[DRAFT –USGS INTERNAL REVIEW IN PROCESS]

DRIFT ISSUES OF TALL BUILDINGS DURING THE MARCH 11, 2011 M9.0 TOHOKU EARTHQUAKE, JAPAN - IMPLICATIONS

Mehmet Çelebi
Earthquake Science Center
US Geological Survey
Menlo Park, CA 94025
(celebi@usgs.gov)

ABSTRACT

One of the most significant effects of the M9.0 Tohoku, Japan earthquake of March 11, 2011 is the now well-known long duration (>10 minutes) shaking of buildings in Japan – particularly those in Tokyo (~ 350-375 km from the epicenter) and in places as far as Osaka (~770 km from the epicenter). Although none collapsed, the strong shaking caused many tall buildings not to be functional for days and weeks.

The purpose of this paper is to discuss the behavior and performance of two tall buildings, considered to be representative of most tall buildings in Tokyo and other locations, and from which shaking data were retrieved. Of particular interest is a building in Osaka that almost reached an average of 0.5% drift ratio – this, with a ground level input motion of ~3% g is significant. What might have happened during an event with input level motion in 10-20% g range and is a legitimate question that must be pondered. The particular building had serious site effects and was in resonance.

Another example is from Tokyo, where approximate computations showed that average drift ratio may have reached ~0.7% and maximum drift ratio likely was > 1% which is the maximum allowed by Japanese practice for buildings taller than 60 m and for collapse protection (level 2) motions (The Building Center of Japan, 2001).

These examples indicate that considerably higher drift ratios were affecting the functionality of tall buildings during Tohoku event. Published documents to date on performances of tall buildings during the Tohoku event in general do not describe actual observed or unobserved possible hidden damages. While no-collapses were reported, there is no certainty that hidden damages in some of the buildings do not exist.

Performances of tall buildings in many seismically active regions of the world (e.g. Chile, Turkey) or those tall buildings affected by long distance long period effects by sources at a distance (e.g. Abu Dhabi, Dubai) are of interest to the earthquake engineering community. Chile imposes 0.2 % drift limit that result in elastic design. USA and Turkey impose 2% drift limit. Such wide variations of drift limits in design practices deserve discussion in light of functionality and performance of tall buildings during the 2011 Tohoku event.

INTRODUCTION

Drift ratios are the best measure to infer damage occurrence or likelihood during design/analyses processes and also during analyses of recorded response data if exist (Çelebi, 2008). The purpose of this paper is to discuss the drift ratio issues related to tall buildings in Japan during the Tohoku earthquake of March 11, 2011 (M=9.0) as inferred from recorded response data from two tall buildings. The effect of long period ground motions to buildings and other long period structures is demonstrated by two cases.

Building A at Osaka Bay ~ 770 km from the epicenter of the earthquake

Detailed discussion of the building, data and analyses are provided in Çelebi and others (2012). Figure 1 shows vertical sections and locations of tri-axial accelerometers. Figure 2 shows plan views, orientation of the building and location of accelerometers at the 52nd floor.

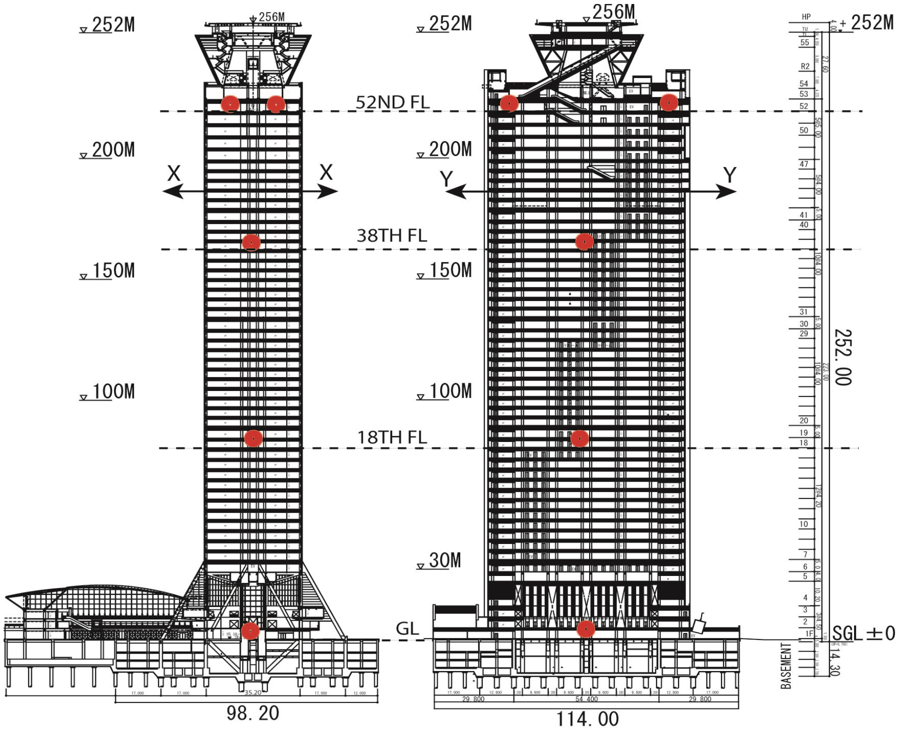

Figure 1. Vertical sections of the building showing major dimensions and locations of tri-axial accelerometers on the 52n, 38th, 18th and ground level (1st Floor). X and Y denote principal axes of the building (see Figure 2) [from Çelebi and others, 2012].

The building was subjected to an input motion at ground level with peak accelerations of ~3% g during the Tohoku earthquake. Furthermore, the building was founded at a site with a site frequency (period) of 0.13-0.17 Hz (5.88-7.69 s) as seen in Figure 3 (left)[1] (Çelebi et al 2012). The building exhibited fundamental frequencies (periods) of 0.152Hz (6.58 s) in the X-direction and 0.145 Hz (6.90) in the Y-direction as seen in Figure 3 (right) . Such close frequencies are clear confirmation of resonance that led to prolonged responses (~1000s recorded) and long duration high amplitude shaking as exhibited in Figure 4 which shows average drift ratios.

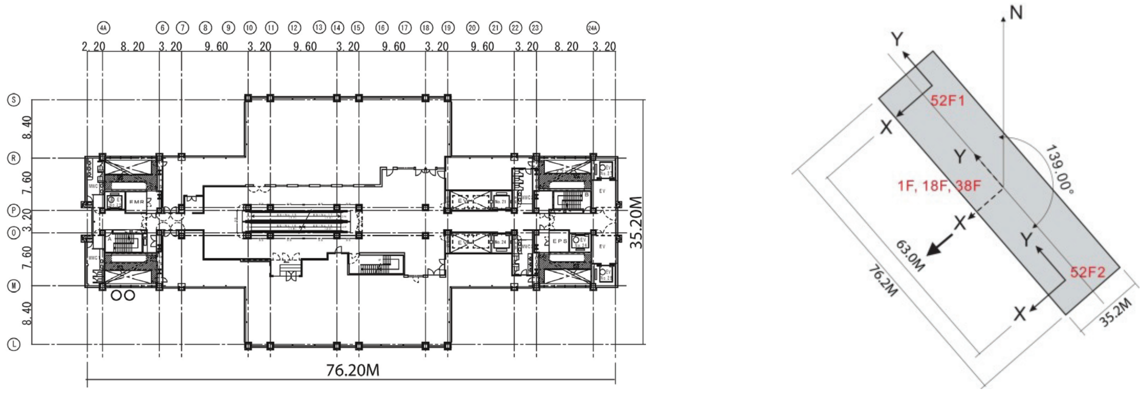

Figure 2. (Left) Typical Plan View (the figure shows 52nd floor) and (Right) Orientations [from Çelebi and others, 2012].

The average drift ratios (Figure 4) computed from relative displacements between many floors indicate that maximum average drift ratios experienced during the mainshock were 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 and 2001b]). 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. 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).

---

[1] The parameters used in computing the site transfer functions are the (depth vs. Vs) Profiles A, B, and C shown in Figure 3 (left). 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 3 (left), 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 3 [left]).

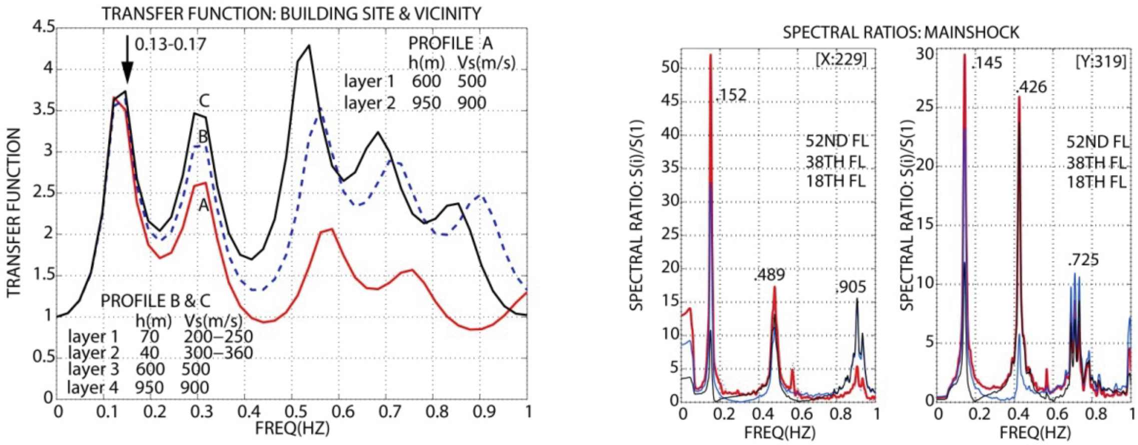

Figure 3. (Left) Transfer functions computed for Profile A (near the OSKH02 strong-motion site, www.kik.bosai.go.jp/, last accessed 09/16/2011)) and Profiles B and C below the building. The depth of the softer upper two layers (to about 1500 m depth) below the building do not significantly change the position of the peaks in the transfer function, particularly for the fundamental mode of the site. (Right) 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.

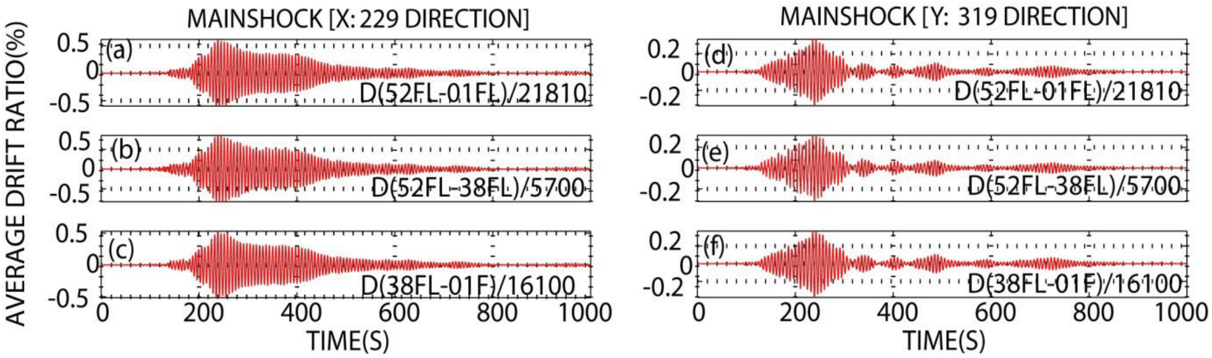

Figure 4. Average drift ratios computed from displacements between 52nd, 38th and 1st floors. In each frame, the numbers in denominators are distances (in cm) between the designated floors.

In summary, during the Tohoku event, the building was subjected to ground level input acceleration of 3% of g. But the deformations were large enough (to realize sizeable average drift ratios) that the building lost its functionality for several weeks. Elevator cables were entangled. This case, along with others, deserves further studies as to how tall buildings would respond if the ground input accelerations were 20-30% of g with similar low frequency content that could be caused by a closer earthquake (e.g. ~100 km or less) [e.g. similar to 1923 Kanto and 1994 Kobe earthquakes]. It is important to note that same elevator cable tangling problems also occurred in several tall buildings in the Shinjuku area of Tokyo, Japan (Hisada et al, 2012).

Furthermore, the actual drift ratios computed from relative displacements divided by story heights between some of the pairs of two consecutive floors are certainly to be larger than the average drift ratios computed from differential displacements between several floors, due to sparse deployments of instruments, as in this case.

Building B – 55 Story Shinjuku Center Building in Shinjuku, Tokyo

According to EERI Special Earthquake Report (EERI Newsletter, 2012), the 54-story Shinjuku Center Building was constructed in 1979. The report states: “The structure’s height is 223m, and the first natural period of the structure is 5.2 and 6.2 seconds in two perpendicular directions. In 2009, the building was retrofitted from the 15th to 39th floor with 288 oil dampers that were configured to exhibit a form of deformation dependency, in addition to velocity dependency. In the March quake, the dampers were calculated to have reduced the maximum accelerations by 30% and roof displacement by 22%.” Figure 5 shows a picture of the building and the oil dampers installed in 2009.

Figure 5. (Left) Picture of Shinjuku Center Building , and (right) dampers installed in 2009 (from EERI Newsletter, 2012).

Figure 6 shows recorded acceleration time-history at first floor and computed displacements at roof during the Tohoku earthquake at ~375 km from the epicenter. As shown in Figure 6, the maximum roof displacement is indicated to be ~ 150 cm (figure from *pers. comm.* J. Moehle, 2012)[2]. Assuming that this is the same as the maximum relative displacement between ground floor and the roof, with a height of 223 m, the average drift ratio can be assumed to be ~150/22300 ~.67%. This is the average. Hence, the maximum drift ratio is >.67% even with claimed reductions in maximum accelerations and displacements thanks to the dampers. Without actual data, it is not possible to compute possibly how much larger than 0.67% is the drift ratio between any two consecutive floors.

Again, similar to the discussion related to Building A, the actual drift ratios computed from relative displacements divided by story heights between some of the pairs of two consecutive floors are certainly to be larger than the average drift ratio computed using the maximum roof displacement divided by the height of the building, due to unavailable detailed data, as done in this case.

---

[2] It is important to note that Hosozawa et al (2012) as well as Takewaki et al (2011) report approximately ~60cm of roof displacement (~40 % of the 150 cm reported above) for this building. The author attributes this to possible different methods of processing of data. Nonetheless, the general conclusions drawn in this paper should not change.

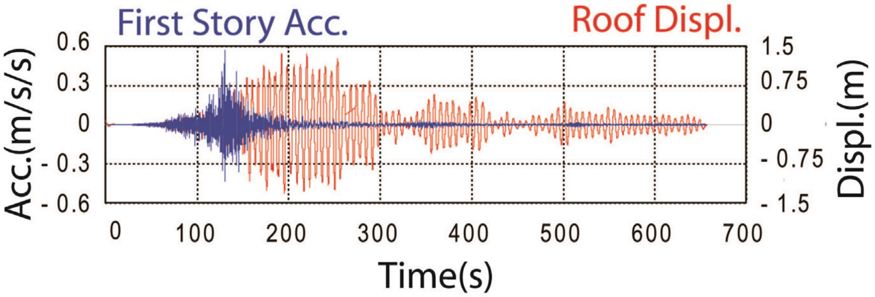

Figure 5. Recorded first story accelerations (max ~6% of g) and computed roof displacements (max ~ 150 cm) during Tohoku earthquake (figures courtesy, J. Moehle).

DISCUSSION AND CONCLUSIONS

The two examples presented herein demonstrate that many tall buildings at far distances from the epicenter of M9.0 Tohoku earthquake of March 11, 2011 were affected by long period ground motions. Tall buildings in Tokyo (~350-375 km from epicenter) and Osaka (~770 km from the epicenter) shook for long durations. In addition to these examples, other examples are discussed in Takewaki et al (2011).

For small ground level input ground motions as in the two cases presented herein, these two tall buildings deformed significantly to experience sizeable drift ratios. The drift ratios discussed herein are computed from recorded response data from accelerometers that are sparsely deployed in both buildings; hence, the reason for referring them as average drift ratios. Therefore, it is deduced that, for some pairs of consecutive floors, the actual drift ratios are larger than the average. Collection of such data is essential (a) to assess the effect of long period ground motions on long period structures caused by sources at large distances, and (b) to consider these effects and discuss whether the design processes should consider reducing drift limits to more realistic percentages and lower them to significantly less than 1% as in Japan[3] much lower than 2% as in USA[4] and perhaps at levels closer to 0.2 % as in Chile[5], and (c) finally, further applications of unique response modification features are feasible to reduce the drift ratios.

Finally, the following points are made:

1. Behavior and performances of these particular tall buildings far away from the strong shaking source of the M9.0 Tohoku earthquake of 2011 and large magnitude aftershocks 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.

2. The risk from closer large-magnitude earthquakes that could subject the buildings 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 Tohoku earthquake of 2011.

---

[3] Maximum limit of drift ratio is 1% in Japan for collapse protection level motions (level 2 used for buildings 60 m or taller [The Building Center of Japan, 2001a and 2001b]).

[4] 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).

[5] The Chilean Code for Earthquake Resistant Design of Buildings (NCh433.Of96, 1996)

3. Prolonged cyclic behavior of the buildings, 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 buildings are modified, low-cycle fatigue can be an important concern for the health of the building in the future.

REFERENCES:

ASCE/SEI Standard 7-10, 2007. Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers.

Çelebi, M., 2008, Real-Time Monitoring of Drift for occupancy Resumption, PROC. 14WCEE (CD-ROM), Beijing, China, Oct.13-17, 2008.

Çelebi, M., Okawa, I., and Kashima, T., S. Koyama, and Iiba, M., 2012, Response of a tall building far from the epicenter of the March 11, 2011 M=9.0 Great East Japan earthquake and its aftershocks, Journal of Design of Tall Buildings and Special Structures (a John Wiley Journal).

Çelebi, M., Sereci, M., Boroschek, R., Carreno, R. and Bonelli, P., 2011, Identifying the Dynamic Characteristics of a Dual core-wall and frame Building in Chile using aftershocks of the 27 February 2010 (M_w=8.8) Maule (Chile) Earthquake**,** *paper in print***,** to appear in May 2013, Earthquake Spectra.
EERI Newsletter (2012).

EERI special earthquake report (January 2012)- Learning from earthquakes "Performance of Engineered Structures in the Mw 9.0 Tohoku, Japan, Earthquake of March 11, 2011

Hisada, Y., Yamashita, T., Murakami, M., Kubo, T., Shindo, J., Aizawa, K., and Arata, T., 2012 Seismic response and damage of high-rise buildings in Tokyo, Japan, during the Great East Japan earthquake, PROC. Int'l Symposium on Engineering Lessons Learned from the 2011 Great East Japan Earthquake, March 1-4, 2012 Tokyo, Japan, pp. 1110-1119.

Hosozawa. O., Kimura, Y., and Aono, H., 2012, Seismic retrofit of high-rise building with deformation-dependent oil dampers, PROC 2nd International Conference on Archi-Cultural Translations through the Silk Road, Mukogawa Women's Univ., Nishinomiya, Japan, July 14-16, 2012.

NCh433.Of96 (Earthquake Resistant Design of Buildings).1996. Instituto de Normalization (INN), Chile.

Takewaki, I., Murakami, S., Fujita, K., Yoshitomi, S., and Tsuji, M., 2011, The 2011 off the Pacific coast of Tohoku earthquake and response of high-rise buildings under long-period ground motions, Soil Dynamics and Earthquake Engineering, 31, pp. 1511-1528.

The Building Center of Japan, 2001a. Time history response analysis building performance evaluation manual, Technical appraisal Department, Structural safety Section Report No: BR KO-02-01 (adopted June 1, 2000, amended April 25, 2001).

The Building Center of Japan, 2001b. Manual for time history response analysis of building performance evaluation manual, Technical appraisal Department, Structural safety Section Report No: BR KO-02-01 (adopted June 1, 2000, amended April 25, 2001).