Response of a tall building far from the epicenter of the March 11, 2011 M=9.0 Great East Japan Earthquake

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SUMMARY:
The March 11, 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 300-1000 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.

Keywords: Resonance, tall building, long distance effect, site frequency, building frequency

1. INTRODUCTION

The March 11, 2011, M 9.0 Great East Japan earthquake 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 32 km depth; http://earthquake.usgs.gov/earthquakes/eqinthenews/2011/usc0001xgp/, last accessed July 15, 2011). Through July 15, 2011, the time 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 minutes after the mainshock (Table1). The earthquake caused a major disaster in Japan and generated one of the most significant tsunamis, a tsunami that left its mark by destroying a four-unit nuclear power plant and by causing the largest percentage of the 15,776 fatalities[1] associated with this event. It also caused widespread destruction and damage of major port and other facilities on a wide portion of north-east coast of main island of Honshu (Japan). It is widely reported that material loss may reach $300B.

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. The purpose of this paper 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 March 11, 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[2] and KIKNET[2]. In addition, the building records also exhibit many distinct

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[1] From The Japanese National Police Agency (http://www.npa.go.jp/archive/keibi/biki/higaijokyo.pdf) as confirming -15,597 deaths, 5,694 injured and 4,980 people missing across eighteen prefectures, as well as over 125,000 buildings damaged or destroyed.

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

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 closest free-field station to the building, is used to infer and confirm site characteristics computed from geotechnical logs. Reconnaissance observations related to the Great East Japan earthquake main shock are outside the scope of this paper.

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 United States was during the M=7.3 Kern County earthquake of July 7, 1952, that 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 March 28, 1970, M=7.1 Gediz earthquake in inland western Turkey damaged several buildings at a car-manufacturing factory in Bursa, 135 km north west from the epicenter (Tezcan and Ipek,1973). One of the most dramatic examples of long-distance effects of earthquakes is from the September 19, 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 sec) of the lakebed areas of Mexico City (Anderson and others, 1986, Çelebi and others, 1987). To the best knowledge of the authors, 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 March 11, 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 for large metropolitan areas in Japan, but also in other parts of the world, including the United States (*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 August 23, 2011 was felt in 21 states of the Eastern and Central U.S., that include large cities such as New York and Chicago (http://earthquake.usgs.gov/earthquakes/eqinthenews/2011/se082311a/#summary, last accessed July 15, 2011).

2. RECORDED BUILDING RESPONSES AND LONG DURATION OF STRONG SHAKING

The earthquakes studied in this paper and the peaks of motions of the records are summarized in Table 1. Figure 1 shows the locations of the epicenters of events with respect to the building. All of the events occurred at shallow depth (< 40 km). The large epicentral distances are again noted.

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 about 140 s.

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 end-bearing piles that are approximately 60-70 m long 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 January1995).

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

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

(319° clockwise from N; *see* Figure 3). A plan view of the building at the 52nd floor is also shown in Figure 3. Furthermore, approximate locations and orientations of the accelerometers on the 52nd, 38th, 18th and ground level (also 1st floor) are displayed within the vertical sections of Figure 3. 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, 0.18 and 0.27 Hz respectively.

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

Event	Time	Name of Event & Epicenter Coordinates	M (JMA)	Dist (km)	Largest Peak Acc.(gals) 1st Fl./52nd Fl
1 (**)	201103111446	Off Sanriku [Mainshock] 38^{o}06’11‖N,142^{o}51’36‖E	9.0	769	34.3/130
2	201103111515	Off Ibaraki Pref 36^{o}06’29‖N,141^{o}15’53‖E	7.7	555	9.2/120
3	201103120359	N Nagano Pref 36^{o}59'06"N, 138^{o}35'48"E	6.7	387	1/7
4	201103152231	E Shizuoka Pref 35^{o}18'29"N, 138^{o}42'47"E	6.4	309	1/6
5	201104072332	Off Miyagi Pref 38^{o}12'11"N, 141^{o}55'11"E	7.1	704	2/8
6	201104111716	Hama-dori, Fukushima Pref 36^{o}56’42‖N,140^{o}40’18‖E	7.0	539	1.5/8
7	201107051918	N Wakayama Pref 33^{o}59'24"N, 135^{o}13'59"E	5.5	74	5/7
8	201107100957	Off Sanriku 38^{o}01’54‖N,143^{o}30’24‖E	7.3	816	1.5/13

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.

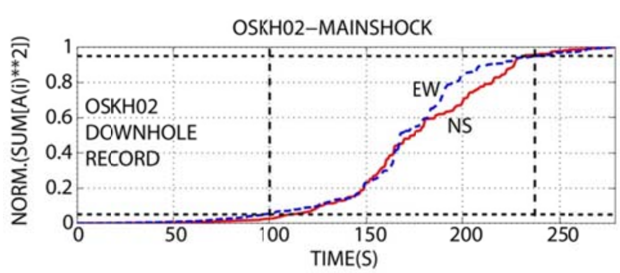

Figure 2. Normalized cumulative sum of squares of acceleration time-histories at the borehole of KIKNET station OSKH02 station indicates strong shaking duration as 130-140 seconds.

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. 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 and others (2008) 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 (*pers. comm.*, 1997), which is based on Haskell's shear wave propagation method (Haskell, 1953 and 1960). In this method, the transfer function is computed using linear propagation of vertically incident SH 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 $\xi = 1/(2Q)$.

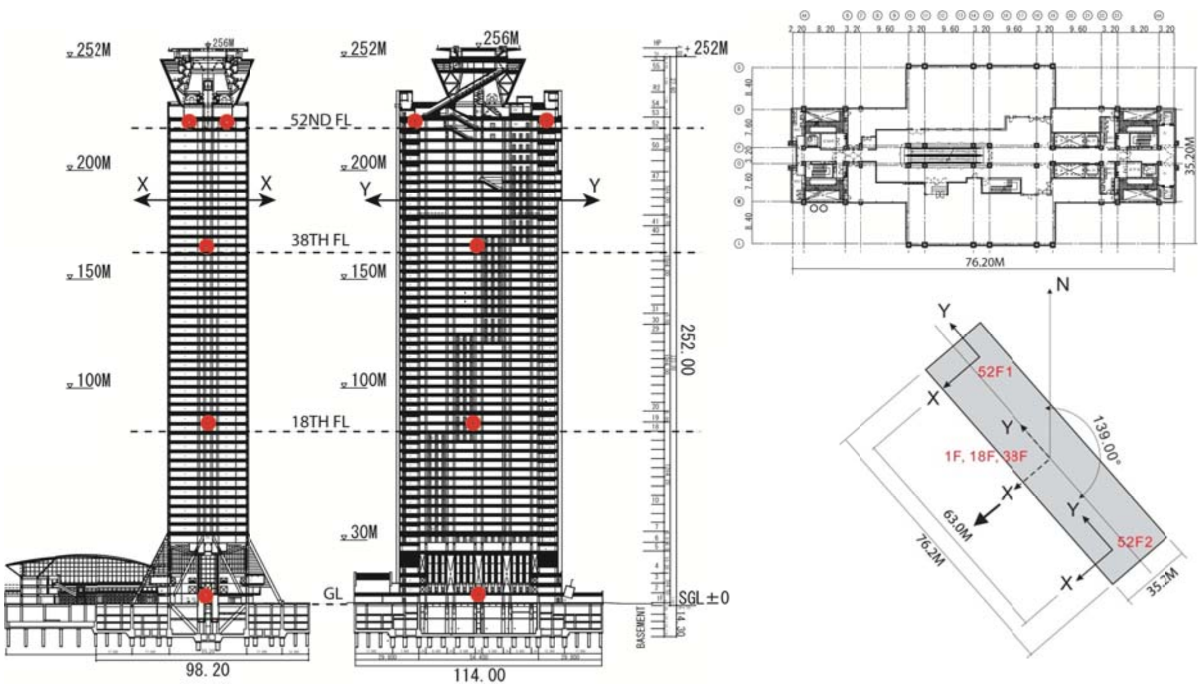

Figure 3. (Left) Vertical sections of the building showing major dimensions and locations of tri-axial accelerometers on the 52n, 38th, 18th and ground level (1st Floor). X and Y denote principal axes of the building. (Right Top) Typical Plan View (the figure shows 52nd floor). (Right Bottom) Principal axes of the building in plan view showing general locations of the sensors on 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.

The parameters used in computing the site transfer functions are the Profiles A, B, and C shown in Figure 4. Profile A is an approximation based on the geotechnical data for free-field KIK-NET station OSKH02 that is near (~2.5km) 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 25-60 for shear wave velocities between 200-600 m/s – having been approximately interpolated to vary linearly within these bounds.

As seen in Figure 4, 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 4).

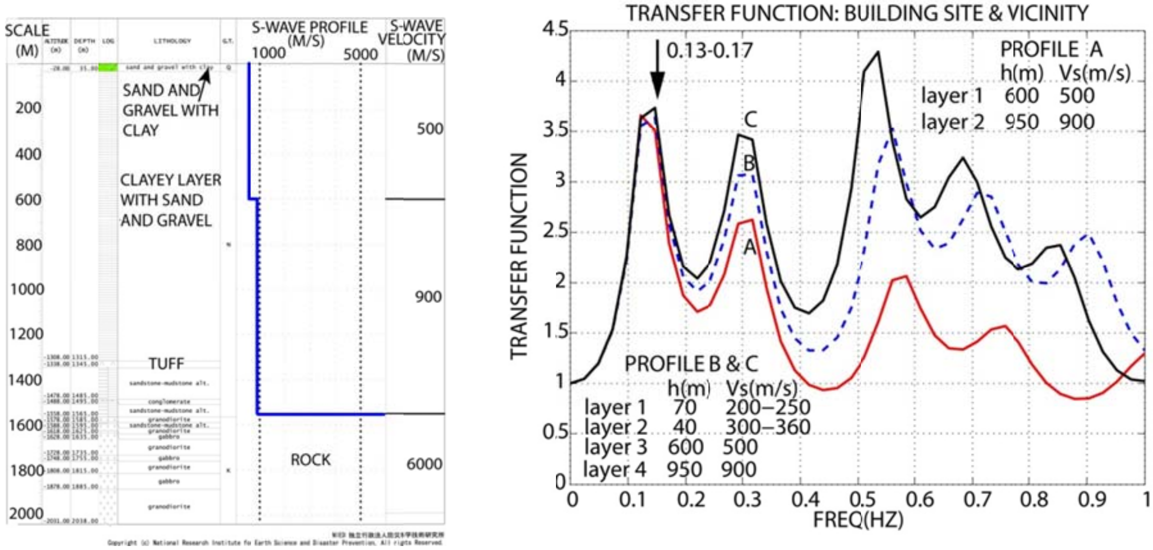

Figure 4. (Left) Depth versus V_S profile of OSKH02 KIKNET site (modified from NIED,2011:www.kik.bosai.go.jp/, last accessed 09/16/2011). (Right) Transfer functions computed for Profile A (near the OSKH02 strong-motion site) and Profiles B and C below the building. The depth of the softer upper two layers (to about 1500 m depth) below the building do not significantly change the position of the peaks in the transfer function, particularly for the fundamental mode of the site.

5. ANALYSES OF MAINSHOCK RESPONSE RECORDS

5.1. 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 5 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.

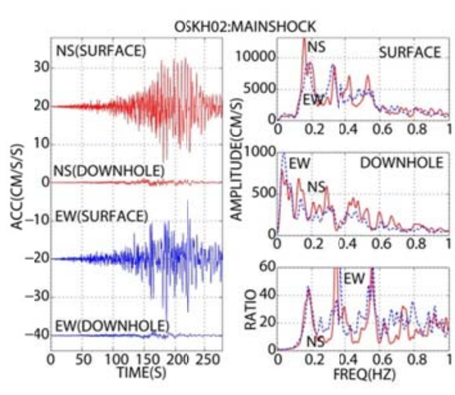

Figure 5. Mainshock acceleration time histories at the surface and downhole of KIKNET station OSKH02 (top two panels), corresponding amplitude spectra (next two panels), and spectral ratios (lower panel). Note the relative amplitudes of motions and spectra at the surface with respect to the downhole, as reflected in the scaling.

5.2. 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 6 shows a plot displaying the unprecented 1000 s-long records of responses from different levels of the building in the X- and Y-directions. 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 but discussed elsewhere (Çelebi et al, 2012).

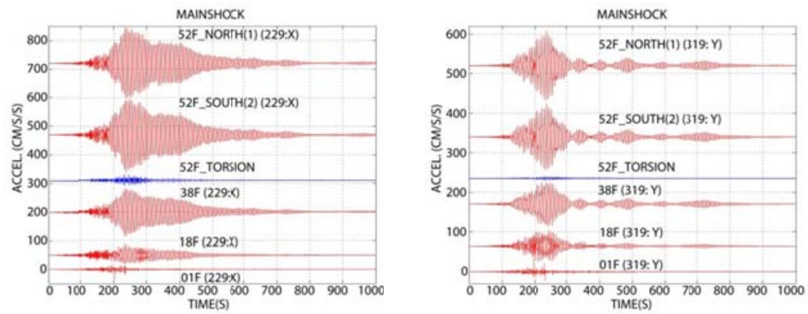

Figure 6. Recorded mainshock acceleration responses at the ground level (01* Floor), 18th, 38th and 52nd floors of the building. Torsional time-history is the differential parallel horizontal records at locations 1(north) and 2 (south) of the 52nd floor.[*as used in database but is interchangeably referred as 1st floor also. It is also Ground Level].

Figure 7 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 accelerations 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.

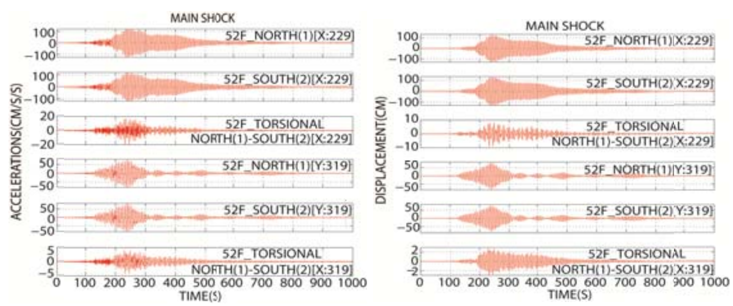

Figure 7. Comparison of accelerations and displacements at the 52nd floor. Torsional effects are plotted from difference of two parallel channels at locations 1 and 2 on that floor.

Figure 8 shows amplitude spectra of mainshock accelerations at the 52nd floor. Figure 9 shows spectral ratios of amplitude spectra of accelerations at the 52nd, 38th and 18th floors with respect to those from first floor. In all cases clear, narrow-band peaks in frequency are indicative of low damping percentages. Identified frequencies (periods) are 0.152, 0.489 and 0.905 Hz (6.58, 2.06 and 1.11 s) for the first three modes in the X-direction, 0.145, 0.426 and 0.725 Hz (6.90, 2.34 and 1.38 s) for the first three modes in the Y-direction, and 0.214 and 0.580 Hz (4.69 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- and Y-directions, 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 10 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 User's Guide (1988 and newer versions) for detailed formulations of the ARX and other system identification methods. Some of the key frequencies for two modes in X-direction and three modes in the Y-direction, as well as associated modal damping percentages (ξ), are identified by the SISO system identification method. First floor accelerations are used as input and 52nd floor accelerations as output. The recorded and computed accelerations at 52nd floor and their corresponding amplitude spectra match well. The damping percentages extracted from system identification analyses are quite low (1.2-1.6% for the fundamental modes) and is 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 system identification methods, as well as those determined during design/analyses process, are all summarized in Table 2 and discussed later in the paper.

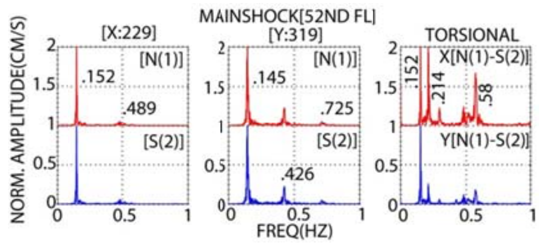

Figure 8. Amplitude spectra of accelerations at the 52nd floor displays the frequencies in the principal axes of the building. Torsional frequencies are also identified from the difference (N-S) between two parallel accelerations at the North (1) and South(2) end locations and in the X- and Y-directions respectively. North(1) and South(2) denotes locations at the 52nd floor (Figure 3). Third mode (~0.9 Hz) in X-direction is not identified from amplitude spectrum.

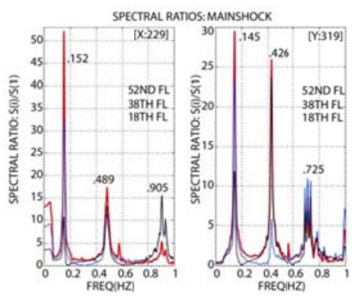

Figure 9. Spectral Ratios of amplitude spectra at 52nd floor, 38th floor and 18th floor with respect to that at first floor. Note that 3rd mode in X-direction is identified from the ratios. Different colors (red, black and blue) are used only to distinguish lines corresponding to different floors in descending order.

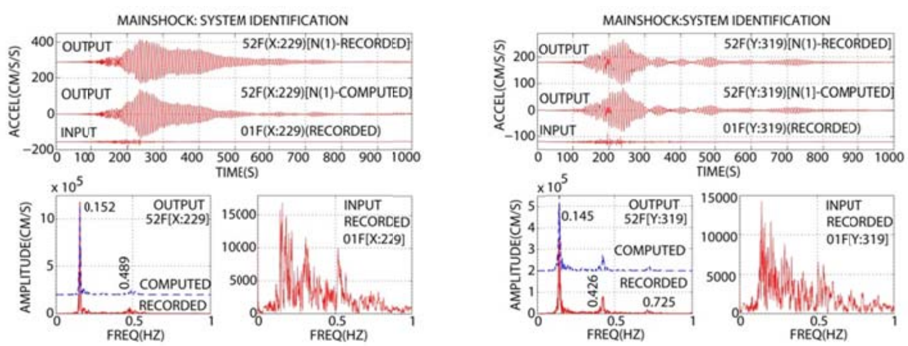

Figure 10. System identification applied to mainshock records. First floor accelerations are used as input and 52nd floor accelerations as output. The computed 52nd floor accelerations match well those that were recorded.

An important observation is that the fundamental frequencies (periods) in the X and Y directions 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 soil-structure interaction between the building and the subsurface below the foundation. Resonance caused by soil-structure interaction and enhanced by low structural critical damping percentages (ξ) resulted in the prolonged responses. These motions involve more than one hundred 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.

5.3. 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 11 shows drift ratios between the 52nd and 1st floors, the 52nd and 38th floors, and the 38th and 1st floors for the X- and Y-directions. 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 1st 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 ZPA (zero period acceleration) is much larger than 3% g (The Building Center of Japan, 2001a and 2001b). In the United States, the comparative maximum drift ratio for tall buildings for Risk Category 1 or 2 is 2% (Table 12.12, ASCE7-10, 2007).

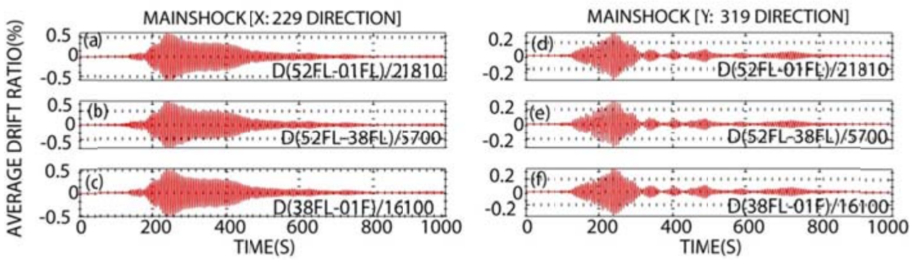

Figure 11. Average drift ratios computed from displacements between 52nd, 38th and 1st floors. In each frame, the numbers in denominators are distances (in cm) between the designated floors.

5.4. 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 12. In Figure 13, 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.

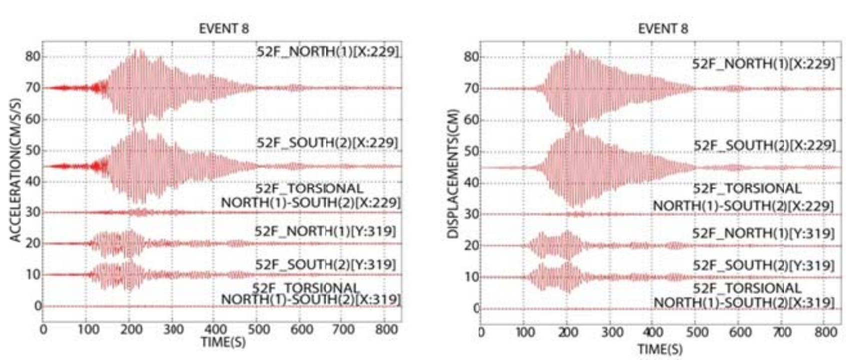

Figure 12. Comparison of accelerations and displacements at the 52nd floor for Event 8. As in Figure 7, torsional effects are plotted from difference of two parallel channels on that floor.

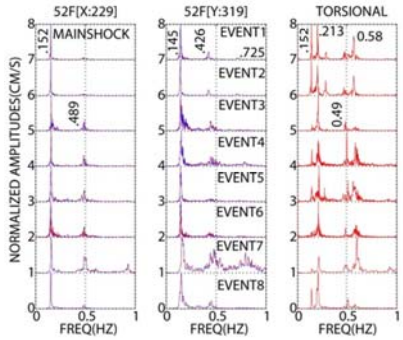

Figure 13. Normalized amplitude spectra of accelerations recorded at the 52nd floor of the building depicts consistent translational and torsional frequencies for main shock plus 7 events that followed.

Table 2. Summary of frequencies [periods] determined by spectral analyses and system identification techniques applied to mainshock and two aftershocks (Event 2 that occurred 30 minutes 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								
Freq(Hz) [T(s)]	.1887 [5.3]			.1724 [5.8]			.2703 [3.7]	
MAINSHOCK [EVENT 1] (Spectral Analyses)								
Freq(HZ) [T(s)]	0.152 [6.58]	0.489 [2.06]	0.905 [1.11]	0.145 [6.90]	0.426 [2.34]	0.725 [1.38]	.213 [4.69]	.58 [1.72]
SYSTEM IDENTIFICATION								
MAINSHOCK [EVENT 1] (System Identification)								
Freq(Hz) [T(s)]	0.1524 [6.56]	0.4887 [2.05]	N/A	0.1447 [6.91]	0.4264 [2.35]	0.7250 [1.38]		
Damping (ξ)	0.012	0.020		0.016	0.001	0.020		
AFTERSHOCK [EVENT2] (System Identification)								
Freq(Hz) [T(s)]	0.1552 [6.44]	0.4791 [2.08]	N/A	0.1430 [6.99]	0.4241 [2.36]	0.7154 [1.40]		
Damping (ξ)	0.010	0.016		0.016	0.004	0.008		
AFTERSHOCK [EVENT8] (System Identification)								
Freq(Hz) [T(s)]	0.1535 [6.51]	0.4864 [2.06]	N/A	0.1497 [6.68]	0.4427 [2.26]	0.7787 [1.28]		
Damping (ξ)	0.011	0.013		0.033	0.024	0.041		

6.0. SUMMARY AND CONCLUSIONS

The fundamental translational and torsional frequencies (periods) of the building identified are shorter (longer) than those determined during the design of the building. The fundamental site frequency (period) determined using records from a nearby free-field station and by computation of transfer functions using geotechnical logs is close to that of the building to cause resonance and prolonged long-duration shaking of the building. Also, low (1-2%) damping determined by system identification methods likely contributed to the prolonged, repetitious and resonating cyclic behavior of the building. 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. Soil-structure interaction was not considered during design/analyses process. 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. However, average drift ratios are much larger than expected for an input motion with a small peak acceleration in the order of only 3% g. Therefore, in light of the substantial drift ratios under the low peak input motions experienced during this earthquake, the risk from closer large-magnitude earthquakes that could subject the building to larger peak input motions should be assessed. Immediate remediation to improve the behavior of the building by applying response modification technologies (e.g., adding dampers at select bays and floors) should be considered.

The instrumentation in the building should be denser. Deployment of additional sensors on different floors would facilitate better correlation and identification analyses. 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 should serve as a reminder that, in the United States as well as in many other countries, risk to such built environments from distant sources must always be considered.

ACKNOWLEDGEMENTS

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