# LESSONS OF THE 2011 GREAT EAST JAPAN EARTHQUAKE FOCUSED ON CHARACTERISTICS OF GROUND MOTIONS AND BUILDING DAMAGE

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**ABSTRACT**: This paper starts from overview of ground motion characteristics, the observed high acceleration records are discussed together with many long-duration records as specific features. Then, ground motion characteristics during the 2011 earthquake are compared to those during the 1978 Miyagi-ken Oki earthquake. The site amplification characteristics are also discussed based on the authors' network in Sendai in relation to 8- and 9 story buildings' damage of Tohoku University. Pile foundation damage and ceiling board damage are also introduced as specific building damage.

Key Words: Great East Japan earthquake, ground motion characteristics, site specific amplification, building damage, vibration characteristics,

# INTRODUCTION

A major earthquake occurred in the Pacific Ocean off the coast of Miyagi, Japan on March 11, 2011 and caused huge human and physical damage. The moment magnitude of 9.0 is the largest recorded in Japan since modern seismograms became available. The fault plane has dimension with 450km by 200km (Fig.1). The earthquake caused many aftershocks. During three months after the main shock 500 aftershocks with magnitudes more than 5 were occurred. Some induced earthquakes with magnitudes 6 to 7 class were also generated.

Although the most significant aspect of the 2011 Tohoku earthquake is the large Tsunami that attacked the Pacific coast of Japan and claimed about 20,000 lives mainly by tsunamis together with huge amount of physical damage, the earthquake caused structural damage by also severe ground motion with long duration. Most of all the damaged buildings were designed by the old building code and damaged by lack of seismic strength, short column shear failure due to sauce wall and a breast wall, eccentricity of structural elements. Buildings with appropriate seismic reinforcement/retrofit were mostly escaped from damage, which indicates effectiveness of the seismic reinforcement/retrofit. But the structural damage caused by ground motion amplification was recognized. 8- and 9-story buildings at Aobayama campus of Tohoku University were damaged due to ground motion amplification in the site. Many pile foundation building were damaged during the earthquake. As damage of non-structural elements, the tremendous number of ceiling board dropped during the main

shock and the major aftershock. Some of them caused killed persons for the first time.

In this paper, the author starts from overview of observed ground motion characteristics during the earthquake, then investigates the observed high acceleration records with PGA of 2,700 cm/s/s and the corresponding JMA seismic intensity 7 (instrumental intensity: 6.6) at the K-NET Tsukidate station. Structural damage was quite light in the surrounding area are discussed together with many long-duration records as specific features.

Then, ground motion characteristics during the 2011 earthquake are compared to those during the 1978 Miyagi-ken Oki earthquake (M7.4) at the same observation site, basement floor of Sumitomo building near Sendai station, which is recognized as engineering bedrock motion. Site specific ground motion amplification in Sendai area is also addressed based on strong motion networks including the authors' DCRC network. Difference of ground motion due to geological conditions is discussed. Regarding specific building damage of 8- and 9-story buildings at Aobayama campus of Tohoku University, ground motion amplification in the site was discussed based on the observation records at a 9 story SRC building of Departments of Civil Engineering and Architecture (THU building). Dynamic behavior of the damaged THU building due to the amplified ground motion is also discussed together with long-term monitoring of amplitude dependent dynamic characteristics of the building. As specific building damage, the two pile foundation buildings which were damaged during the 1978 earthquake comparatively discussed. An example of the pile foundation damage of the building constructed after the New Seismic Design Code in 1981 is addressed. As damage of non-structural elements, some examples of ceiling board drop damage are introduced. Finally, the author address the learning and lessons from the 2011 earthquake for stronger earthquake countermeasures of urban and building structures.



(after NIED)



Fig.1Slip distribution on the fault plane Fig. 2 Instrumental JMA Seismic Intensity and Observed acceleration waveforms at K-NET stations

# **OBSERVED GROUND MOTION CHRACTERISTICS**

#### Overview

Fig. 2 shows the observed acceleration waveforms in the Tohoku and Kanto regions. In Tohoku region, the characteristic two phases are recognized but in Kanto including Tokyo the only one big phase. Table 1 shows a list of large amplitude observation records in order of JMA seismic intensity, instrument intensity (AIJ, 2011). The large intensity stations are concentrated in Miyagi prefecture, especially in the cities of Kurihara, Sendai and Ohsaki. As for the records in Sendai, detailed information is described in section 3.

				JMA	PGA	PGV
No	Organization	Station	Address	Int.	(cm/s/s)	(cm/s)*
1	NIED	MYG004	Tsukidate, Kurihara	6.6	2700	94
2	TOHTECH	smt.CCHG	Arai, Sendai	6.5	1074	-
3	TOHTECH	smt.IWAK	lwakiri, Sendai	6.4	859	-
4	DCRC	dcr.009	Matsumori, Sendai	6.4	821	88
5	NIED	MYG013	Nigatake, Sendai	6.3	1517	74
6	JMA	4B9	Mikka-machi, Ohsaki	6.2	550	85
7	NIED	MYG006	Kita-machi, Ohsaki	6.1	572	98
8	DCRC	dcr.018	Okino, Sendai	6.1	512	79
9	NIED	FKS016	Shin-shirakawa, Fukusima	6.1	1295	59
10	TOHTECH	smt.NAKI	Nanakita, Sendai	6.1	1853	-
11	DCRC	dcr.023	Oroshimachi, Sendai	6.1	613	77

Table 1 List of large amplitude observation records

\*cut-off period of 50s

At the K-NET Tsukidate (MYG004) station, quite high acceleration records with PGA of 2,700 cm/s/s in NS component and 1880 cm/s/s in UD component were observed. The corresponding JMA seismic intensity 7 (instrumental intensity: 6.6). At the time around 100s, NS and UD components have especially large amplitudes. It is noted that structural damage was quite light in the surrounding area. The authors investigated the high acceleration records and indicated unfavorable behavior with partial uplifting and slipping of the foundation of the seismometer was recognized based on the non-stationary spectral analysis and particle orbit analysis as shown later (Motosaka and Tsamba. 2011).

Fig. 3 shows 5%-damped pseudo velocity response spectra for the major stations in Table 1, together with spectra of Takatori record at the 1/17/1995 Kobe earthquake and Kawaguchi-machi record at the 10/23/2004 Mid Niigata Prefecture earthquake as representatives of the earthquake records in the heavily damaged zone by the past disastrous earthquakes.

Spectra of Takatori and Kawaguchi-machi records are predominant around 1-2 seconds, while spectrum amplitude at MYG004 (Terrace) is large at the shorter periods but the 1-2s amplitude is smaller. On the other hand, MYG006 (Ohsaki plain) and MYG013 (Sendai plain) have larger 1-2s amplitude compared with MYG004, and 4B9 at the center of Furukawa (Ohsaki plain) have similar amplitude level to that of Takatori. Spectral characteristics of MYG004 and MYG006 are discussed next.



Fig.3 Pseudo velocity spectra at observation stations with larger seismic intensity in Tohoku earthquake and past damaged earthquakes

#### Investigation of High Acceleration Records at K-NET Tsukidate Station

The K-NET Tsukidate station (MYG004) is located at the south of parking lot of Kurihara Culture Hall. Observation hut is on a cliff with the EW directionality and height of 3m. The configuration of the observation site including the hut with foundation is shown in Fig.4. The foundation of the seismometer is concrete mass with dimension of 80cm x 80cm and the height of 100cm. Fig.5 shows the observed acceleration waveforms of the 3 components during the March 11, Tohoku earthquake. The horizontal PGA in the NS component is 2,700 gal and that in the EW component is 1,269. Findings from the waveform characteristics of the observation records at K-NET Tsukidate station are as follows.

- 1) The waveforms comprise two distinctive phases and the second phase has larger amplitude.
- 2) The acceleration values are large in the NS and UD components showing pulse-like waveform at the time of 95 seconds in Fig.5
- 3) The dominance of NS component is consistent with the general property of the normal direction's dominance to the cliff, which suggests the topography effect



Fig.4 Configuration of observation site and section of seismometer's foundation (after NIED)



Fig.6 shows acceleration waveforms and the corresponding wavelet basis non-stationary spectra of NS- and UD-components for 3 seconds (from 94s to 97s) at around the time of peak acceleration generation showing in Fig.5. In the wavelet analysis, complex continuous Morlet wavelet is adopted as mother wavelet. Fig.7 shows Fourier amplitude spectra for the same time section. Findings from those figures are as follows.

- 1) For the dominant frequency of 4.2Hz in the horizontal (NS) component, odd number higher harmonics are induced at around 12.6Hz (3 times) and 21.0Hz (5 times).
- 2) Vertical motion with even number higher harmonics is induced at around 8.4Hz (2 times) and around 16.8 Hz (4 times).

These characteristics are consistent with the higher harmonic motion due to partial uplifting of the foundation (Motosaka and Nagano, 1994).

Fig.8 shows acceleration orbit of NS-UD plane in the time section for 3 seconds from 94s to -97s. Non-symmetric behavior is clearly shown. The particle orbit suggests unfavorable behavior with partial uplifting and slipping of the foundation of the seismometer.



Fig.6 Acceleration waveforms for 3 seconds at around the peak value generation time and the corresponding wavelet basis non-stationary spectra



Fig.7 Fourier amplitude spectra for 3 seconds at around the peak value generation time



Fig.8 Acceleration Particle Orbit of NS-UD plane

#### **Comparison of Response Spectra for Past Earthquakes at K-NET Stations**

Fig.9 and Fig. 10 show acceleration-displacement response spectra (Sa-Sd spectra) in the larger amplitude horizontal direction at MYG004 and MYG006. In these figures, the spectra due to the 7/4/2011 after shock and 14/6/2008 Iwate-Miyagi Nairiku earthquake are shown for comparison. It is found form these figures that the observed motion at Tsukidate with JMA seismic intensity 7 is consisted of shorter period contents than 0.3 sec and the displacement at around 1 sec is small as 10cm even if the amount is larger compared to that for the 2008 earthquake.

In the case of K-NET Furukawa, it is found that the displacement during the 2011 earthquake was

larger compared to the 2008 earthquake by comparing the displacement spectral values for the two earthquake and that the response displacement in the period range shorter than 1sec due to aftershock on April 7 is almost the same as that due to main shock. These ground motion characteristics at Tsukidate of Kurihara city and Furukawa of Osaki city are consistent with damage features in the two areas.

T=0.25sec

T=0.5sec

20



Fig.9 Sa-Sd spectra for the earthquake records at MYG004



Sd (cm)

40

Furukawa: MYG006

T=1sec

60

2011.03.11 (M9.0)

2011.04.07 (M7.1)

2008.06.14 (M7.2)

EW

h=5%

T=2sec

100

80

# **Investigation of Long Duration Records**

During the huge earthquake with long duration, building structures were shaken by many numbers of displacement cycles. It has been reported that many lead damper devices were damaged. In Ohsaki city, Miyagi prefecture, a base-isolated building with lead damper device was damaged.

Fig. 11 shows observed acceleration waveform in EW component at K-NET Furukawa station (MYG006) and its displacement response spectra. The dominant period at around 4s is due to sedimentary basin. Fig.12 shows relation between the numbers of displacement cycles and threshold





Fig.11 Observation Record at K-NET Furukawa (EW-comp) and its displacement response spectra



displacement for various period of SDOF system with 10% damping. Note that the SDOF system with 4s was shaken 3 times with displacement level of 40 cm and about 15 times with displacement level of 20 cm. Fig. 13 shows accumulated displacement for various period of SDOF system. The accumulated displacement of 4s system exceed 20m and that of 3s system and 5s system reach 17m.

Fig.14 shows comparison of the accumulated displacement for different site for the Tohoku earthquake and the past major earthquake records. For the same Tohoku earthquake, the value of Sumitomo site near Sendai station is 5.9m for 3s, and Aobayama site is 7.8m for 3s. The value of Takatori site for Kobe earthquake indicates 5.5m for 4s, and Kawaguchi-machi for the Niigata-Chuetsu earthquake shows 3.3m for 4s. At the same K-NET Furukawa station, the value for the 2008 Iwate-Miyagi earthquake is 13m for 4 s.

The information on numbers of displacement cycles and the accumulated displacement would be very important for discussing the damage of base-isolation devices, structural and non-structural element of building structures.



#### SITE SPECIFIC GROUND MOTION CHARACTERISTICS IN SENDAI AREA

### Observation network and geological condition

The author's Disaster Control Research Center (DCRC), Tohoku University, has strong motion network in Sendai City and valuable observation records were obtained during the Tohoku earthquake (Ohno and Motosaka, 2011). Seismometers are installed at ground floor (some are also at top floor) of public buildings (Ohno et. al, 2004). Site locations and geological information are shown in Fig. 15.

DCRC strong motion network observed not only 2011/3/11 main shock (9.0), but also 2011/3/09 (M7.3) foreshock and 2011/4/07(M7.1) after shock. Table 2 shows outline of the observed records for these earthquake events. During the main shock, the observation records were obtained at 14 stations. The range of PGA values is from 300cm/s/s to 840cm/s/s and that of PGV values is from 30cm/s to 80cm/s. PGA values were large at north part of Sendai city where the soft surface layers deposit on the hard rock. In these sites, dominant period of the ground motion is 0.4s to 0.7s.

Fig.16 shows pseudo velocity response spectra at east and west sides of Nagamachi-Rifu fault. The spectrum at No.27, locating near Sendai station, is commonly plotted in each sides as a reference. Spectra at west sides are equal to or relatively larger at short period (less than 1 second) than the No.27 spectrum, while the spectra at east sides are significantly larger than the No.27 spectrum, especially around 1 and 3 seconds. Such spatial difference may be due to the difference of geological structure: west side of the fault is terrace and east side is lowland (alluvial deposits) in Sendai.



Fig.15 Site locations of DCRC strong-motion network and surface geological information. (a) Obs. Site (b) surface topography (c) Quaternary layer bottom

		2011/4/7		2011/3/11		2011/3/9					
			PGA	PGV*	JMA	PGA	PGV**	JMA	PGA	PGV*	ЈМА
No	Sensor	Station	(cm/s <sup>2</sup> )	(cm/s)	Int.	(cm/s <sup>2</sup> )	(cm/s)	Int.	(cm/s <sup>2</sup> )	(cm/s)	Int.
2	ETNA	Rokugo Elementary School	311	42.1	5.7	Ν	lissing		Ν	lissing	
3	ETNA	Furujiro Elementary School	251	22.4	5.1	320	59.5	5.7	24	3.1	3.3
4	ETNA	Higashi Rokugo Elementary School		Removed	k	613	74.2	6.0	29	3.4	3.4
5	QDR	Daiichi Jr. High School	230	19.3	5.1	383	39.4	5.6	28	2.9	3.5
8	QDR	Shogen-Chuoh Elementary School	534	25.3	5.5	840	60.4	6.0	30	2.2	3.2
9	QDR	Matsumori Elementary School	767	75.5	6.2	822	85.7	6.4	46	4.2	3.7
10	QDR	Miyagi Prefecture Library 1F	279	18.0	5.0	407	62.7	5.6	20	2.4	3.2
11	QDR	Miyagi Prefecture Library 3F	N	Missing Missing		34	3.1	3.5			
12	QDR	Seiryo SecondarySchool 1F	N	lissing		Missing		19	3.5	3.3	
14	QDR	Tsurugaya Elementary School 1F	432	30.6	5.7	Missing		20	1.9	3.1	
16	QDR	Nakano Jr. High School 1F	Missing Missing		40	3.2	3.6				
18	QDR	Okino Elementary School 1F	360	31.8	5.6	512 77.6 6.2		37	3.5	3.5	
20	QDR	Minami Koizumi Elementary School	220	25.7	5.3	381	63.0	5.6	19	2.4	3.1
21	QDR	Nishitaga Jr. High School	186	16.4	5.0	400	45.1	5.5	23	3.0	3.4
22	QDR	Tomizawa Jr. High School	232	21.1	5.2	416	54.6	5.7	29	3.2	3.4
23	QDR	East Water Supply Center	472	37.3	5.8	613	75.4	6.1	30	2.6	3.3
24	QDR	Ryutaku-Ji	Removed		Missing		Missing				
25	QDR	Nagamachi Minami Community Center	264	29.5	5.5	494	69.3	6.0	59	6.0	4.0
26	QDR	Aoba Ward Office	318	21.9	5.2	Missing		24	3.2	3.3	
27	SSA-1	Sumitomo Seimei Bldg.	167	14.0	4.9	318	29.2	5.3	15	2.2	3.1
28	SMAC-MD	Tohoku Univ. 1F	N	lissing		333	53.7	5.6	35	4.4	3.7

	Table 2 Outline of earthq	uake records by DCRC	strong-motion network,	Tohoku University
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<sup>\*</sup> cut-off period of 10s, \*\* 50s



Fig.16 Comparison of pseudo velocity response spectra of DCRC strong-motion network

Fig.17 shows pseudo velocity response spectra (damping 5%) at Oroshimachi (No.23), Nagamachi (No.25) compared to Sumitomo Building near Sendai Station (No.27). The safety limit design spectrum is also shown in this figure. It is found that the period contents less than 1s are amplified by 3 times in Orishimachi, and that 1s period content at Nagamachi is 4 times amplified in NS direction compared to Sumitomo site and shows the very large response value of 300cm/s. But in EW direction the spectral amplification of 1s period content is not significant compared to NS direction. It is also noted that 3s period content is dominant at Nagamachi and amplified by 3 times in NS direction. This is effect of deep underground structure. Ground motion amplification characteristics in Aobayama are discussed in the later section.



Fig.17 Comparison of velocity response spectra due to different soil conditions

# Comparison of strong-motion characteristics between the 2011 and the past disastrous earthquake records at the same stations

The observation records at Sumitomo building near Sendai station by DCRC, Tohoku University, are very valuable to compare the records during the 1978 Miyagi-ken Oki earthquake at the same observation point. It is noted that the observation record was situated as engineering bedrock motion. The observed ground motions at this observation point during the 2011 earthquake are compared with

those due to 1978 Miyagi-ken Oki earthquake and also 2005 Miyagi-ken Oki earthquake.

Fig.18 shows the acceleration waveforms of horizontal two directions for 2011 Tohoku earthquake (M9.0), 1978 Miyagi-ken Oki earthquake (M7.4), and 2005 Miyagi-ken Oki earthquake (M7.2). In this figure, the maximum acceleration values are shown for the three earthquakes. Fig.19 shows the corresponding pseudo velocity spectra. Findings focused on the comparison of the 1978 earthquake from these figures are as follows.

- 1) The envelope shape of the 1978 earthquake is almost same as the first phase of the 2011 earthquake.
- 2) Period contents shorter than 1.5s of the 2011 earthquake is larger than those of 1978 earthquake, about 20% larger at 1s, and twice as large at 0.5s
- 3) The period contents around 3s of the 2011 earthquake is more than two times as large compared to those of 1978 earthquake.



Fig.18 Comparison of observed acceleration waveforms for 2011 Tohoku earthquake and two Miyagi-ken Oki earthquakes



Fig.19 Comparison of observed acceleration waveforms for 2011 Tohoku earthquake and two Miyagi-ken Oki earthquakes

# Site Specific Ground motion amplification in Aobayama hill and Resonance to Damaged Building

It is important to investigate the ground motion characteristics at Aobayama campus of Tohoku University where some 8- and 9-story buildings were severely damaged. Ground motion amplification compared to Sumitomo building was investigated for the 2011 earthquake and 1978 Miyagi-ken Oki earthquake. One of the damaged buildings is a 9-story building of Departments of Civil Engineering and Architecture, Faculty of Engineering, Tohoku University (THU Building). The building damage

feature and the related dynamic behavior are described later.

Fig.20 shows acceleration waveforms in the NS direction (Transverse direction) at 1<sup>st</sup> floor and top (9<sup>th</sup>) floor of THU building for the 2011 earthquake and the 1978 Miyagi-ken Oki earthquake.

Fig.21 shows the site specific ground motion spectral amplification in Aobayama for the two earthquakes, the 2001 Tohoku and the 1978 Miyagi-kun Oki earthquakes. Fig.22 shows the spectral amplification in the THU building for the two earthquakes. It is noted that the amplification characteristics of the 1<sup>st</sup> phase (phase A) and the 2<sup>nd</sup> phase (phase B) are different. In NS direction, the second phase is amplified by more than two times at around 1s period content at Aobayama campus compared to Sumitomo building near Sendai station. The amplification characteristics are almost the same as those of the 1978 Miyagi-ken Oki earthquake and the THU building was strongly amplified by resonance.



Fig.20 Comparison of acceleration records at 1<sup>st</sup> floor (left) and 9<sup>th</sup> floor (right)



Fig.21 Site specific ground motion spectral amplification in Aobayama



THU building for the 2011 and 1978 earthquakes

# DYNAMIC BEHAVIOR OF THU DAMAGED THU BUILDING DURING THE 2011 GREAT EAST JAPAN EARTHQUAKE

# Description of the damaged building and Observed Strong Motion

The THU building is a 9 story SRC (non-full web type) building and was constructed in 1969 and experienced 1978 Miyagi-ken Oki earthquake. The observed acceleration waveforms of the 3 components at 1<sup>st</sup> floor and 9<sup>th</sup> floor are shown in Fig.23. The configuration of the building and sensor locations are shown in Fig.24. In the building, SMAC-MD type seismometer is installed at 1<sup>st</sup> and 9<sup>th</sup> floor. From December, 2007, a continuous monitoring system (NetDAS/ MicroSMA) with sensors at

 $1^{st}$ ,  $5^{th}$  and  $9^{th}$  floor has been operated (Motosaka et al., 2008). The system enables to measure from microtremor to strong motion.



Fig.23 Observed records during the main shock

Fig.24 Configuration of THU Building and Sensor location

In the THU Building, long-term monitoring of dynamic characteristics has been performed by strong motion observation, forced vibration test, microtremor observation, and etc, for about 40years since the completion of the building in 1969 (Motosaka, et al., 2004, Motosaka, et al., 2011). After experience of the structural damage due to 1978/6/12 Miyagi-ken Oki earthquake (Shiga et al., 1981), seismic retrofit work was performed from autumn of 2000 to spring of 2001. It is noted that the seismic strength index, Is-value at the damaged 3 rd floor in the transverse direction increased from 0.53 to 0.84. Then, the building experienced the 2005 Miyagi-ken Oki earthquake, 2008 Iwate-Miyagi Nairiku earthquake, and 2008 Iwate Northern Coast earthquake, and so on.

Photo 1 shows the damaged THU Building. The building was heavily damaged at bottom of 4 corner columns. The severe crack of the side shear wall due to possibly partial uplifting was suggested at the level of third floor (Tsamba and Motosaka, 2011).



Photo 1 Damage feature of bottom of one of the damaged four corner columns of THU building

# Dynamic hysteretic behavior

To investigate the dynamic hysteretic characteristics, the acceleration records at 9th floor and 1st floor are doubly integrated and the relative displacement is calculated. The maximum relative displacement is 31cm in NS direction.

Fig.25 shows the dynamic hysteretic behavior based on force-displacement relation for the 16 time sections which are obtained from acceleration waveform at 9th floor and the relative displacement. Findings from this figure are as follows.

- 1) Starting from linear behavior at smaller amplitude level, hysteresis shows the inverse S shape slightly as recognized in section 5. Then shows linear behavior but the stiffness is reduced in section 6 compared to section 1.
- 2) Then, with increasing displacement, the hysteresis shows softening and hysteretic loop is recognized only for the larger displacement level. The hysteresis shows origin oriented behavior as recognized in (section 7).
- 3) Then decreasing the displacement, the hysteresis shows the characteristic inverse S shape in section 9. Although the amplitude decreases from Section 9 to Section 10, this hysteresis leads to stiffness reduction, which consistent with the dominant frequency reduction.
- 4) Then decreasing displacement gradually returns to linear behavior with the reduced stiffness in sections from 13 to 16 compared to section 6. The stiffness change is consistent with microtremor observation before and after the earthquake.



Fig.25 Relative displacement waveform and Hysteresis loops for each time section (NS-Direction)

#### **System Identification**

To investigate non-stationary dynamic characteristics due to structural non-linearity, the system identification technique using Kalman filter is used to determine natural frequency and damping factor as equivalent SDOF system (Takahashi et al., 2010).

Fig.26 show the result of the identified system parameters, natural frequency and damping factor for NS direction and EW direction, respectively. In these figures, the calculated relative displacement waveform is compared to that obtained from observed records. As for the identified system parameters, natural frequency and damping factor, the smoothed curves are shown in these figures. Findings from the system identification results are as follows.

- 1) In the transverse (NS) direction, dominant frequency decrease down to about 0.8 Hz with increasing amplitude and the dominant frequency is not changed even if amplitude is decreased.
- 2) The damping factor in the transverse direction increases with increasing the amplitude at time section 7 in Fig.25 At the time sections with the inverse S type hysteresis, damping factor is decreasing with increasing the amplitude.
- 3) In the longitudinal (EW) direction, dominant frequency decrease down to about 0.9 Hz but return to about 1 Hz when amplitude level becomes small.
- 4) The smoothed damping factor in the longitudinal direction seems to be lager compared to the Transverse direction. This may be due to difference of hysteretic energy consumption.



Fig.26 Identified damping and Natural Frequency

#### Investigation of uplifting vibration

In case of structural vibration with partial uplift, odd number higher harmonics are induced in the horizontal directions and even number higher harmonics are induced in the vertical direction (Motosaka and Nagano, 1993).

Fig.27 shows non-stationary characteristics expressed by wavelet coefficients of the 9th floor's acceleration waveform in the transverse direction for 10 sec time sections including the peak value. Fig.28 (a) shows the wavelet coefficients of 9th floor for the vertical direction. Fig.28 (b) is the wavelet coefficients of 1st floor for the vertical direction. Findings from these figures are as follows.

1) The dominant frequency corresponding to the two times frequency (about 2Hz) of the vertical direction is clearly seen at large amplitude range corresponding to the time sections from 7 to 9.

But the dominant frequency is not seen at the 1st floor.

2) This suggests the occurrence of partial uplift of the damaged building at the 3rd floor.



Fig.27 Acceleration waveform at 9F and wavelet transform

Fig.28 Wavelet transforms of acceleration in the vertical direction at (a) 9F and (b) 1F

# LONG TERM MONITORING OF DYNAMIC CHARACTRISTCS OF THU BUILDING

In the building, earthquake observation has been performed since completion in 1969 and microtremor observations also have been performed. The forced vibration test was performed before and after the retrofit work in 2000. The amplitude dependent dynamic characteristics have been investigated (Motosaka et al., 2004).

Table 3 shows change of 1<sup>st</sup> natural frequency of the building based on not only the earthquake records of main shock and foreshock and together with micotremor records before and after the earthquake events. It is noted that the natural frequency of the two directions due to microtremor observations were the same (1.61Hz) before and after the 3/9 foreshock and also the same (1.26Hz) in the two directions during the foreshock. But during the main shock, reduction of the natural

 Table 3
 Change of fundamental frequency

Date	Event name	Natural frequency (HZ)			
		NS	EW		
2011/3/9	Microtremor	1.61	1.61		
2011/3/9	Foreshock (Sanriku Oki)	1.26	1.26		
2011/3/11	Microtremor	1.61	1.61		
2011/2/11	Main shock (Phase A)	1.05	1.05		
2011/3/11	Main shock (Phase B)	0.78	0.88		
2011/3/19	Microtremor	1.17	1.37		
2011/3/19	Aftershock (Ibaraki-Ken)	0.93	1.16		
2011/5/3	Microtremor	1.17	1.37		
2011/5/31	Microtremor	1.37	1.48		

frequency is remarkable in NS direction compared to EW direction, which is consistent with the damage feature of the building. The frequency reduction is large in the second phase (phase B). The stiffness reduced up to 23% in NS direction and 37% in EW direction compared to the stiffness due to microtremor before the main shock. The stiffness in the microtremor level was reduced to 53% in NS direction and 72% in EW direction. It is noted that the THU building was temporary repaired at the damaged 3<sup>rd</sup> floor in May, which lead to the natural frequency increase from 1.17Hz to 1.37Hz in NS direction and 1.37Hz to 1.48Hz in EW direction. But the microtremor observation at May 3 (before the repair work) shows no natural frequency change from March 19, even if the building was shaken by the large aftershock on April 7 and April 11.

Table 4 shows the maximum acceleration list of major earthquake observation records at THU building. Fig.29 shows the relation between the deflection angle and the natural period for both two directions. In the figure, the different symbols are used for the 4 terms, namely, Term 1: From

completion to 1978 earthquake, Term 2; After 1978 earthquake to retrofit work in 2000, Term 3: After retrofit work to 2011 Tohoku Earthquake, Term4: After Tohoku earthquake. Furthermore Term 3 and Term4 are divided into two terms.

Date	Magnitude	1 Floor (max.acc (cm/s/s) NS EW		9 Floor (max.acc (cm/s/s)		Are a name		
				NS EW				
1978/2/20	6.7	170	114	421	298	Miyagi-Ken Oki		
1978/6/12	7.4	258	203	1040	523	Miyagi-Ken Ok	i	
1998/9/15	5.2	138	451	190	379	Miyagi-Ken South	ern	
2003/5/26	7.1			231	264	Miyagi-Ken Ok	i	
2003/7/26	6.2	33	27	98	102	Miyagi-Ken North	ern	
2003/9/26	8.0			29	22	Tokachi Oki		
2005/8/16	7.2	87	81	329	287	Miyagi-Ken Oki		
2008/5/8	7.0	19	22	261	226	Ibaraki-ken Oki		
2008/6/14	7.2	88	70	392	293	Iwate-Miyagi inland		
2008/7/24	6.8	59	77	275	367	Iwate-Ken North Coast		
2011/3/9	7.2	37	34	171	89	Sanriku Oki		
2011/2/11	0.0	207	216	594	617	Off Desifie Coast Tabeler	(Phase A)	
2011/3/11	9.0	333	330	908	728	On Pacific Coast Tonoku	(Phase B)	
2011/3/19	6.1	15	18	34	56	Ibaraki-ken Northern		
2011/4/11	7.0	72	70	141	172	Fukushima-Ken Southern		
2011/4/12	6.4	24	28	43	84	Fukushima-Ken Oki		
2011/4/23	5.4	17	27	23	57	Fukushima-ken Oki		
2011/7/10	7.1	21	18	95	58	Sanriku Oki		
2011/7/23	6.5	10	11	60	42	Miyagi-Ken Oki		
2011/7/25	6.2	48	62	106	99	Ibaraki-ken Northern		
2011/7/31	6.4	36	31	70	45	Fukushima-ken Oki		

 Table 4
 Maximum acceleration list of major earthquake observation records at THU building





Findings from this figure are as follows.

 The amplitude level of the first phase (phase A) is smaller than that of the 1978 earthquake in NS direction but the amplitude level of the second phase (phase B) became larger compared to 1978 earthquake in the both directions.

- 2) The change of the natural period due to the 2011 Tohoku Earthquake is lager in NS direction compared to EW direction, which is consistent with the damage feature.
- 3) It is confirmed through the continuous observation that the dominant period at mictotremor level is not changed if the deflection level is smaller than the experienced maximum deflection.

# DAMAGE FEATURE OF THE DAMAGED AREA DURING THE 1978 MIYAGI-KEN OKI EARTHQUAKE

As above mentioned, Oroshimachi area, the damaged area during 1978Miyagi-ken Oki earthquake , which was located in the alluvial plain, was severely shaken during the 2011 earthquake compared to Sumitomo site near Sendai station located on a diluvium plateau (refer to Fig. 15).

In Oroshimachi area, only two buildings among 338 buildings were collapsed during the 3/11 main shock but number of collapsed buildings was increased to 5 due to 4/7 severe after shock.

Considering many buildings were heavily damaged during the 1978 earthquake, the buildings in this area became strong due to effectiveness of the new building code and seismic reinforcement/retrofit. Photo 2 shows the collapsed two buildings during the main shock. Both were constructed in 1969 and experienced the 1978 earthquake. Estimation of



Photo 2 Collapsed two buildings in Oroshimachi, Wakabayashi-ku, Sendai

accumulated damage is important. Both the collapsed buildings are damaged in NS direction, which is consistent with the ground motion characteristics in Fig.30. The author confirmed through witness by two survived persons who were at the 1st floor of the left building in Photo 2 that the building was not collapsed during the first phase (phase A) but collapsed during the second phase (phase B).



Fig.30 Comparison of observation record in the two horizontal directions at Oroshimachi and response spectra

# SPECFIC STRUCTRAL DAMAGE

#### **Damage of Pile foundation buildings**

The authors inspected the damaged pile foundation buildings during the 1978 earthquake immediately after the 2011 Tohoku earthquake. It was reported that 7 piles are inspected and two among 7 piles were repaired but not reinforced at that time (Shiga, 1980). Photo 3 shows the 14-story SRC building for condominium, one of L shaped two buildings, connected by exp. joint each other, was inclined by 1/56 due to main shock and extended the inclination up to 1/45. Fig.31 shows the result of subsidence measurement at 11 points. The measured values in meter (shown as blue) are

relative elevation from the reference point. In the inclined right building, the level differences are 31cm for 14m arm length in the south (No.10 and No.11) and center (No.4 and No.3), which are consistent with the inclination angle of 1/45. This subsidence measurement suggests the pile failure of east side due to dominant ground motion in the NS direction.



Photo 3. Inclined 14-story SRC pile foundation building; (left) Overall view, (right) Opening of expansion point part



Photo 4. Inclined 4-story RC pile foundation building; (left) Overall view (right) Subsided bottom part of the south east corner column

### **Ceiling Board Damage**

Many ceiling board drop accidents were occurred in the large span structures like culture hall and large store. The ceiling board damage killed 4 persons during the Tohoku earthquake.

Photo 5 show the damage feature of suspended ceiling board drop at a culture center hall in Sendai, which the author inspected and investigated vibration characteristics based on microtremor observation. Fig.32 shows the Fourier spectra at roof of the building. It is noted that the vertical motion is induced at the roof due to horizontal input motion. In the design of the ceiling board, the ceiling board system would be taken into the induced vertical motion as well as enough clearance for horizontal motion. Judging from the damage feature, the drop seems to be caused by the lack of strength to the vertical motion and the unexpected collision between hanging bold and equipments such as air conditioning duct. The seismic standard for vertical load of the ceiling board system would be reconsidered especially for large span structures and also for the many numbers of displacement cycles due to ground motion with long duration and also due to accompanied many aftershocks. It is also important to recognize that the response of the ceiling board is affected by the vibration characteristics of earthquake supporting soil and structure including roof slab. If resonance phenomena occur, applied earthquake force would significantly large. For a lager ceiling board with area more than  $1,000m^2$ , floor response spectral method would be needed. It is also important to consider that ceiling board system is recognized as the bending board supported by springs and vibration energy should not be propagate horizontally (Motosaka, 2006).



Fig.31 Measurement of subsidence of the damaged building

A 4-story RC building with pile foundation (Photo 4) was inclined by 1/30 due to pile damage. No damage was recognized in the superstructure. The building was constructed in 1983 after execution of the new building code. It was recognized the surrounding soil was liquefied.



Photo 5 Damage feature of ceiling board drop at a culture center hall in Sendai



Fig.32 Dominant frequencies based on microtremor measurement at the center of roof at the damaged culture center hall in Sendai

### CONCLUSIONS

In this paper, ground motion characteristics and the related building damage during the 2011 Great East Japan earthquake (M9.0) are described. Buildings with appropriate seismic reinforcement/retrofit were mostly escaped from damage, which indicates effectiveness of the seismic reinforcement/retrofit. But the structural damage caused by ground motion amplification was recognized. The following learning and lessons from the 2011 earthquake are addressed for stronger earthquake countermeasures of urban and building structures.

- 1) Necessity of the seismic microzoning considering ground motion difference due to geological conditions,
- 2) Necessity of appropriate seismic indices corresponding to objective building damage,
- 3) Reconsideration of the setting place / setting method of the seismometer,
- 4) Necessity to evaluate the safety of structural elements for number of displacement cycles due to the long-duration earthquake and repetition by many aftershocks,
- 5) Consideration of non-stationary of ground motion the nonlinearity of the building for the huge earthquake,
- 6) Total balance of structural element, non-structural elements, and equipments, and also balance of foundation and superstructure for synthetic seismic performance of the whole building,
- 7) Evaluation of residue performance of the buildings damaged by past earthquakes and this earthquake.

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