March 11, 2011 M=9.0 Great East Japan earthquake: The story of a retrofitted building damaged and repaired

M. Çelebi.
Earthquake Science Center,USGS, Menlo Park, Ca. 94025,USA

I. Okawa & T. Kashima
Building Research Institute, Tsukuba, Japan

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SUMMARY:
The March 11, 2011 M=9.0 Great East Japan generated long duration strong shaking that affected the performance of tall buildings and other long-period structures, hundreds of km distant from the epicenter. The records obtained from a 9-story building that was damaged during the earthquake are of great interest to the engineering community. The historical record of the building is interesting due to repeated cycles of being damaged during an earthquake and subsequent retrofit that include the Great East Japan event and its aftershocks. In this paper we focus on the records obtained from the building two days before, during, and after the mainshock to study temporal changes in its dynamic characteristics. Analyses of these records reveal significant shifts in dominant response frequencies that likely reflect changes in stiffness that occurred due to damage suffered during the mainshock, and later frequency shifts after temporary repairs were made two months after the mainshock.

Keywords: Building, damage, 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) near the east coast of Honshu, Japan, with epicentral coordinates 38.322°N, 142.369°E and at a depth of 32 km (http://earthquake.usgs.gov/earthquakes/eqinthenews/2011/usc0001xgp/, last visited 9/21/2011). As one of the largest events ever recorded, the earthquake caused a major disaster in Japan and generated one of the most significant tsunamis, a tsunami which left its mark by destroying the four-unit Fukushima nuclear power plant and by causing the greatest percentage of the 15,776 fatalities[1] associated with this event. 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 (discussed in detail elsewhere [Çelebi and others, 2012a and 2012b]) 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, at hundreds of km distance from the epicenter, were strongly affected, but only a small number of buildings were damaged by the shaking, in contrast to the many buildings damaged by the tsunami. Furthermore, publically available recorded responses of buildings and other structures damaged during earthquakes are rare. Therefore, one of the buildings of interest to the engineering community in Japan and elsewhere is an instrumented 9-story building that was damaged during previous earthquakes, retrofitted, damaged during the Great East Japan event, and then temporarily repaired two months after the mainshock. Its shaking response two days before and during the mainshock, during numerous aftershocks following the mainshock, and after the implementation of the temporary repair were successfully recorded by two tri-axial accelerometers in the building. Furthermore, response data were also recorded from events several months and years prior to the mainshock. The purpose and motivation of this paper is to present and

[1] *from* The Japanese National Police Agency (http://www.npa.go.jp/archive/keibi/biki/higaijokyo.pdf, last visited 9/20/2011) as confirming "15,776 deaths, 5,694 injured and 4,980 people missing across eighteen prefectures, as well as over 125,000 buildings damaged or destroyed".

study the records obtained with the limited instruments from two days before, during, after the mainshock and after the implementation of the temporary repair. Spectral methods are used to analyze the data for indications of variations in the dynamic characteristics of the building due to changes in physical conditions over the time period of about five months. Reconnaissance observations related to the mainshock of the earthquake are outside the scope of this paper. Furthermore, ambient and/or microtremor response data from the building (e.g., Motosaka and others, 2011) were not available to us and therefore are not used in this study.

2. THE BUILDING, ITS SITE AND INSTRUMENTS

The 9-story, vertically irregular building (identified subsequently as THU) is on the Tohoku University Campus (Sendai, Japan), 177 km from the epicenter of the mainshock of the March 11, 2011 event. Figure 1 (Left) shows location of the building relative to the epicenter of the mainshock. The figure also shows the locations of some of the nearby free-field stations of the KNET[2] and KIKNET[2] networks of Japan. Figure 1 (Right) shows relative locations of the THU and epicenters of the mainshock and four other events, whose data are included in this study.

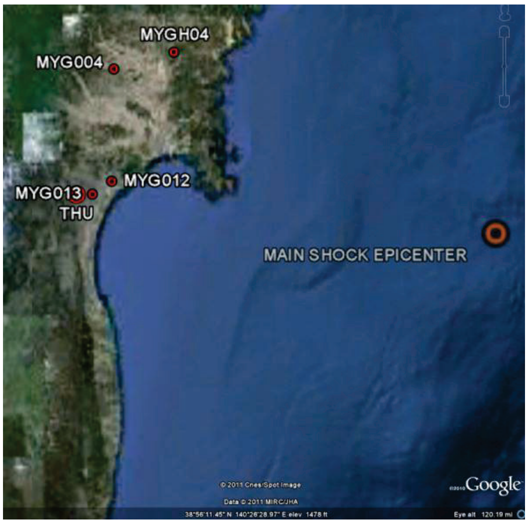


Figure 1. (Left) Google Earth Map showing the location of the 9-story building (THU), location of the mainshock epicenter (large circle) and some of the KNET and KIKNET stations closest to the building in Sendai City, Miyagi Prefacture, Japan. (Right) Epicenters of events considered in this study.

Two vertical sections and two plan views of the building are shown in Figure 2. Starting on the 3rd floor, the plan area of the building reduces from 33.6mx72.0m to13.5mx40.0m. The figure also shows approximate locations and orientations of the horizontal channels of the two tri-axial seismic instruments deployed by Building Research Institute (BRI), Tsukuba, Japan. These two tri-axial instruments provide very sparse coverage in terms of understanding the behavior and performance of the building. Details of BRI strong-motion deployments and data are at http://smo.kenken.go.jp/ (last visited August 5, 2011). The two tri-axial instruments are vertically aligned near one of the walls. It is important to mention that a separate array is also deployed in the building, which serves as a real-time seismic health monitoring system for researchers of Tohoku University (Motosaka and others, 2004). This additional array comprises tri-axial accelerometers at each of the 1st, 5th and 9Th floors. Data from this privately owned system is not publically available. This system, as well as temporary deployments of additional sensors, have been used by Tohoku University researchers to record several sets of ambient and/or microtremor data during different times. Such ambient and/or microtremor data have been used to infer torsional response characteristics as well as variation of the frequencies with amplitude of shaking (Motosaka, 2009, Motosaka and others, 2004) during different physical conditions of the building. In this study, only BRI earthquake data obtained with the two tri-axial accelerometers are used (http://smo.kenken.go.jp).

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

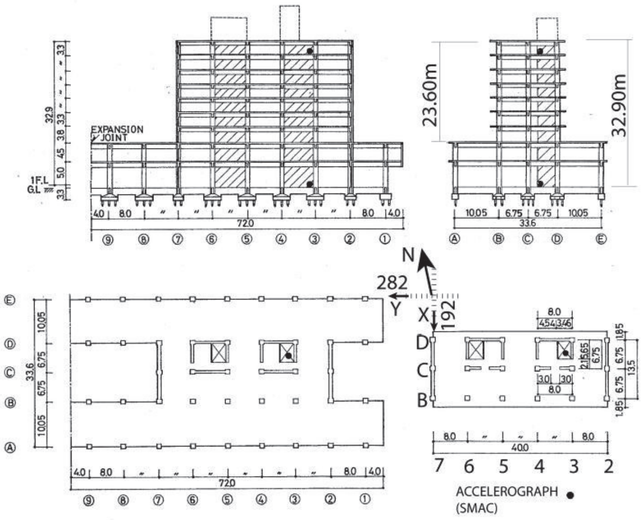
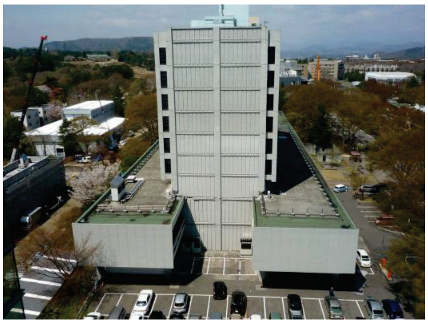


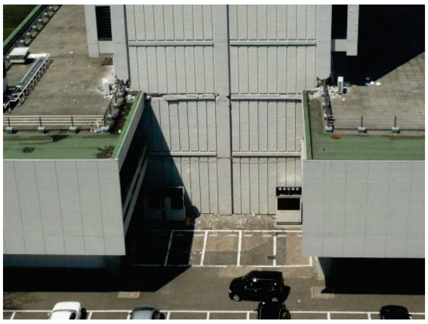
Figure 2. Vertical sections and typical plans. key column lines of the plan view of the tower, heights of the tower and complete building are highlighted. Approximate locations of the two tri-axial sensor/recorders are shown by solid circles. SMAC is a trademark of strong-motion accelerograph manufactured in Japan[3]. Arrows indicate the positive directions of the horizontal components of the sensors as X(192) and Y(282). Figure modified from Shiga and others (1981.

Constructed in 1969, the building has a long history of shaking during small, medium and large earthquakes that occurred both nearby and at large distances. Furthermore, Shiga and others (1973) reported on forced vibration dynamic tests and earthquake data analyses and found differences between experimental data and analytical results. There are documented earthquake-inflicted damages on the building and resulting related studies prior to the 2011 event as well. For example, the building was damaged during the June 12, 1978 Miyagi-ken Oki earthquake (M=7.7) when the peak acceleration recorded on the 9th floor was ~1g. Detailed information on the building and its site, its performance during the 1978 event, and analyses of the then analog recorded data are also available in National Building Standards Report 592 (Ellingwood [ed.], 1980). That document reports that the zero period design acceleration of the building was 0.2 g. Furthermore, the report states that the original design is a reinforced concrete frame structure built on a pile foundation and that during the 1978 Miyagi-ken-Oki event, when the recorded peak accelerations were 0.24g and 1.0 g at the 1st and 9th floors respectively, there was cracking in the structural frame system. An earlier M=6.5 earthquake on February 20, 1978, located off the coast of Honshu, had caused cracks in shear walls and window breakage as well. Significant repair/retrofit work was completed within the 2000-2001 timeframe (Motosaka and others, 2004). The most significant part of the retrofit program was for seismic lateral strengthening and included the construction of shear walls for the whole height of the building at bays between B and C and along column lines 2 and 7 (Figure 2). Before and after the retrofit, Motosaka and others (2004) performed forced vibration tests and reported the changes in the dynamic characteristics of the building due to the retrofit. Since the retrofit, the building was also tested by several earthquakes, including the M=7.2 Off Miyagi (August 16, 2005) and the M=6.8 Iwate Prefacture (July 24, 2008) events. A comprehensive summary of the chronological changes in the lateral resistance capabilities and dynamic characteristics of the building is provided by Motosaka and others (2004), Motosaka (2009) and Tanaka and Motosaka (2009) and therefore will not be repeated herein. However, for the purposes of this paper, only the relevant dynamic characteristics identified from an event two days before the mainshock of the Great East Japan earthquake will be used as a baseline to study the changes thereafter. Site effects to the response of the building are deemed not to be significant but are discussed in detail elsewhere (Çelebi and others, 2012b).

[3] Mentioning commercial names is for descriptive purposes only and does not constitute endorsement of the product or hardware.

In Figure 3, the first four pictures show general views of the building and also damage to a column and end walls. The visible damage is at the 3rd floor level where the plan area and therefore the stiffness of the floor changes. Unfortunately, lack of seismic sensors at that and neighboring floors negated direct recording of the effect of change of stiffness on the behavior and performance of the building. The last two pictures (e,f) in the figure depict the part of visible temporary repair performed in May 2011 using tension-steel bars. Motosaka (*written communication*, 2011) described the repair as "the damaged part of the 3rd floor was reinforced by adding concrete pier-like bases with tension steel bars to resist axial forces induced by bending vibration. Also, four shear walls have been constructed next to the damaged four corner columns to resist LN (longitudinal [282]) direction force". Therefore, one of the five events studied herein is deliberately selected to identify the changes in structural frequencies due to the post-mainshock temporary repair.


(a)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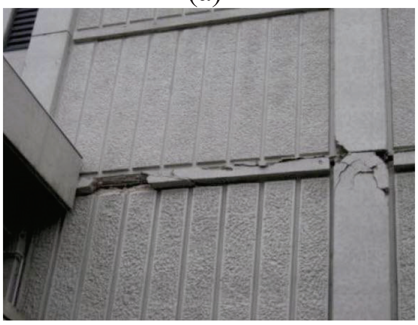
(c)


(d)


(e)


(f)

Figure 3. (a) General view of the building and (b) damage to a column and end wall (top row photos: courtesy of T. Kashima), center row: (c,d) adopted from Motosaka (2011), bottom row: (e,f) post-mainshock repairs.

3. RECORDED BUILDING RESPONSES AND ANALYSES

Beginning in 1987 and through July 15, 2011, THU building responses from 172 events have been recorded. In this paper, records from only five events are used: one pre-mainshock that serves as a baseline, the mainshock, and 3 aftershocks to illustrate the immediate post-earthquake state and the state after repairs. Details about these events and the corresponding peak accelerations at the 1st and 9th floors of the building are summarized in Table 1. Horizontal components of the records from the five events are presented in Figure 4.

These records are impressive both in length for performing different types of spectral analyses, and in the variations in amplitude to infer the behavior and performance of the building affected by significantly large peak accelerations as in the mainshock of the event (~0.33g at 1st floor and ~0.9 g at the 9th floor). However, the sparse instrumentation precludes direct assessment of torsional or rocking behavior of the building. Furthermore, the data are insufficient for determining where damage might be localized in order to compare such results with the physically observed damage (e.g. Figure 3). In such a case, cross-spectral techniques to infer the relationships between orthogonal data acquired at the same point is possibly the only means to infer torsional characteristics of the building.

Table 1. Events and particulars recorded by THU building instruments [http://smo.kenken.go.jp/]

Event	Time	Name of Event& Epicenter Coordinates	M (JMA)	Dist (km)	Largest Peak Acc (gals)1st Fl./9th Fl
BEFORE M=9.0 GREAT EAST JAPAN EVENT					
1	201103091145	Off Sanriku 38^{o}19’41”N,143^{o}16’41”E	7.3	212	34.8/154
M=9.0 GREAT EAST JAPAN EVENT 2(*) AND AFTERSHOCKS USED IN THIS STUDY					
2 (*)	201103111446	Off Sanriku [Mainshock] 38^{o}06’11”N,142^{o}51’36”E	9.0	177	333/906
3	201103111515	Off Ibaraki Pref 36^{o}06’29”N,141^{o}15’53”E	7.7	241	32/66
4	201104111716	Hama-dori, Fukushima Pref 36^{o}56’42”N,140^{o}40’18”E	7.0	146	67.9/170
5	201107100957	Off Sanriku 38^{o}01’54”N,143^{o}30’24”E	7.3	234	20.6/92

3.1. Baseline: Event 1

To establish baseline dynamic characteristics of the building shortly before the mainshock, data from Event 1 are used. In Figure 5 (Left), amplitude spectra of the two orthogonal horizontal acceleration time series at the 9th floor are shown. Several peaks are observed. The frequency at 1.26 Hz appears in the spectrum of accelerations in the X-direction and the one at 1.22 Hz appears in the spectrum of accelerations in the Y-direction. The frequency at ~1.15Hz appears in both spectra. From this observation, it is difficult to distinguish with certainty which frequency belongs to which mode of the building. Better clarification is provided by Figure 5 (right) which shows the auto-spectra of and cross-spectrum between the same 9th floor orthogonal accelerations. With improved confidence, the 1.26 Hz and 1.22 Hz frequencies are the translational frequencies in the X- and Y-directions. The 1.15 Hz frequency that appears in all three spectra is most likely a torsional frequency[4].

Plot of cross-correlation of the two orthogonal motions at the 9th floor also does not clearly identify the torsional frequency. Figure 6 shows the amplitude spectra of accelerations at the 9th floor and 1st floor and spectral ratios of the two for the two orthogonal directions. Similar peaks are identifiable in this figure but it is not possible to infer that one belongs to torsional mode. Furthermore, it is noted in this figure that the amplitude spectra at the 1st floor and 9th floor have significant peaks that are between 0.5-1.0 Hz. However, the ratios of spectra are mostly near unity within frequency range 0-0.7Hz– indicating that the frequencies associated with these peaks are not structural but are due to the input ground motions. This is an important observation to note for assessing similar frequencies identified from recorded motions during the mainshock as discussed later. Earlier, it was concluded that the site frequencies are larger (~4 Hz) than the structural frequencies; hence, these peaks do not belong to site frequencies.

[4] It is important to note that, under different physical conditions (before and after the 2000-2001 retrofit of the building) that are not comparable to the physical condition of the building at the time of Event 1, Motosaka and others (2004) reported identified torsional frequencies between 2.05-2.55Hz from their analyses of forced vibration and microtremor data obtained with a denser distribution of temporarily deployed sensors.

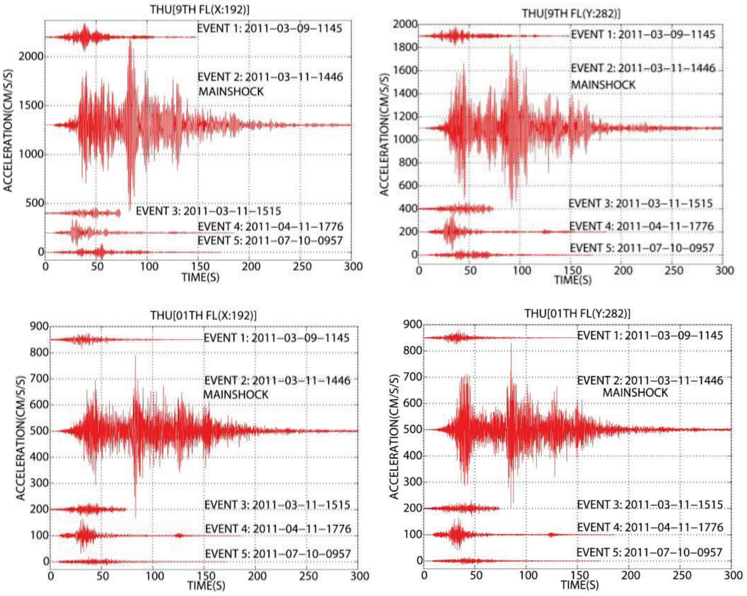

Figure 4. Horizontal acceleration records in the X (left column) and Y (right column) directions at the 9^{th} (top row) and 1st (bottom row) floors of the building for the 5 events. Note that within each frame the amplitudes are normalized by the largest PGA, but the scaling varies frame-to-frame in order to provide the maximum resolution of acceleration within each frame.

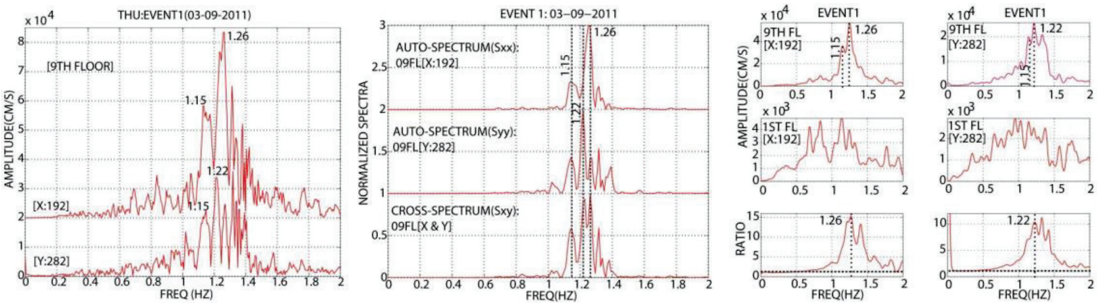

Figure 5. (Left) Amplitude spectra of orthogonal horizontal accelerations at the 9th floor of the building. (Center) Auto-spectra of and cross-spectrum between the same two accelerations. (Right) Amplitude spectra and spectral ratios of the two orthogonal horizontal accelerations (Event 1) at the 9th floor and 1st floor do not allow determination of torsional frequency. Note the differing scales of amplitude spectra. Note also that the spectral ratio is ~1 up to 0.7 Hz, indicating that there are no structural frequencies within that range.

3.2. Mainshock Response

The accelerations recorded by the two tri-axial instruments during the mainshock (Event 2) and the displacements computed by double integration of these time series are shown in Figure 6.

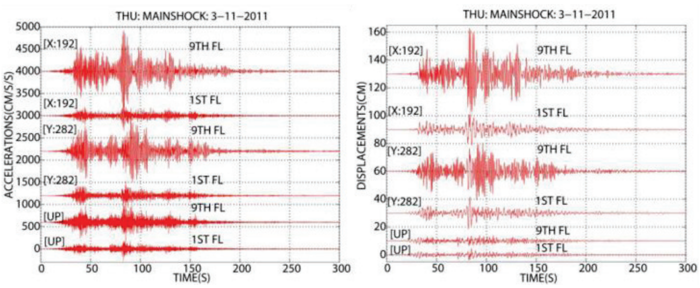

Figure 6. Time-history plots of all six channels of accelerations (left) recorded during the mainshock and computed displacements (right).

Acceleration time-histories of the two orthogonal horizontal accelerations recorded at the 9th floor and 1st floor, their amplitude spectra computed for the two time windows [A(0-70 s) and B (70-150s) into the record], and respective spectral ratios are shown in Figure 7 (left). Compared with those from

Event 1 in Figure 5, Figure 7 (left) reveals a sizeable shift of the frequencies of significant peaks to below 1Hz. However, it is clear that there is even a shift of frequencies in the time period A when compared to the frequencies of Event 1. This indicates that significant damage occurred within each of the time windows. That the shift in structural frequencies, in this case due to documented and observable physical damage to the structure, occurred during the specified time windows is also clearly demonstrated in Figure 7 (center and right) by the moving-window amplitude spectra of the 9th floor accelerations. From Figure 7(left), the frequencies of the damaged building are determined to be 0.78 and 0.84 Hz in the X and Y directions, respectively.

Furthermore, the spectral peaks between 0.5-1 Hz are much larger than unity – indicating that they are frequencies of the damaged building - since the possibility of site resonance was ruled out earlier.
From the frequencies of Event 1 and Event 2 (mainshock), absent any other nonlinearities besides the physical damage inflicted on the building, one simple way of approximating reduction of overall stiffness is from ratios of $f_1=(1/2\pi)(k_1/m)^{0.5}$ and $f_2=(1/2\pi)(k_2/m)^{0.5}$. This yields $k_2=k_1(f_2/f_1)^2$. Thus, in the X direction $k_2(X)=k_1(X)(0.78/1.26)^2 \sim 0.383k_1(X)$, and in the Y-direction $k_2(Y)=k_1(Y)(0.84/1.22)2\sim 0.474k_1(Y)$. These are very significant reductions in stiffnesses (~61% and ~52% in the X and Y directions respectively).

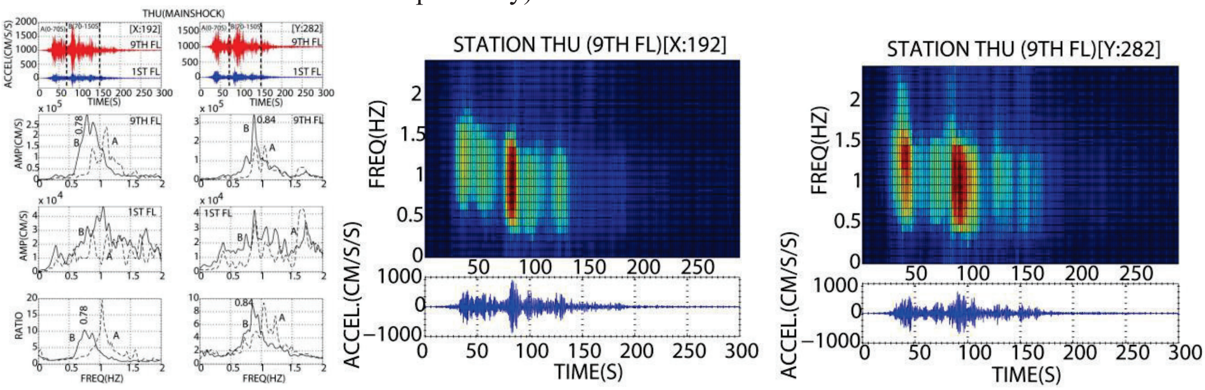

Figure 7. (Left) Acceleration time-histories of the two orthogonal horizontal accelerations recorded during the mainshock at the 9th floor, their amplitude spectra and ratios showing the changes in the frequencies from those within the first 70 second period to those between the 70-150 seconds into the record. Note the shift of the significant peaks after 70 seconds into the record. (Center and Right) Moving window amplitude spectra indicates the clear shift of predominant frequencies for both horizontal components.

Damage that occurred is also confirmed by quantified drift ratios presented in Figure 8. However, as stated earlier, due to sparse instruments, it is only possible to compute average drift ratios using relative displacements between the 9th and 1st floors divided by 3290 cm which is the height between 1st floor and 9th floor (Figure 2). This evaluation is presented with the understanding that it is very likely that the average drift ratio of the building above the third floor (where the plan area decreases significantly) was most likely significantly larger than the first 2 floors of the building. It is noted that, in the X-direction, ~1% average drift ratio is experienced at about 83-84 seconds into the record, and 0.5% and above average drift ratio persists between 82 to 88 seconds and then between 97-102 seconds. In the Y-direction, 0.5% average drift ratio is experienced between 90-98 seconds into the record.

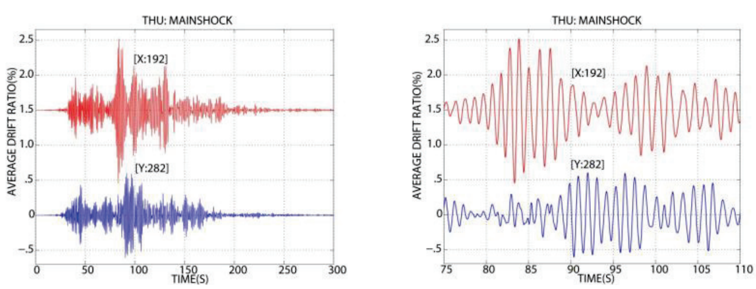

Figure 8. Equiscaled average drift ratios (in %) computed from relative displacements between the 9th and 1st floors.

In summary, the 0.5% and above drift ratios experienced during the main shock were at levels known to cause damage, or at least are indicators of the start of non-linearity due to inelastic behavior. In this case, the threshold is exceeded during multiple cycles in two intervals, as illustrated in Figure 8 (right). This observation is not surprising, since the peak acceleration at the 1st floor of the building is ~0.33 g (Table 1), a level considerably above the 0.2 g ZPA (zero period acceleration) used in the original design of the building that does not reflect the effect of the 2000-2001 retrofit, and is at a level comparable to the ZPA (for Sendai: 0.32 g) for limit capacity design (Table 7.2.5, AIJ, 2004) corresponding to an extremely rare earthquake. Furthermore, the 1% drift ratio is the maximum limit for design of buildings in Japan for the collapse protection (level 2) motions (The Building Center of Japan, 2001a and 2001b). In the United States, the comparative maximum drift ratio for similar buildings for Risk Category 1 or 2 is 2% (Table 12.12, ASCE7-10, 2007).

Critical damping percentages were approximately assessed only for the mainshock by both logarithmic decrement method using Figure 8 (right) and by system identification methods. The damping percentage was estimated to be between 6.5-8.5% of critical.

The possibility of identifying torsional frequencies was explored for the 70-300 second window (presumably after damage occurred) of the orthogonal accelerations at the 9th floor. Auto-spectra (Sxx and Syy) and cross-spectrum (Sxy) as well as phase angle and coherency plots of the two orthogonal channels at the 9th floor are shown in Figure 9. From the auto-spectra, the fundamental frequencies in the X and Y directions are again clearly identified as 0.78 and 0.84 Hz with high coherency. However, two additional frequencies at 0.7 Hz and 1.15 Hz also appear in the cross-spectrum. Of these, the 0.70 Hz frequency has 0^o phase and the 1.15Hz frequency has ~180^o phase. Both have relatively high coherencies. One possible explanation is that there may be two torsional modes – one at 0.70 Hz corresponding to the part of the building above the third floor where the plan, and therefore the stiffness, changes drastically, and where concentrated damage was observed, and the other at 1.15Hz for the total building. Another possibility is that one may be first torsional mode and the other second.

Again, lack of additional strategically deployed channels of sensors prevents straightforward and reliable identification of torsional frequencies.

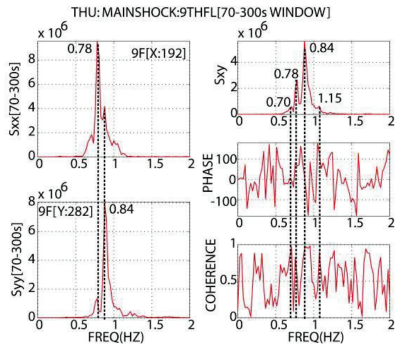

Figure 9. Auto-spectra, cross-spectrum, phase angle and coherency of and between the two orthogonal motions at the 9th floor for the 70-300 second window of accelerations.

3.3. All Five Events

Figure 10 shows both normalized amplitude spectra of 9th floor accelerations in the two orthogonal horizontal directions for all 5 events, and normalized spectral ratios of 9th floor accelerations with respect to 1st floor accelerations. Clearly, the chronological shifts in the fundamental frequencies are displayed. It is again noted that damage occurred during the mainshock (Event 2) and temporary reinforcement was implemented after event 4 but before event 5. Table 2 summarizes the changes and compares them with analyses by Motosaka and others (2011).

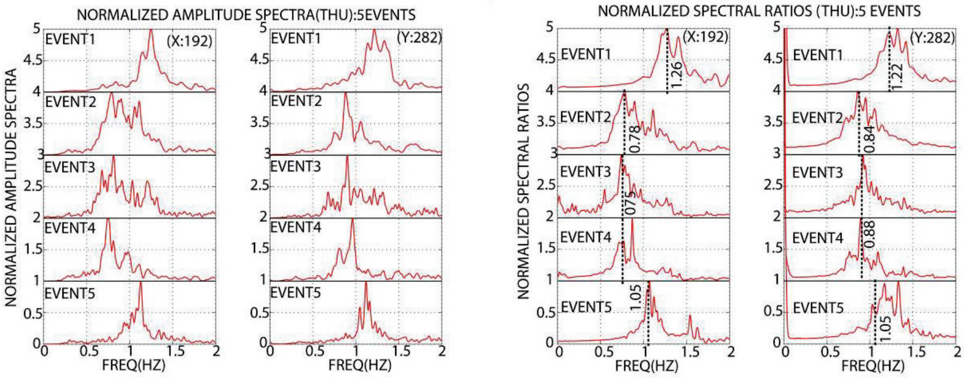

Figure 10. (Left) Normalized amplitude spectra of 9th floor accelerations and (Right) Normalized spectral ratios of 9th floor accelerations with respect to 1st floor accelerations for all five events. These plots depict the changes in the dominant frequencies in the building response which most likely are due to physical changes in the lateral stiffness of the building.

Table 2. Events and Comparison of Identified Frequencies (Hz)

Events	This Study			Motosaka and others (2011)		
	X	Y	Torsion	X	Y	Torsion
BEFORE M=9.0 GREAT EAST JAPAN EVENT						
Event 1 (Baseline) 201103091145	1.26	1.22	1.15?	1.26*	1.26*	
M=9.0 GREAT EAST JAPAN EVENT 2(***) AND AFTERSHOCKS						
Event 2(***) (Mainshock) 201103111446	0.78	0.84	1.15? or 0.7?	0.78*	0.88*	
Event 3 201103111515	0.75	0.88				
Event 4 201104111716	0.75	0.88				
Event 5 (after temporary strengthening) 201107100957	1.05	1.05		**	**	**
*Motosaka and others (2011) report that for both event days, ambient data yielded 1.61 Hz in both directions. ** Using low-amplitude microtremor data, Motosaka (written communication, 2011) assessed the frequency change before and after temporary strengthening as : in the X direction (1.17 Hz to 1.37 Hz), Y(1.36 Hz to 1.48 Hz), torsional(1.87 Hz to 2.04 Hz).						

4. CONCLUSIONS

Important temporal variations in the response characteristics of a 9-story building at Tohoku University were identified using the records from two tri-axial accelerographs. The duration of strong shaking of the building from the 2011 Tohoku earthquake was on the order of 100 s, and peak accelerations reached ~0.33g on the 1st floor and 0.9 g on the 9th floor. Significant shifts of the predominant frequencies in the spectra of the response data occurred during the strong shaking and are inferred to reflect the co-seismic damage that was observed following the earthquake. Absent any other non-linearities, the identified frequencies represent approximately 61% and 52% reductions of stiffness in the X- and Y-directions, respectively, during the mainshock as compared the baseline event of two days before the mainshock. A subsequent shift in the predominant frequencies was identified in the records from a large aftershock that occurred after partial repair was completed about two months after the mainshock.

Although the vertical alignment of the instruments deployed on the 1st and 9th floors is poorly suited for assessing torsional behavior of the building, cross-spectral analysis of the records reveal spectral peaks at frequencies distinct from the translational motions that likely are related to torsional responses. Additional accelerometers deployed in a more extensive configuration are needed to better identify and reliably compute pertinent dynamic characteristics of the building.

ACKNOWLEDGEMENTS

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