Field Investigation and Dynamic Analysis of Damaged Structure on Pile Foundation during the 2011 off the Pacific Coast of Tohoku Earthquake

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SUMMARY:
The 2011 off the Pacific Coast of Tohoku Earthquake caused settlement and tilting damage to school buildings on pile foundations of a junior high school in Furukawa district in Miyagi Prefecture, Japan. Compression failure at the pile heads of the school building was observed by excavation surveys around the pile heads. In order to clarify a factor of the damage, earthquake response analyses considering a nonlinear interaction of a soil-pile-structure system were conducted using an observed ground motion. From these analyses, the factor of the damage of the pile foundation was confirmed that a response of the pile foundation exceeded an ultimate limit state, which is a flexural deformation capacity. As a result, it is presumed that a flexural compression failure occurred at the pile head and the failure caused the settlement and tilting damage of the school building.

Keywords: 2011 Tohoku earthquake, settlement, field investigation, nonlinear interaction, pile-structure system

1. INTRODUCTION

The 2011 off the Pacific Coast of Tohoku Earthquake (M_w=9.0) occurred on March 11th, 2011. In Furukawa district of Osaki city which is located about 40 km north of Sendai city in Miyagi Prefecture, Japan, strong motions were observed. Collapse of wooden houses, floating of manholes caused by soil liquefaction and settlement of low-rise RC buildings were reported by damage investigations for the west side area of JR Furukawa Station (Goto, 2011). K-NET Furukawa, which is an observation station of strong motions, has been installed by National Research Institute for Earth Science and Disaster Prevention (NIED) in Furukawa district. The peak ground acceleration of the observed motion in the EW direction was about 570 cm/s/s at the ground surface during the earthquake, and that was 460 cm/s/s at an observation station of Japan Meteorological Agency (JMA). A junior high school in Furukawa district, which is located about 2 km southeast of K-NET Furukawa, suffered settlement and tilting damage of RC school buildings on pile foundations from such a large ground motion.

In order to clarify a factor of settlement and tilting of the buildings, this paper firstly describes field investigation results of the superstructure and the pile foundations, excavation surveys around the pile heads and extraction surveys of the pile foundations. Next, earthquake response analyses considering a nonlinear interaction of a soil-pile-structure system are conducted using the observed motion and the main factor of this structural damage is clarified.

2. OUTLINE OF EARTHQUAKE AND RC SCHOOL BUILDING

Fig. 1 shows the locations of Furukawa district and Sendai city in Miyagi Prefecture. Furukawa district is located about 40 km north of Sendai city as mentioned above. The epicenter of the earthquake is also shown in this figure. The earthquake occurred with its epicenter offshore of Sanriku, about 180km east-southeast from Furukawa district.

Fig. 2 shows the relationships between acceleration and displacement response spectrum of the observed motions at K-NET and JMA Furukawa. The response spectra of the observed motions at several points around Furukawa district are also shown in this figure. These observation points are K-NET or KiK net station by NIED and located about 20 km away from Furukawa district. The spectral amplitude of the observed motions at K-NET and JMA Furukawa are larger than the amplitude of other points at the period of around 1s. From these observed motions, it is shown that a ground motion at Furukawa district was larger than other points during the earthquake.

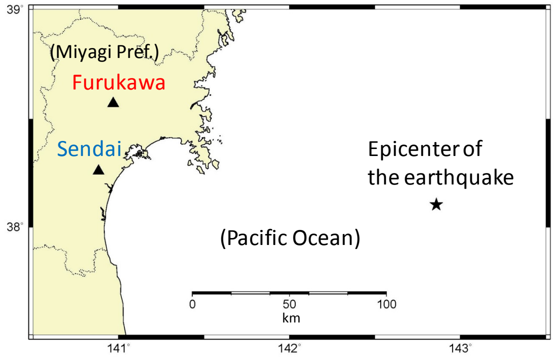


Figure 1. Locations of Furukawa and Sendai and epicenter of the 2011 Tohoku earthquake

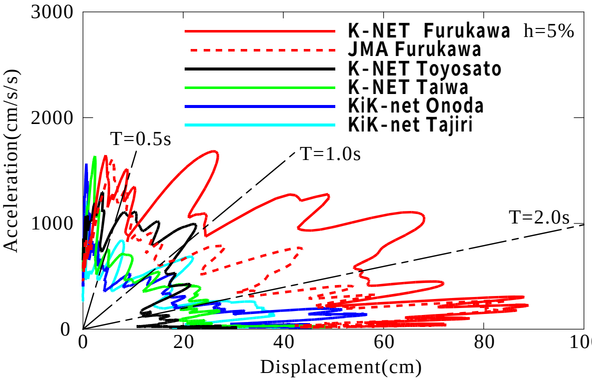


Figure 2. Response spectra of observed motions (EW)

Photo 1 shows the south view of the damaged school buildings, and Fig. 3 shows the plan of the school buildings and the pile foundations. This junior high school comprises four buildings. The main buildings are the west and east school buildings, and two small-scale buildings, which are called the north and new school buildings in this paper, are connected with the east school building through the Exp. J. The west and east school buildings were constructed in 1979, and the north and new school buildings were added each in 1989 and 1991. The superstructure of the west and east school buildings, and the new school building comprises a three-story RC structure, and that of the north school building comprises a one-story RC structure.

Table 1 shows the outline of the pile foundation of each school building. The west and east school buildings are supported by pre-stressed concrete (PC) piles with a diameter of 35cm and a length of 20m. The PC pile is A-type, which means that initial pre-stress is 4N/mm^2. Pre-stressed high-strength concrete (PHC) piles with a diameter of 45cm and a length of 20m are used for the new school building. The PHC pile of B-type is used at the upper part 10m in 20m, and the lower part 10m is A-type. B-type means initial pre-stress is 8N/mm^2. PHC piles with a diameter of 35cm are used for the north school building.

Fig. 4 shows the boring log and the pile foundation. The surface soil over 20m depths comprises very soft layers of peat, clay and silt, which is underlain by a layer of dense gravel with N value=50. The pile foundation shown in Fig 4 is PC pile with a diameter of 35cm, which is used for the west and east school buildings. The pile foundation is supported on a gravel layer at a depth of 20m.

Photo 1. South view of damaged school buildings

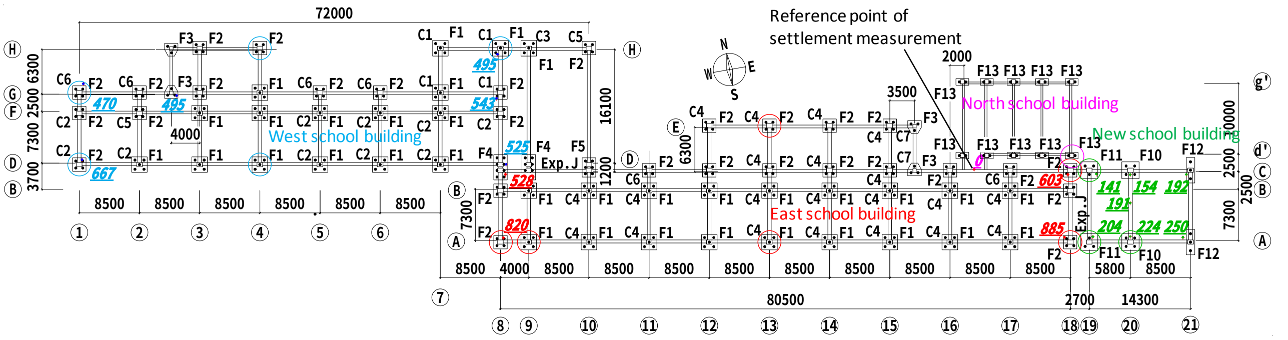


Figure 3. Plan of school buildings and pile foundation
(Settlement is shown by colored number (unit:mm), Excavation survey point is indicated by circle symbol)

Table 1. Outline of pile foundation

School building	Pile type	Diameter(Φ) Thickness(t) Length(L)
West East	PC pile A-type	Φ350mm,t=65mm L=20m
New	PHC pile B-type(upper part 10m) A-type(lower part 10m)	Φ450mm,t=70mm L=20m
North	PHC pile	Φ350mm

A-type : initial pre-stress 4N/mm^2
B-type : initial pre-stress 8N/mm^2

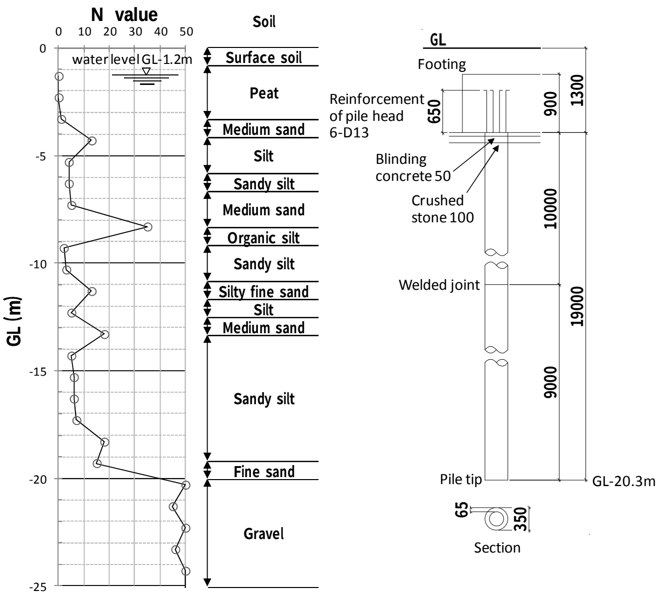


Figure 4. Boring log and pile foundation (D14 in Fig. 3)

3. FIELD INVESTIGATION OF DAMAGE TO SUPERSTRUCTURE AND PILE FOUNDATION

The school buildings on the pile foundations suffered severe settlement and tilting damage from the earthquake. The settlement measured in a field investigation at each point of the school buildings is shown in Fig. 3. The west school building had the maximum settlement of 667mm, and the south side was inclined by the maximum angle of 1/50 rad. The damage of the east school building was severer than the west school building, the maximum settlement was 885mm, and the maximum tilt angle was 1/25 rad to the south side. Concerning the new school building, the maximum settlement was 250mm, and the maximum tilt angle was 1/200 rad to the south side, the damage was slighter than the west and east school buildings.

Photo 2 shows the crack of the entrance floor of the west and east school buildings. The damage was caused by the tilting damage to the south side. Photo 2 also shows the shear failure of the RC non-structural wall of the long side direction of the east school building. Shear cracks of a width of from about 0.2mm to 2mm occurred on other columns. Photo 3 shows the settlement of the east school building and the level difference between the east and new school buildings. The level difference was caused by the differential settlement of the both buildings.

As mentioned above, although the superstructure of the west and east school buildings had from minor to moderate damage, it was judged that damage level was severe by the settlement and tilting damage.

Photo 2. Crack of entrance floor (left) and shear failure of RC non-structural wall in east school building (right)

Photo 3. Settlement of east school building (left) and
level difference between east and new school building (right)

In order to confirm a factor of the settlement and tilting damage, excavation surveys around the pile heads and extraction surveys of the pile foundations were conducted in this study. The excavation surveys were conducted for the pile foundations indicated by circle symbols in Fig. 3.

Photo 4 shows the damage around the pile head of the west school building (D14 in Fig. 3). The pile head had only a few flexural cracks of a width of from 1 to 2 mm, no severe damage was observed on the pile. From this damage, it is thought that a joint condition at the pile head was a pin joint. And the maximum tilt angle observed among the investigated piles was about 15%. It is thought this tilting was caused by an effect of a rotation of the pile head. The extraction survey of this D14 pile was conducted. A partial deficiency of the pile caused by a failure at a depth of about 4m was observed as shown in Photo 4. From these result, it is judged that the factor of the settlement damage of the west school building was the damage of the pile at the position which is deeper than the pile head.

Photo 5 shows the damage around the pile head of the east school building (A9 in Fig. 3). The pile head had a compression failure. From this damage, it is thought that a joint condition at the pile head was a fixed joint unlike the west school building. The extraction survey of this pile was conducted. A flexural crack of a width of 2 mm was observed at a depth of about 15m from the pile tip as shown in Photo 5, no failure occurred on the pile unlike the west school building. From these result, it is judged that the factor of the settlement damage of the east school building was the damage of the pile head.

Photo 4. Pile damage of west school building (D14 in Fig. 3)
Flexural crack at pile head (left), Partial deficiency of pile at a depth of about 4m (right)

Photo 5. Pile damage of east school building (A9 in Fig. 3)
Compression failure at pile head (left), Flexural crack at a depth of about 15m from pile tip (right)

4. EARTHQUAKE RESPONSE ANALYSIS CONSIDERING NONLINEAR INTERACTION

4.1. Evaluation of soil response during the earthquake

Firstly, a bedrock motion is evaluated using the observed motion on the surface at K-NET Furukawa site, which is located about 2 km northwest of the junior high school. An evaluation of the bedrock motion is conducted by a direct integration method considering a soil nonlinearity using the surface motion (Yamada and Miura, 2002). Secondly, a soil response of the site of the junior high school is evaluated using the calculated bedrock motion as an input motion.

Table 2 shows the soil constants of K-NET Furukawa and the junior high school. The S-wave velocities over 17m depths of K-NET Furukawa are based on soil survey results at the site by NIED, and those under the depth are based on soil survey results at a point near JR Furukawa station. The results are shown in the report on earthquake and soil investigation in Miyagi Prefecture (Miyagi Prefecture, 1985). The S-wave velocities over 19.9m depths of the junior high school are calculated from equations for clay and sand (Ohta and Goto, 1976) using N-value in the boring log in Fig. 4. However, since an equation for peat is not evaluated, the S-wave velocity of peat is the same as that of K-NET Furukawa, Vs=70m/s, and the S-wave velocities under 19.9m depths are also the same as those of K-NET Furukawa. Fig. 5 shows the relationships of the shear modulus ratios and the damping factors with the shear strains for the surface layers. These nonlinearities are set by RO model on the basis of laboratory test results including Furukawa area. The results are shown in the previously mentioned report (Miyagi Prefecture, 1985). The observed motion used in the soil response analysis is the motion in EW direction, since the direction corresponds with the long side direction of the school building, and a response of the direction will be examined in section 4.3

Fig. 6(a) shows the pseudo velocity response spectra for a damping ratio of 5% of the bedrock motion of K-NET Furukawa and the calculated ground surface motion of the junior high school. Because of an effect of a soil amplification in the layer over 71m depths, the spectral amplitude of the bedrock motion at the period of around 1s decreases in a half in general to the observed motion on the surface at K-NET Furukawa. The spectrum of the observed motion at the ground surface of KiK-net Onoda by NIED, which is located about 16 km west of K-NET Furukawa, is shown in Fig. 6(a). The S-wave velocity at the ground surface of the KiK-net Onoda is generally the same as that at the bedrock of K-NET Furukawa. Since the calculated bedrock motion corresponds to the observed motion of KiK-net Onoda, it is evaluated that amplitude of the calculated bedrock motion is appropriate as amplitude of a bedrock motion in this area.

Fig. 6(b) shows the distributions of the maximum shear strain and the equivalent S-wave velocity in the layers over 25m depths in the EW direction. The shear strain becomes about 2% at a depth of 3.6m, which is lower end depth of the very soft layer of peat with Vs=70m/s. The equivalent S-wave velocities in the layers over 19.9m depths decrease in a half in general to the initial S-wave velocities.

Table 2. Soil constants of K-NET Furukawa and junior high school

(a) K-NET Furukawa

Depth(m)	Soil	Vs (m/s)	ρ (t/m^3)	Nonlinearity
2.0	Peat	70	1.43	Peat
6.0	Sand	130	1.79	Sand
11.0	Silt	130	1.73	Clay
13.0	Sand	130	1.79	Sand
17.0	Silt	130	1.73	Clay
32.0	Gravel	400	2.11	(Linear)
38.0	Silt	210	1.70	(Linear)
46.0	Silt	300	1.70	(Linear)
66.0	Gravel	500	2.10	(Linear)
71.0	Gravel	500	1.80	(Linear)
(Bedrock)	Rock	700	1.80	(Linear)

(b) Junior high school

Depth(m)	Soil	N-value	Vs (m/s)	ρ (t/m^3)	Nonlinearity
3.60	Peat	0	70	1.43	Peat
4.70	Medium Sand	13	150	1.79	Sand
7.45	Silt	4	130	1.73	Clay
8.70	Medium Sand	35	210	1.79	Sand
11.10	Silt	3	130	1.73	Clay
11.80	Fine sand	13	190	1.79	Sand
12.60	Silt	5	150	1.73	Clay
13.50	Medium Sand	18	200	1.79	Sand
19.15	Silt	8	170	1.73	Clay
19.90	Fine sand	15	210	1.79	Sand
(19.9m～71m : Same as K-NET Furukawa)					
(Bedrock)	Rock	>50	700	1.80	(Linear)

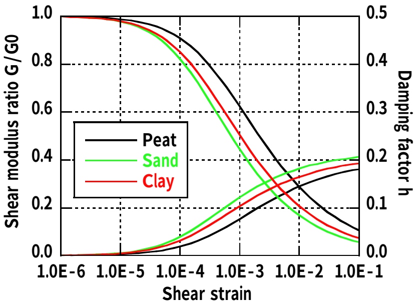
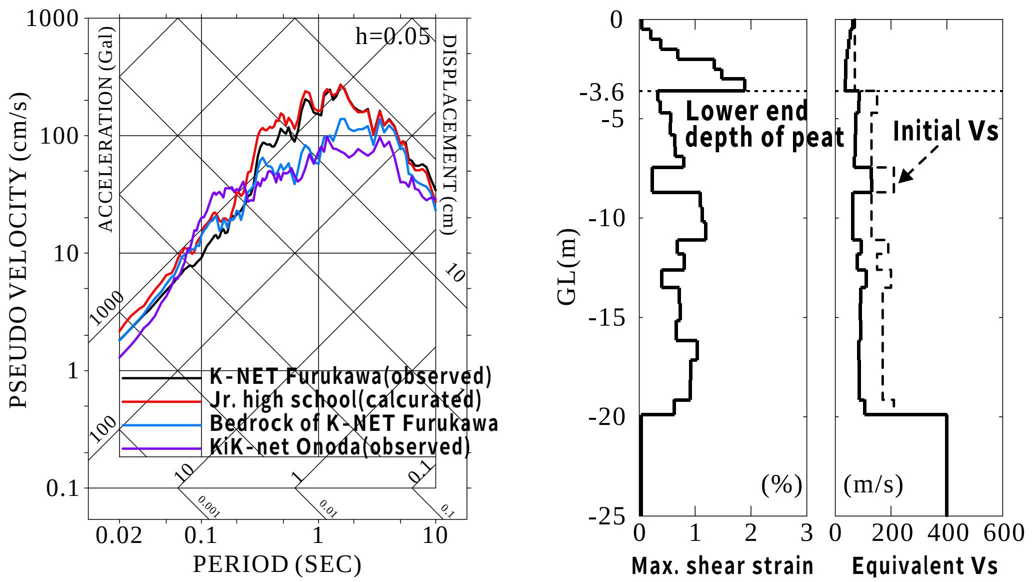


Figure 5. Relationships of shear modulus and damping factor with shear strain



(a) Pseudo velocity response spectra

(b) Distribution of maximum shear strain and equivalent S-wave velocity

Figure 6. Result of soil response analysis

4.2. Analysis model of soil-pile-structure system

The building examined in this study is the east school building, since the damage of the pile heads of the building was confirmed by the excavation surveys around the pile heads, and the building had the maximum settlement and tilting among the four school buildings in the junior high school, as mentioned earlier in chapter 3. Analyses for the long side direction (EW) of the building are conducted, since the damage of the RC columns and non-structural walls in the direction was confirmed.

Earthquake response analyses of the structure supported on the pile foundations are conducted using a beam-interaction spring model (Miyamoto et al., 1997), as shown in Fig. 7. The superstructure and the pile foundations are idealized by a one-stick model of lumped masses and beam elements. Here, the number of the pile foundations is 182, which are idealized by a one-stick model considering a pile group effect. The lumped masses of the pile foundation are connected to free field soil through lateral and shear interaction springs. A linear rotational spring related to an axial stiffness of the piles is also incorporated at the pile head. Initial values of the lateral and shear interaction soil springs are obtained using Green's functions by ring loads in a layered stratum. Nonlinearity of the interaction soil spring is a function of a relative displacement between the soil and the pile.

Table 3 shows the height and weight for each story of the superstructure. The relationship between the shear force and the shear deformation of the superstructure is assumed to be trilinear model for each story, as shown in Fig. 8(a). The strength of the superstructure is based on the value shown in the report on the seismic capacity evaluation for the junior high school. The nonlinear property of the pile foundation is incorporated into the relationship between the bending moment and the curvature, as shown in Fig. 8(b). This is evaluated by a bending analysis with a fiber-model using a material property of the pile, considering a stationary load. Here, the concrete compressive strength and the yield strength of the steel bar of the pile are assumed to be each 50N/mm^2 and 1275 N/mm^2, and the stationary load is set to the weight of the building per one pile, 277kN. The trilinear model is set so that the model fits the analysis result, then the ultimate point is set as the point which a compression strain of the concrete at the edge of the pile become 0.3%.

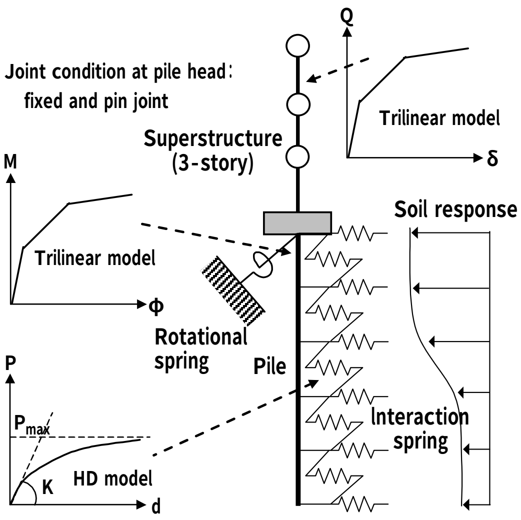
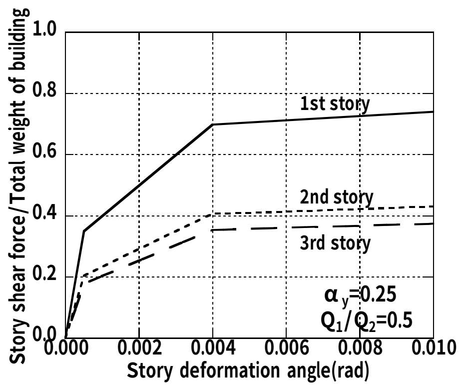


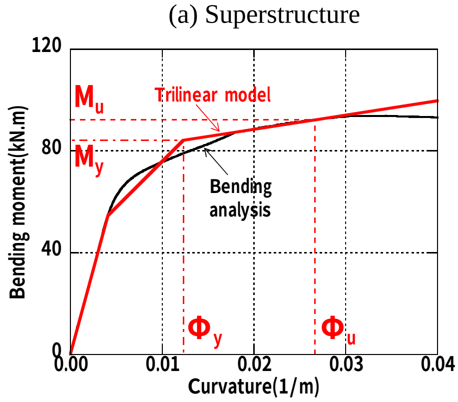
Figure 7. Analysis model of soil-pile-structure system

Table 3. Height and weight of superstructure

Story	Height(m)	Weight w_i (kN)	$W_i = \Sigma w_i$ (kN)
3	3.80	12251	12251
2	3.80	14515	26766
1	4.65	15528	42294
Base	-	8058	50352



(a) Superstructure



(b) Pile foundation

Figure 8. Trilinear model

4.3. Analysis result

Table 4 shows the analysis cases in this study. The analyses of Case-1,2,3 are conducted in order to examine an effect of nonlinearity of a pile and an interaction spring, and an effect of a joint condition at a pile head. In addition to these cases, the analysis of Case-2', in which a superstructure is removed from a soil-pile- structure system, is conducted in order to examine an effect of a shear force from a superstructure (inertial component) and a soil displacement (kinematic component) to a pile response.

Fig. 9 shows the distributions of the maximum relative displacements of the pile-structure system and the maximum story deformation angles of the superstructure. The relative displacement at the pile head of Case-2 is larger than that of Case-1 by the effect of nonlinearity of the pile and the interaction spring. As a result, the story deformation angle of Case-1 is larger than that of Case-2 and exceeds 1/100(0.01) rad at the second story. The relative displacement at the pile head of Case-3 is larger than that of Case-2 by the effect that the joint condition at the pile head is a pin joint, and this result cause that the story deformation angle of Case-3 is smaller than that of Case-2 and becomes about 1/250(0.004) rad at the first and the second story.

It is thought that the joint condition at a pile head of Case-2 corresponds with the actual condition of the east school building, since the damage of the pile heads was observed in the field investigations. And the shear cracks in the RC columns and the non-structural walls at the first story of the east school building were observed in the field investigations. Therefore, it is presumed that the first story deformation angle of Case-2, about 1/250 rad, is generally in agreement with the actual damage.

The maximum relative displacement of the soil with respect to the pile tip is also shown in the figure of the relative displacement. In comparison with the soil displacement, the pile displacement under 3.6m depths, which is lower end depth of the very soft layer of peat, is generally the same as the soil displacement. On the other hand, the pile displacement over 3.6m depths is larger than the soil displacement by the effect of the shear force from the superstructure. It is presumed that since almost the entire shear force from the superstructure was transmitted to the pile foundations by the effect of the very soft layer of peat, the pile displacement over 3.6m depths became larger than the soil displacement

Fig. 10 shows the distributions of the maximum bending moments and the maximum ductility ratios of the pile foundation. The ductility ratios are the ratios of the maximum curvature to the yield curvature. In the case of Case-2, the bending moment exceeds the ultimate bending moment M_u, and the ductility ratio exceeds the ultimate ductility ratio μ_u, which is the ratio of the ultimate curvature to the yield curvature. Therefore it is presumed that the result of Case-2 is generally in agreement with the actual damage, which is the flexural compression failure at the pile heads of the east school building. In the case of Case-3, in which the joint condition at the pile head is a pin joint, the bending moment and the ductility ratio exceed each M_u and μ_u at a depth of 3.6m, which is lower end depth of the very soft layer of peat. Although the analyses for the east school building, it is presumed that the result of Case-3 is generally in agreement with the actual damage at the west school building. The damage is the partial deficiency of the pile caused by the flexural compression failure at a depth of about 4m.

In the case of Case-2', in which a superstructure is removed from a soil-pile-structure system, the bending moment is less than M_u, and the ductility ratio is less than 1.0. From this result, as mentioned earlier, it is presumed that the effect of the shear force from the superstructure (inertial component) is dominant to the response of the pile foundation.

From these results, the factor of the damage of the pile foundation observed in the field investigations of the east school building is confirmed that the response of the pile foundation exceeded the ultimate limit state. It is thought that the response was caused by almost the entire shear force from the superstructure was transmitted to the pile foundation by the effect of the very soft layer of peat over 3.6m depths.

As a result, it is presumed that the flexural compression failure occurred at the pile head, the pile foundation could not support an axial force by the failure at the pile head and the failure caused the settlement and tilting damage of the east school building.

Table 4. Analysis cases

Analysis case	Nonlinearity of Pile	Nonlinearity of interaction spring	Joint condition at pile head	Analysis model
Case-1	Linear	Linear	Fixed joint	Soil-pile-structure system
Case-2	Nonlinear	Nonlinear	Fixed joint	Soil-pile-structure system
Case-2'	Nonlinear	Nonlinear	Fixed joint	Pile model(superstructure is removed)
Case-3	Nonlinear	Nonlinear	Pin joint	Soil-pile-structure system

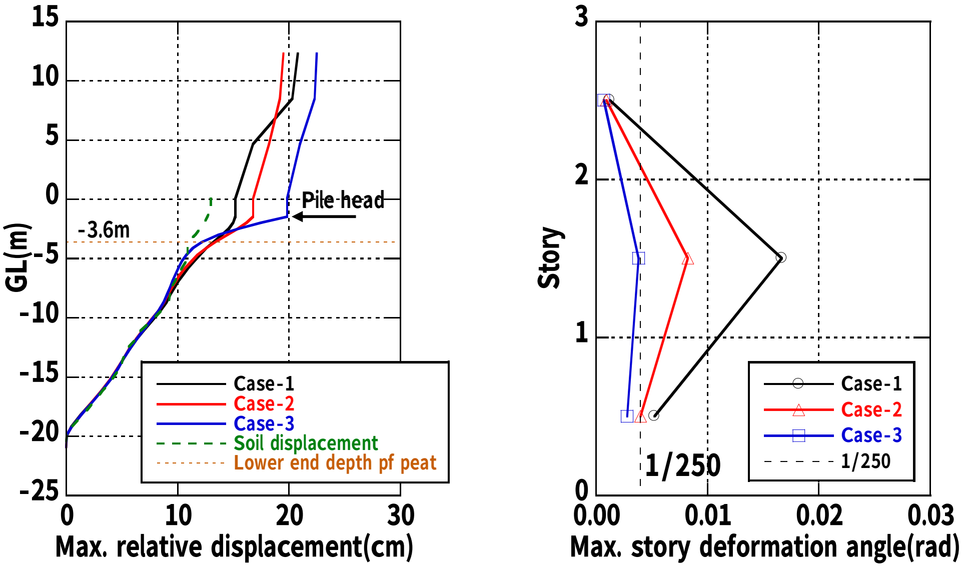


Figure 9. Distributions of maximum relative displacements of pile-structure system and maximum story deformation angles of superstructure

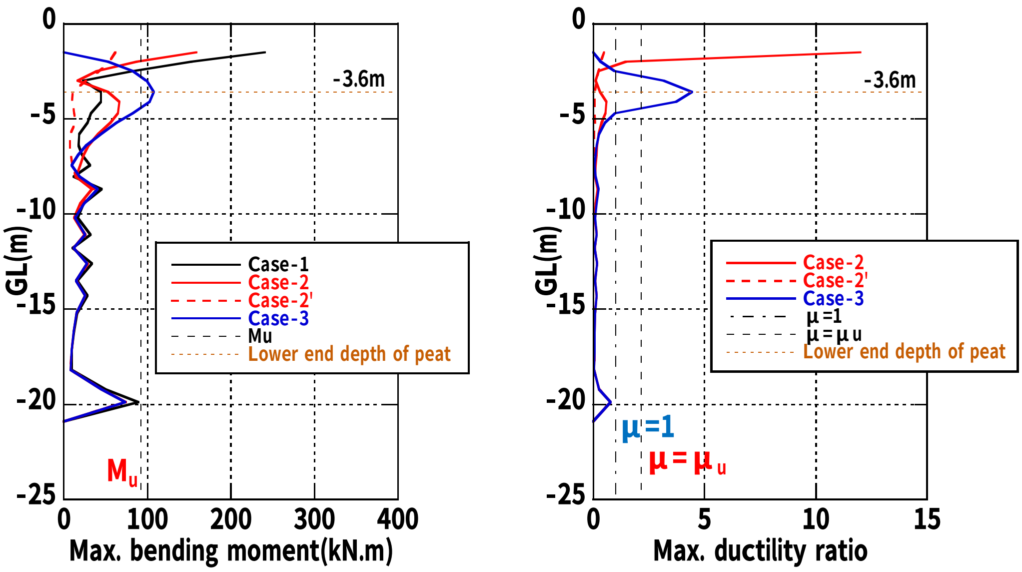


Figure 10. Distributions of maximum bending moments and maximum ductility ratios of pile foundation

5. CONCLUSIONS

The 2011 off the Pacific Coast of Tohoku Earthquake caused settlement and tilting damage to school buildings on pile foundations of a junior high school in Furukawa district. In order to clarify a factor of the damage, field investigations and earthquake response analyses were conducted. The results are described below.

(1) The school building had the maximum settlement of about 80cm, and the south side was inclined by the maximum angle of 1/25 rad. Excavation surveys around the pile heads clarified a rotation of the pile heads and a compression failure at the pile heads. Extraction surveys of these piles revealed a partial deficiency of the pile caused by a failure at a depth of about 4m and a flexural crack of a width of 2 mm at a depth of about 15m from the pile tip.

(2) In the case of an analysis considering a fixed joint at a pile head, the pile response exceeded the ultimate limit state at the pile head. In the case of an analysis considering a pin joint at a pile head, the pile response exceeded the ultimate limit state at a depth of 3.6m, which is lower end depth of a very soft layer of peat. It is thought that these results are generally in agreement with the actual damage mentioned above. On the other hand, in the case of an analysis which a superstructure was removed from a soil-pile-structure system, the pile response was less than the ultimate limit state.

(3) From these results, the factor of the damage of the pile foundation was confirmed that the response of the pile foundation exceeded the ultimate limit state. It is thought that the response was caused by almost the entire shear force from the superstructure was transmitted to the pile foundation by the effect of the very soft layer of peat over 3.6m depths. As a result, it is presumed that the flexural compression failure occurred at the pile head, the pile foundation could not support an axial force by the failure at the pile head and the failure caused the settlement and tilting damage of the school building.

It was thus shown that the result of the earthquake response analysis in this study was generally in agreement with the actual damage. However, in the case of Case-2, the maximum story deformation angle of the 2nd story of the superstructure exceeded 1/100 rad, it is thought that this result overestimates the actual damage. And the pile deformation angle over 3.6m depths of Case-3 is about 4%, it is thought that this result underestimates the maximum angle 15% observed in the excavation surveys. These points are future research issues including a quantitative evaluation of the factor of the damage.

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REFERENCES

Goto, H. (2011). Chapter 4:Characteristics of earthquake and strong motion, *Report on damage investigation of the 2011 Great east Japan earthquake, Japan Society of Civil Engineers*, 4-18 - 4-25. (in Japanese)

Miyagi Prefecture (1985). Report on earthquake and soil investigation in Miyagi Prefecture. (in Japanese)

Miyamoto, Y., Sako, Y., Koyamada, K. and Miura, K. (1997). Response of pile foundation in liquefied soil deposit during the Hyogo-ken nanbu earthquake of 1995, *Journal of structural and construction engineering, AIJ*, **493**, 23-30. (in Japanese)

Ohta, Y. and Goto, N. (1976). Estimation of S-wave velocity in terms of characteristic indices of soil, *Butsuri-tanko(Geophysical exploration)*, **29:4**, 31-41. (in Japanese)

Yamada, A. and Miura, K. (2002). Simulation analyses of groundmotions by characteristic method during the 2000 Tottori-ken seibu earthquake, *Journal of structural and construction engineering, AIJ*, **558**, 77-83. (in Japanese)