Preliminary Investigation of the Pile-foundation Buildings Leaned By Damage in the 2011 off the Pacific Coast of Tohoku Earthquake

K. Mitsuji
Yamagata Univeristy, Japan

Z. Liu, S. Ohno & M. Motosaka
International Research Institute of Disaster Science, Tohoku University, Japan

SUMMARY:
High-rise buildings supported by pile foundations were damaged by the main shock of 2011/03/11, and one of them leaned to the south as a result of the damage due to strong shaking. Damage investigation and microtremor measurements were carried out in the damaged buildings. Inclination angle of the building is estimated about 1/50. Results made it clear that the first natural frequency of the building appears around 1Hz and dynamic characteristics of the buildings are different in higher mode of TR and LN directions, and rocking motion in TR direction is predominant.

Keywords: The 2011 off the pacific coast of Tohoku earthquake, microtremor measurement, pile-foundation

1. INTRODUCTION

In the 2011 off the Pacific coast of Tohoku earthquake, lots of buildings have been suffered by the damage caused by strong shaking. As one of the severe damage of buildings, several leaned buildings have been observed. Some of them are supposed to have been caused by strong shaking of main shock, and others are by liquefaction where the site condition is soft soil ground.

The authors are in charge of damage investigation of the leaned buildings in Sendai city. One of them is the housing complex consisting of two high-rise buildings (building A and B) located in the eastern part of Sendai city. The damaged site is called by “STM” in the followings. Each building is 14-storey SRC structure and supported by pile groups. The building A is considered to be leaned by damage of pile foundation and shows distinct inclination from the outside appearance.

The buildings had been also damaged by the 1978 Miyagi earthquake (Mj=7.4), and then two sets of group piles were heavily damaged. According to the damage investigation report of the 1978 Miyagi earthquake, concrete body of pile were crushed, some steel bars indicated buckling and shear cracks were observed at the pile head in some pile groups (Shiga 1980, Sugimura and Oh-oka 1981). Those buildings were restored after the 1978 Miyagi earthquake, and residents have kept living in those buildings until the earthquake of March 11.

Microtremor measurements were carried out as preliminary investigation for those buildings. The authors also investigated damage survey of super-structure, foundation and surface ground, and inclination measurement of super-structure and foundation. In this paper, the state of the damage is reported and the results of microtremor measurements are presented.

2. LOCATION OF THE SITE

Shown in the left figure of Figure 1, Sendai city is located on the east coast of the Pacific Ocean, about 300km north from Tokyo. There is about 130km between Sendai city and the epicentre of the main shock of 2011/03/11. Location of the damaged site STM is shown in the right figure of Figure 1. Earthquakes have been observed at the site of MYG013 (K-NET), and enormous number of earthquakes after the main shock were also recorded at this site. The damaged site of STM is located near MYG013 of which distance is about 3km as shown in the right figure of Figure 1.

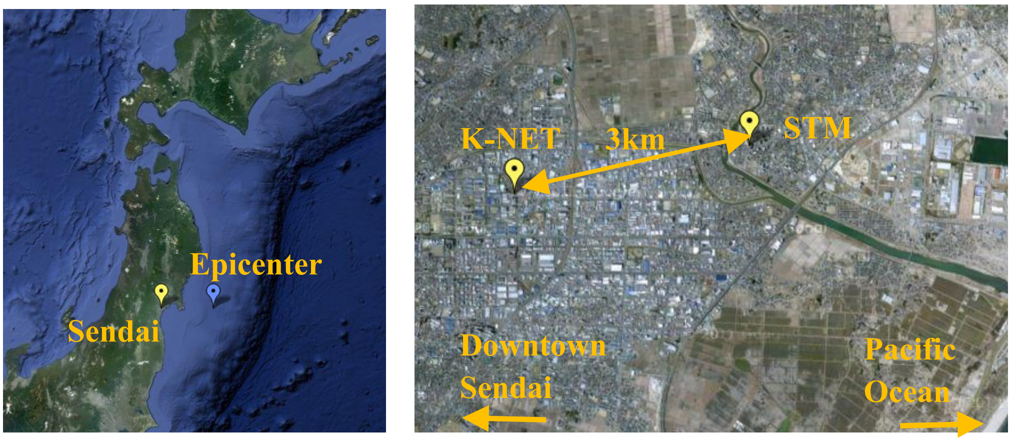

Figure 1. Location of the site
Left: Sendai city and the epicenter of the main shock of 2011/03/11, Right: K-NET MYG013 observation site and the damaged site of STM
(Left: Image@2012 TerraMetrics, Map Data@2012 ZENRIN, Right: Image@2012 Ones Spot Image, Digital Earth Technology, Digital Globe, GeoEye, Map Data@2012 ZENRIN

Velocity response spectra (damping coefficient h=5%) of the main shock of 2011/03/11 and aftershock of 2011/04/07 at MYG013 are shown in Figure 2. The earthquake of 2011/04/07 is recognized as the largest one among the aftershocks continuously occurred. While response in NS direction is predominant in the main shock, response in EW direction becomes much larger than NS direction in the aftershock. It is also distinctive that response spectra of the aftershock indicate two peaks.

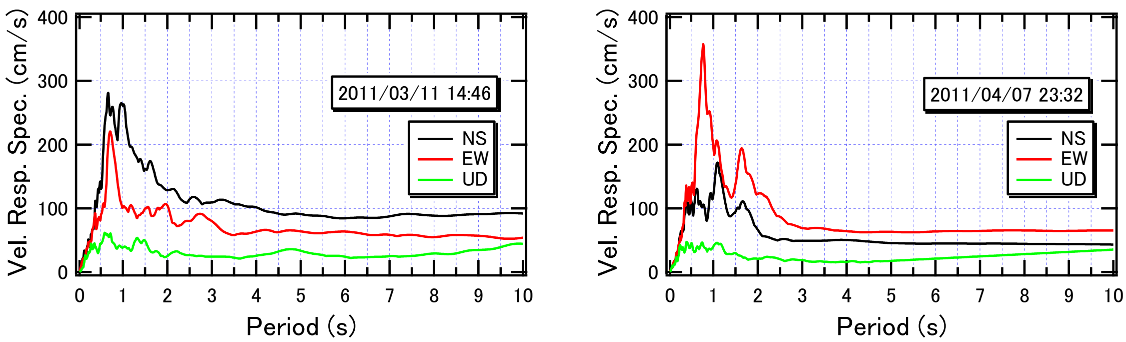

Figure 2. Velocity response spectra (h=5%) at MYG013
Left: 2011/03/11 main shock, Right: 2011/04/07 aftershock

3. SITE CONDITION AND OUTLINE OF THE LEANED BUILDING

Standard penetration tests had been conducted at the four places in the site of STM. One of the N-value distributions along depth is shown in Figure 3. For comparison, the N-value distribution of MYG013 is also illustrated in the right figure of Figure 3.
From GL-25m up to the surface, alluvial sand and silt layers are reciprocally accumulated at the site of STM. On the other hand, the thickness of the alluvial deposit is only 15m at MYG013. Based on the N-value distribution, shear wave velocity distribution along the depth of the site of STM is estimated by use of the empirical relationship (Ohta and Goto 1976). Shear wave velocity distribution of MYG013 is open to the public at the K-NET web site. Shear wave velocity distributions of both sites are shown in Figure 4. It should be noted that depth that the solid gravel or rock layer appears is different in both sites.

Amplitude characteristics of both sites are compared in Figure 5. Amplitude ratio is estimated by theoretical transfer function obtained from linear one-dimensional multi-reflection analysis. Peak around 5Hz meets good agreement with in both sites. The effect of the soft surface layer above from GL-15m seems to appear in the peak around 2Hz at the site of STM.

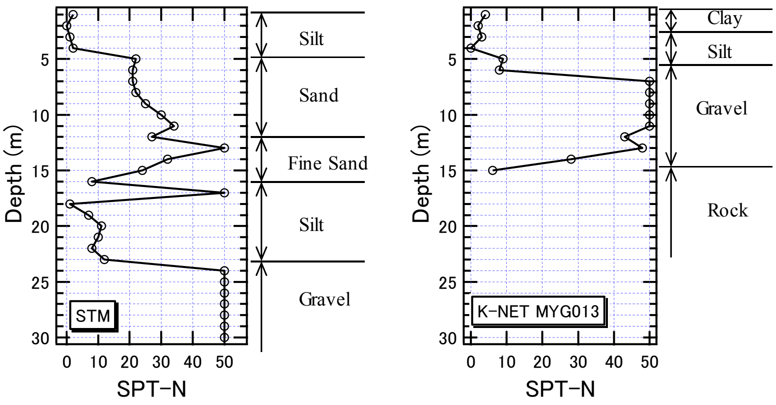

Figure 3. SPT-N distribution of the site of STM and MYG013 (K-NET)

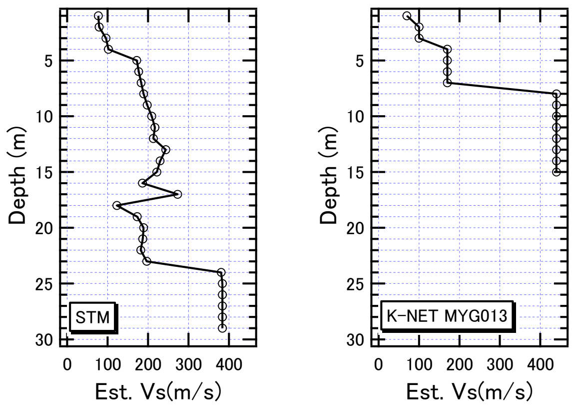

Figure 4. Estimated Shear wave velocity distribution of the site of STM and MYG013 (K-NET)

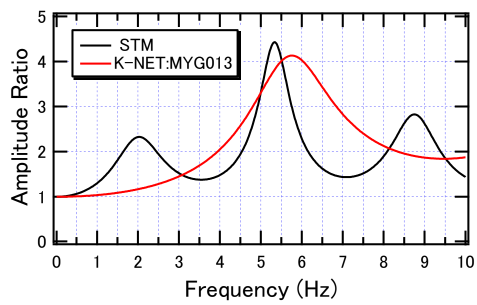

Figure 5. Theoretical amplitude characteristics of the site of STM and MYG013 (K-NET)

Schematic plan and section of the housing complex of STM are illustrated in Figure 6. Photos of the exterior appearance of the buildings are shown in Photo 1. Housing complex STM consists of the two buildings. Both of them are SRC 14-storey buildings supported by pile foundations. Pile foundations are organized by PC piles with autoclave technology of which diameters are 600mm and 500mm. Length of pile is 24m (two 12m-piles are connected by welding). Height of both buildings is about 40m. Construction of the buildings have completed in 1976. After the completion, the buildings suffered severe shaking and were heavily damaged by 1978 Miyagi off shore earthquake.

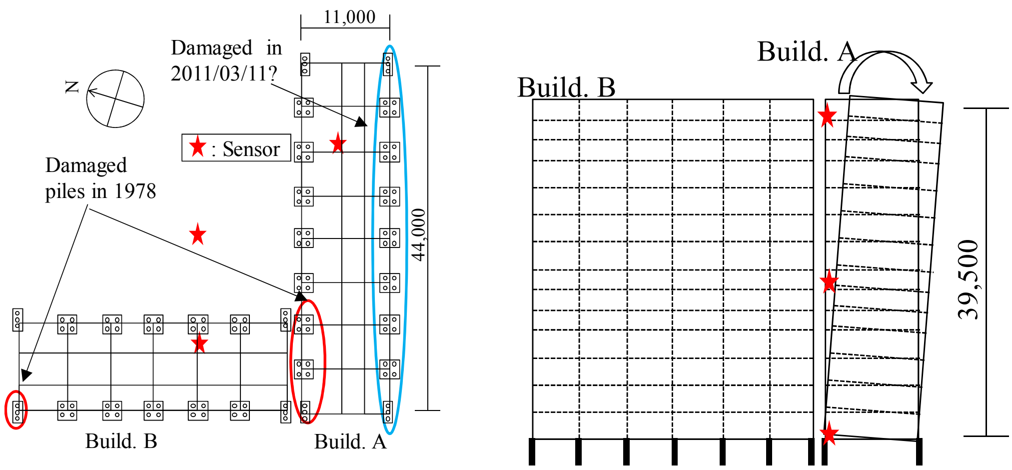

Figure 6. Schematic plan and section of the housing complex of STM

Photo 1. Exterior appearance of the damaged housing complex

The buildings had been also damaged by the 1978 Miyagi off shore earthquake, and then some group piles that are marked by red circles in the left figure of Figure 6 were heavily damaged (Shiga 1980, Sugimura and Oh-oka 1981). According to the damage investigation report of the 1978 Miyagi off shore earthquake, concrete body of pile were crushed, some steel bars indicated buckling and shear cracks were observed at the pile head in some group piles as shown in Figure 7. Those buildings were restored after the 1978 Miyagi earthquake, and residents have kept living in those buildings before the earthquake of March 11 occurred.

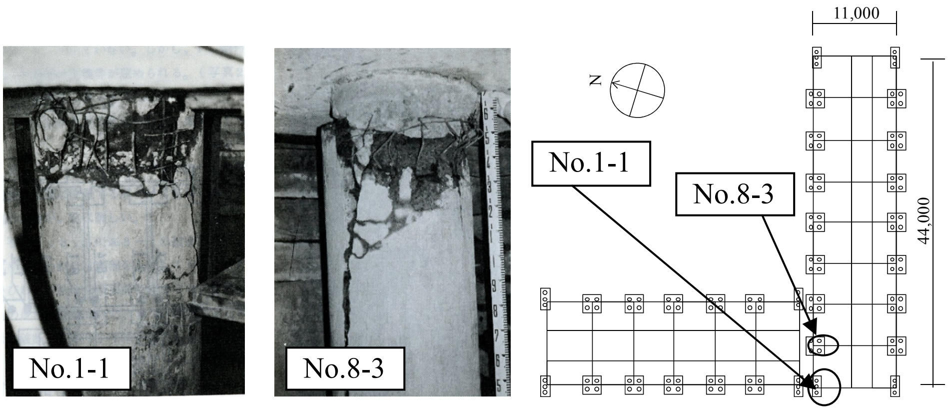

Figure 7. Damage at pile heads in 1978 Miyagi earthquake (After Sugimura and Oh-oka 1981)

After the main shock of 2011/03/11, the buildings have been heavily damaged by strong shaking again. Building A and B were connected by expansion joint that were demolished by dynamic behavior of the buildings. Building A and B forms L-shape plan shown in Figure 6. Dynamic behaviour of each building is completely different, because strong axes cross each other. Most characteristic damage is that building A indicates remarkable inclination in NS direction as shown in the photo in Figure 8. Inclination of the building A creates the large gap between the building A and B. Inclination of the building was not observed in the damage by the 1978 earthquake even though severe damage was observed at the pile head in some pile groups. Gap had been measured by the municipal office of Sendai city after the main shock. The authors also measured gap between the buildings at all the floors in October 2011. Measured gaps at each floor between the buildings are shown in the left figure of Figure 8. The figure clearly indicated gap at each floor increased after the aftershock of 04/07. Gap between two buildings was 1,045mm at the roof floor and 265mm at the first floor. Angle of the inclination of the building A was estimated about 1/60 to 1/40. Outstanding inclination as building A was not recognized in the building B.

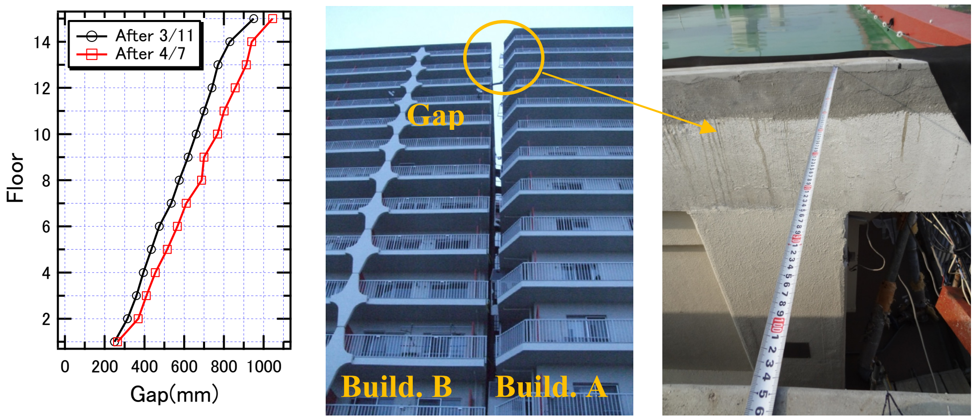

Figure 8. Measured gap between the building A and B

4. DAMAGE INVESTIGATION AFTER THE MARCH 11TH EARTHQUAKE

The state of the damaged buildings is shown in Photo 2, 3 and 4. The left photo of Photo 2 is the gap between the building A and B at the 14th floor that is the largest gap of over 1000mm. Large width shear cracks are observed in almost all the non-structural walls facing corridors and balconies as shown in the right photo of Photo 2. This phenomenon was also observed in the damage by the 1978 earthquake.

The left photo of Photo 3 shows damage around the foundation of the north side of the building A. After the main shock of 2011/03/11, building A leans to the south. Difference in the level shown in the right photo of Photo 3 is considered to have been generated by the uplifting behaviour induced by the inclination of the building A. This difference in the level is about 14cm, and settlement around the building is about 20 to 25cm. The left photo of Photo 3 shows damage of non-structural wall of the south facade at the first floor of the building A. Severe shear crack is observed as well as the upper floors. Large gap running on the ground is recognized in the right photo of Photo 3. This large gap is also considered to have induced by the uplifting behaviour of the building A to the south.
As mentioned above, the building A leans to the south. The inclination is considered to be the effect of the uplifting behaviour caused by heavy damage at the pile head of some pile groups. Damage around the foundation of the building A indicates the inclination of the building. Moreover, the angle of the inclination of each housing unit at all the floors was measured by the electrical spirit level. Results show that the angle of the inclination of the housing unit floor is about 1/40 to 1/60 corresponding to the estimated angle of the gap between the buildings.
Demolition works of both buildings have started after 2012. At the middle of April 2012, the 14th floor of the building A were completely disappeared. The 13th and 14th floors of the building B remained just the structural elements as SRC columns, girders, beams and earthquake resisting walls without interior elements in each dwelling units. Shear cracks on the earthquake resisting walls have been observed at each dwelling units of the 13th and 14th floors of the building B shown in Photo 4. Shear cracks were measured in each dwelling unit at the two floors and width of the largest one was

about 1mm. On the other hand, narrow shear cracks on the earthquake resisting walls have been observed at one dwelling unit of the 11th floor of the building A. Observed shear cracks on earthquake resisting walls of the building B indicate that input energy by shaking to the building B was greater than the building A. This meets good agreement with the fact that the input energy of the earthquake to the leaned building A is considered to have been absorbed at the severely damaged pile heads.

Photo 2. Gap between the buildings at the 14th floor and damaged non-structural wall

Photo 3. Damage around the foundation and damaged non-structural wall at the first floor

Photo 4. Shear cracks on the earthquake resisting walls at the 14th floor of the building B

5. RESULTS OF MICROTREMOR MEAURMENT

As mentioned before, it is expected that pile heads were destructed in some pile groups of the building A. Damage at the pile can be considered to have made the building lean to the south. Detailed investigation under the foundation such as excavation of the ground will be conducted in the near future. As preliminary investigation before the detailed investigation, microtremor measurements were conducted in the building A and B. The information obtained from microtremor measurements is expected to understand the damage mechanism and help the demolition work.

Microtremor measurements were carried on 2011/10/07. The weather condition was clear and partly cloudy with strong wind in mainly NS direction. The objective of the microtremor measurement is to obtaining the vibration characteristics of the leaned building. Direct measutrment between 14F and 1F was preferable. However, length of the cables was limited. Therefore, the measurements were split into the two stages of 7F/1F and 14F/7F. In each measurement, two observation points such as 7F/1F and 14F/7F were distributed in the building. Position of each sensor is shown in Figure 6 by star mark. Each observation point has two sensors for horizontal direction (EW and NS) and one sensor for vertical direction (UD). Sensor is the electromagnetic velocity-meter (VSE-15D, Tokyo Sokushin Co.Ltd.). The duration of one measurement is 10 minutes and sampling frequency is 100Hz. The time window of which length is 40.96 (s) is applied to the observed velocity record for the data processing. The time window is moving toward the end of the record overlapping with the previous time window by half length of 20.48(s). The Vibration characteristics of the buildings are estimated by taking the average of the results from all the trials in the followings. The number of trial is 27.

Amplitude ratios between 7F/1F and 14F/7F of the building A are shown in Figure 9. In case of the building A, transverse direction (TR) of the building corresponds to NS direction, longitudinal direction (LN) to EW direction as indicated in Figure 6. The peak considered the first natural frequency of the building appears around 1Hz in both of TR and LN direction in case of 7F/1F. LN direction has the second distinctive peak around 4Hz. The range of the peak around the first natural frequency tends to be wide in the two horizontal directions. It should be noted that clear peak of 1.5Hz is observed in UD direction. The day when the microtremor measurements conducted was quite windy. Especially, the wind in NS direction corresponding to TR direction of the building was severely strong. Therefore, the results suggest that the effect of the strong wind might appear in the vibration characteristics of the building. In the results of 14F/7F, a few peaks can be observed over 2Hz. While TR direction has three peaks between 2Hz and 3Hz, the peak around 2Hz in LN direction looks ambiguous.

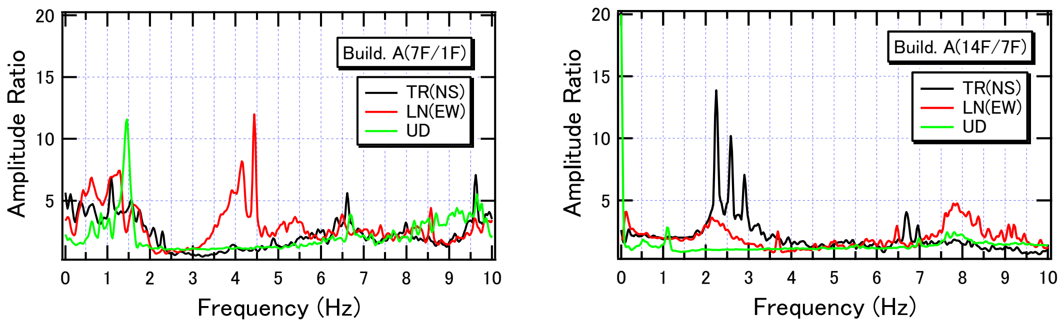

Figure 9. Amplitude ratios between 7F /1F and 14F/7F of the building A

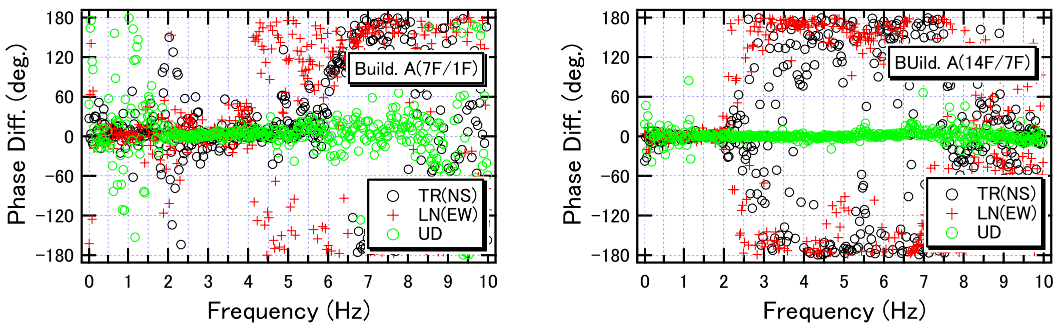

Figure 10. Phase differences between 7F /1F and 14F/7F of the building A

Figure 10 shows phase differences of 7F/1F and 14F/7F. If the peak around 1Hz of 7F/1F is considered the first natural frequency, behavior of phase difference looks odd. The behavior of phase difference around 1Hz in 7F/1F may indicates the effect of strong wind. Strange behavior around 1Hz such as 7F/1F cannot be observed in the result of 14F/7F.

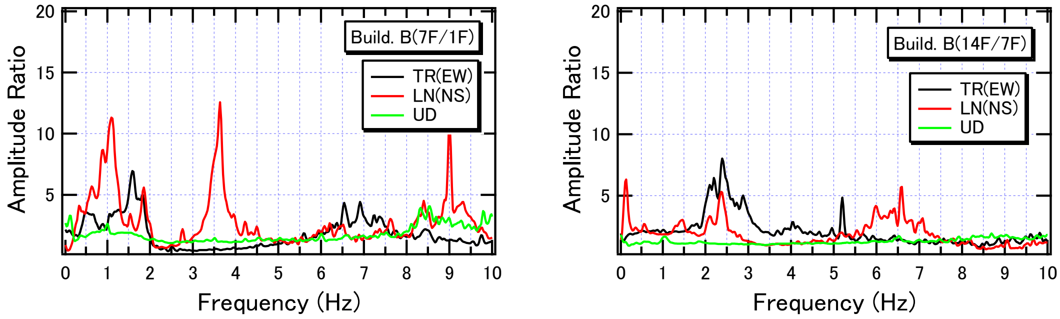

Figure 11. Amplitude ratios between 7F /1F and 14F/7F of the building B

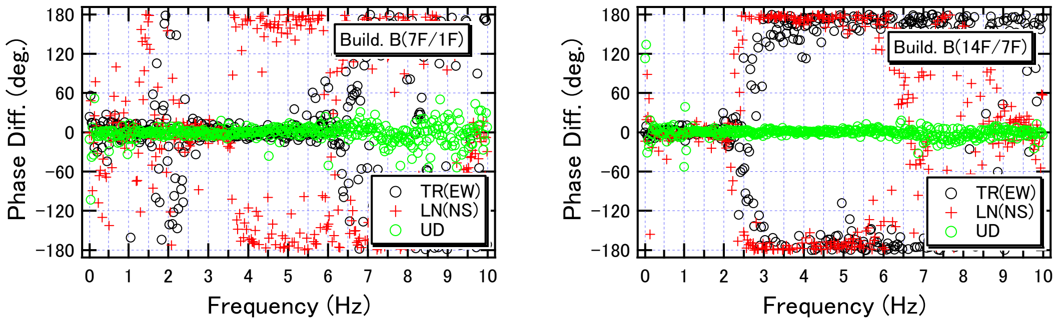

Figure 12. Phase differences between 7F /1F and 14F/7F of the building B

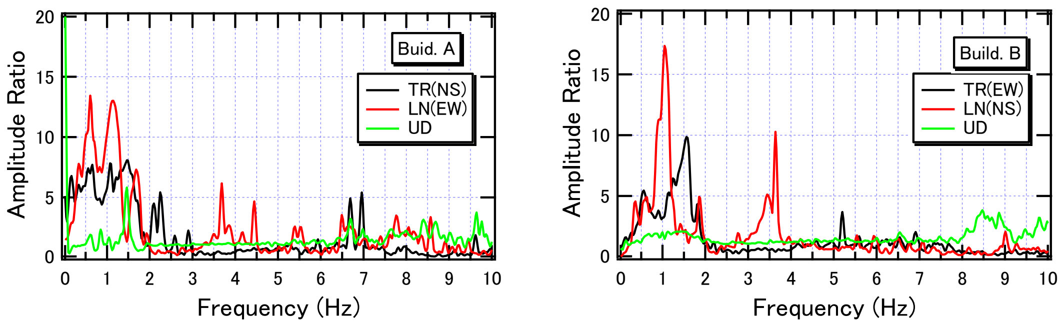

Figure 13. Amplitude ratios between 14F/1F of the building A and B

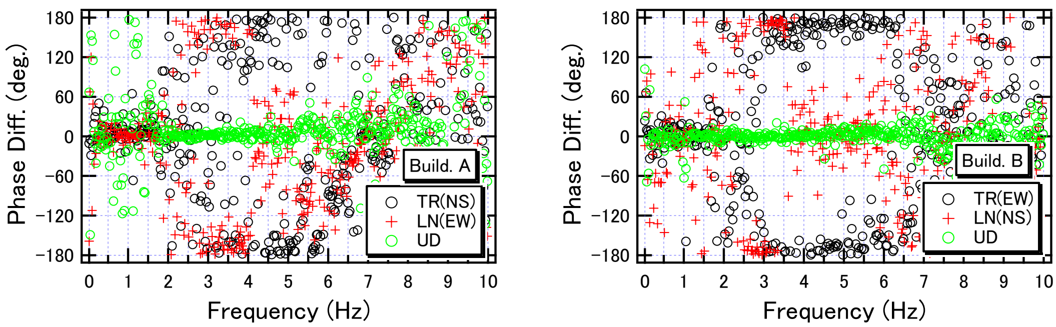

Figure 14. Phase differences between 14F/1F of the building A and B

Amplitude ratios between 7F/1F and 14F/7F of the building B are shown in Figure 11. In case of the building B, It should be noted that transverse direction (TR) of the building corresponds to EW direction, longitudinal direction (LN) to NS direction as indicated in Figure 6. Those are opposite to the building A. The peak considered the first natural frequency of the building appears clearly 1.5Hz and 1Hz in TR and LN direction in case of 7F/1F, respectively. In LN direction, the second peak can be clearly found between 3Hz and 4Hz. Amplitude ratios of 14F/7F indicate that the first peaks appear around 2Hz in horizontal directions as well as the building A.

Phase differences around 2Hz in case of 7F/1F may indicate the effect of strong wind similarly as the building A. The effect of strong wind seems not to be observed in phase difference of 14F/7F.

Through Figure 9 to Figure 12, the first natural frequency of the building seems to be between 1Hz and 2Hz. The second peak can be observed between 3Hz and 4Hz only in LN direction. Second peak cannot be recognized in TR direction. Based on the results, it can be assumed that dynamic behaviour in LN direction tends elastic on the other hand, rigid behaviour in TR direction. Phase difference around the first natural frequency in 7F/1F indicates the effect of strong wind. The reason that the effect of strong wind cannot be recognized in 14F/7F can be due to the assumption that the transfer function between 14F and 7F is isolated from the effect of the dynamic behaviour of the ground.

Combining transfer function of 7F/1F and 14/7F, transfer functions between 14F/1F are estimated. Estimated amplitude ratios and phase differences between 14F/1F are shown in Figure 13 and Figure 14, respectively. Left figures are the results of the building A, and right figures are those of the building B. Amplitude ratios of the building A indicate the peaks around 1Hz having the relatively wide range. Especially, the peak in LN direction split into two peaks around 1Hz. The result suggests torsional motion. LN direction has the second peak between 3Hz and 4Hz. Amplitude ratios of the building B have clear peaks around 1.5Hz in TR and 1Hz in LN direction. LN direction has the second peak between 3Hz and 4Hz as well as the building A. Namely, TR direction has one peak around 1 to 1.5Hz, on the other hand, LN direction has two peaks around 1 to 1.5Hz and between 3Hz and 4Hz. This result can be interpreted as the fact that dynamic behavior of TR direction is close to that of rigid medium with rocking motion at the bottom. Phase difference in UD direction of the building A fluctuates around 1Hz. Broaden wide range of the first natural frequency and phase difference behavior around 1Hz in the building A can be considered to be relevant to the effect of strong wind.

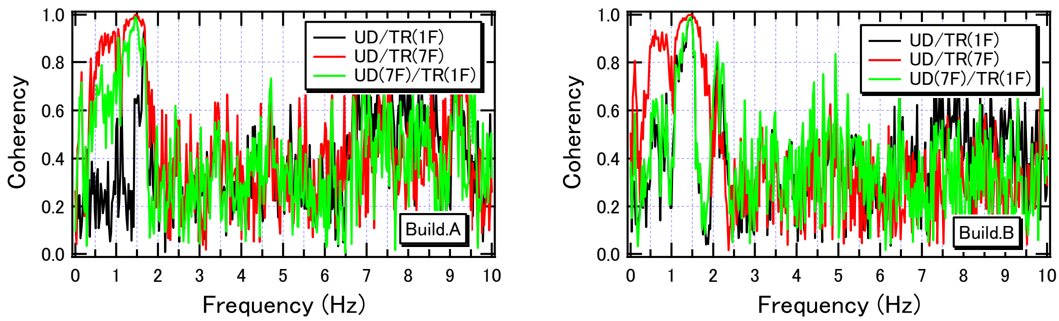

Figure 15. Coherency function between UD and TR motion of 7F /1F

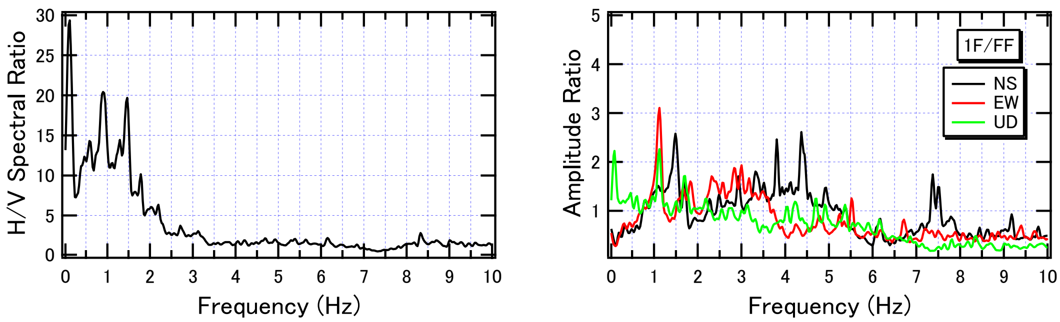

Figure 16. H/V spectrum of the ground and amplitude ratios between 1F and the free field

To examine the spectral peak in 1.5Hz observed in amplitude ratios of UD direction of the buildings in Figure 9, correlation relations between UD and TR motions are investigated. Coherency functions representing correlation relations between 7F and 1F are shown in Figure 15. Coherency functions are obtained for three cases as UD and TR of 7F, UD and TR of 1F, and UD of 7F and TR of 1F.

Results in Figure 15 indicate high coherency from 0.5 to 1.5 Hz at 7F corresponding to the first natural frequency of the building. At the first floor of the building A, coherency function of UD and TR indicates low correlation. High coherency function from 0.5 to 1.5 Hz can be considered to be relevant to rocking motion of the building in TR direction. This phenomenon is observed in the results of both buildings.
Measurement to obtain the dynamic characteristics of the ground was also carried out. Results of the ground are expressed by “FF (Free field)”. H/V spectrum and amplitude ratios between 1F and FF are shown in Figure 16. The peak of H/V spectrum exists around 1Hz. This result doesn’t meet good agreement with the results from the theoretical transfer function using estimated shear wave velocity data shown in Figure 5. Peaks are also observed around the natural frequencies of the buildings in amplitude ratios of 1F/FF.

6. CONCLUSIONS

In this paper, microtremor measurements of the leaned building and the relevant investigation were presented as the preliminary investigation of the damaged buildings by the 2011 off the Pacific coast of Tohoku earthquake. The results are summarized as follows.

1) Uplifting behaviour around the foundation of the leaned building was observed. Broaden shear cracks on non-structural walls were observed at almost all the floor. This type of damage is the same as the case of 1978 earthquake.
2) Gap between the building A and B generated by the main shock of 2011/03/11 indicates over 1,000mm at the roof floor. Gaps at all the floor increased after 2011/04/07 aftershock. Inclination is estimated to be about 1/50 by gap measurement. Inclination measurement at dwelling units of each floor is estimated to be about 1/60 to 1/40.
3) The first natural frequency of the building is estimated around 1 to 1.5Hz for both buildings. The peak in TR direction of the building A indicates to distribute in wide range.
4) While TR direction has only one peak in amplitude ratio, LN direction has two peaks. This result leads to the interpretation that dynamic behaviour in TR direction is close to rigid medium with rocking motion at the bottom.
5) Rocking motion is predominate in TR direction of both buildings
6) H/V spectrum shows that the predominant frequency of the ground may exist 1Hz.

Detailed investigations such as excavation under the foundation and the inspection at the pile heads are expected in the near future.

AKCNOWLEDGEMENT

The authors are deeply grateful to Mr. Shuji TAKE of Sendai city, Mr. Tatsuo SASAKI, technical officer of Tohoku University, and students of Prof. Motosaka’s Lab. of Tohoku Univ. Earthquake records to obtain velocity response spectra were provided by K-NET.

REFERENCES

Shiga, T. (1980). Earthquake damage of concrete piles. *Concrete Journal*, Vol. 18, No. 8, 14-21.

Sugimura, Y. and Oh-oka, H. (1981). Report on the damage of precast prestressed concrete piles during the 1978 Off-Miyagi prefecture earthquake, *Kenchiku Kenkyu Shiryo*, BRI, No. 31.

Ohta, Y. and Goto, N. (1976). Estimation of S-wave velocity in terms of characteristics indices of soil, *Buturi-Tanko*, Vol.29, No.4, 34-41