response of building structures and structural health monitoring.

Shin Koyama received his BS, MSc and PhD degrees from Tokyo Institute of Technology, Tokyo, Japan. He is currently employed as a chief research engineer at Building Research Institute of Japan in Tsukuba, Japan. His current research interests include seismic wave propagation, site response and soil–structure interaction of buildings during earthquakes.

Masanori Iiba received his BS, MSc and PhD degrees from Nagoya University, Nagoya, Japan. He is currently employed as a senior research fellow at Building Research Institute of Japan in Tsukuba, Japan. His current research areas include soil dynamics, soil–structure interaction and seismic isolation for building.

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