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DYNAMIC BEHAVIOR OF A PILE-FOUNDATION BUILDING WITH THE EFFECT OF IRREGULAR PILE-SUPPORTING STRATUM

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Abstract

After the 2011 Tohoku Earthquake, many pile-foundation buildings were found to have been damaged by the strong shaking caused by the main shock. A pile-foundation building supported on an irregular pile-supporting stratum is the focus of this study. One of the important characteristics of this building is that its pile lengths are different on each foundation. The maximum difference of the pile length is 10m between the north and south side with the same structural frame. It is estimated that the pile-supporting stratum is inclined from south to north according to the design documents and the cut and fill distribution map of this area. The thickness of the soft surface layer on the north side of the building is greater than that on the south side.

The authors conducted a damage survey, microtremor measurements, and aftershock observations on this pile-foundation building. Although the damage to the building was not severe, the staircase towers adjacent to the building were severely damaged. Based on the investigations, the shear wave velocity of the surface soil at this site is estimated to be approximately 160 to 180 m/s, and the natural frequencies of the superstructure are 3.5 Hz in the transverse direction and 3.2 Hz in the longitudinal direction. The irregularity of the pile-supporting stratum is also estimated based on the pile lengths of each foundation described in the design documents.

Based on the findings of the field investigations including the damage survey, microtremor measurements, and aftershock observations, a numerical analysis of the building and one of the staircase towers is carried out with the 2D FEM model to simulate the damage caused by the 2011 Tohoku Earthquake. The dynamic characteristics of the surface ground and superstructures, and the piles of the building and the staircase tower are induced by the results of the field investigations. From a practical standpoint, the numerical model is organized to simulate the predominant frequency of the surface ground and the natural frequency of the superstructure.

Based on the results of the numerical analysis, the dynamic behavior of the pile-foundation building is estimated for the main shock and compared with the results of the field investigations. The bending moment at the pile head of the short pile (close to the inclination of the pile-supporting stratum) appears to be larger than the longer pile because of the difference in the pile stiffness and the nonlinear soil response caused by the irregular pile-supporting stratum. The bending moment of the right piles of the building and the staircase tower is close to the capacity of the ductility factor. The responses of the pile of the building tend to be large compared to the damage. However, the bending moments of the piles of the staircase tower are in good agreement with the extent of the damage. It turns out that the effect of the irregular pile-supporting stratum plays an important role in the extent of the damage and the dynamic behavior of the building and the staircase tower.

Keywords: pile-foundation building; 2011 Tohoku Earthquake; irregular ground; FEM

1. Introduction

After the 2011 Tohoku Earthquake, many buildings were found to have been damaged by the strong vibrations caused by the main shock [1]. In the eastern area of the city of Sendai consisting of an alluvial plain, some buildings were damaged owing to a large acceleration amplified by the soft surface layers. The authors conducted damage surveys on those damaged buildings. The dynamic characteristics of a long RC building supported by pile foundations were reported based on the microtremor measurements and the aftershock observation records [2].

One of the important characteristics of this building is that the pile lengths are different on each foundation. It is estimated that the pile-supporting stratum is inclined from south to north according to the design documents and the cut and fill distribution map of this area [3]. The thickness of the soft surface layer on the north side of the building is larger than that on the south side. The amplification characteristics of the ground are assumed to be quite different on the north and south sides of the building. These are considered to be related to the damage of the building. The authors conducted microtremor measurements and aftershock observations. Based on the results of these investigations, the dynamic characteristics of the superstructure of the damaged building and the ground at the site are quantitatively estimated. The effects of the irregular pile-supporting stratum appear in the results of the microtremor measurements and the aftershock observations. However, these observed data were acquired after the main shock of the 2011 Tohoku Earthquake. The dynamic behavior of the building during the main shock is not clearly understood.

In this paper, a numerical analysis using a 2D FEM model is carried out to study the dynamic behavior of the damaged building during the main shock of the 2011 Tohoku Earthquake. The 2D FEM model is organized based on the results from the microtremor measurements and aftershock observation records. The results of the numerical analysis are quantitatively discussed and compared with the extent of the damage. In particular, the response of the pile head is examined considering the effect of the irregular pile length owing to the inclination of the supporting stratum.

2. Outline of building and damage from the 2011 Tohoku Earthquake

The location of the building is shown in Fig. 1. The building exists in the eastern part of Sendai city, approximately 300 km north of Tokyo and 120 km west of the epicenter of the 2011 Tohoku Earthquake, as shown in Fig. 1(a). The eastern part of Sendai city primarily consists of a soft alluvial plain. The hill area near the center of Sendai was developed to extend the residential area after the 1960's. The surface layer of the fill area is reclaimed by the soft soil on an undulating rigid tertiary formation. Therefore, the geological conditions of the man-made residential area are complicated because of the distribution of the cut and fill area.

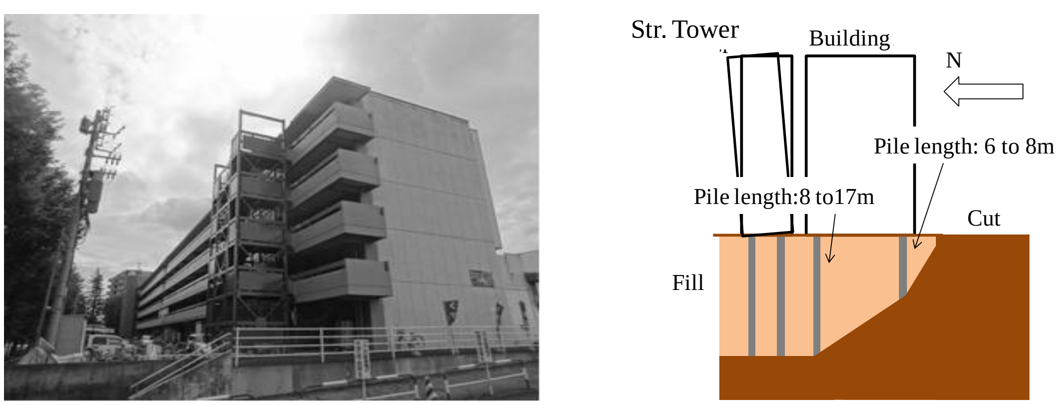
The building was constructed in 1971 in the man-made residential area, approximately 5 km north-east of the center of Sendai. The building site and neighboring fill area distribution are shown in Fig. 1(b). The building is a five-storey RC frame structure that also suffered from the 1978 Miyagi Earthquake. The photo in Fig. 2(a) shows a panoramic view of the building. The vicinity of the site is a man-made reclaimed area, and the building exits around the edge of the fill area. The blue-colored area in Fig. 1(b) indicates the fill area reclaimed by the development of the residential site [3]. According to the design documents, the piles in the north frames are longer than those in the south frames. The difference in pile length between the north and the south is approximately 10 m at most. A schematic figure of the building sections, including the pile foundations and the ground, is shown in Fig. 2(b). The seismic behavior of the building is considered to have been affected by the surface layer of the fill area owing to the irregularity of the pile-supporting stratum.

An outline of the damage and a plan of the building are shown in Fig. 3. The plan of the building consists of a long rectangular shape of 103 m in the longitudinal direction and 9 m in the transverse direction. The building has the three sets of staircase towers adjacent to the north side. The building consists of 17 structural frames, named as Frames A to Q. The structural frames are mainly supported by the foundations, as shown in Fig. 3. The octagon-plan foundation has seven piles in a group, and the rectangular-plan foundation has eight piles in a group. The pile lengths of each foundation are listed in Table 1. All piles turned out to be PC piles with a diameter of 350 mm. The difference in the pile lengths of the north and the south foundations are clearly shown. The piles supporting the staircase towers are much longer than those of the building. The inclination of the pile-supporting stratum from south to north is estimated based on the differences in the pile lengths of the staircase tower, and the north and south foundations of the structural frames of the building.

The authors conducted a damage survey of the building after the 2011 Tohoku Earthquake. One of the most specific characteristics of the damage is a leaning staircase tower resulting from the damage to the foot and pile head of the tower. All three towers indicated a lean to the north. A bending failure at the foot of the tower was observed in the first and second towers. Severe damage at the pile head may have occurred in the third tower, because minor and intermediate cracks were observed. However, no significant failures were found at the foot of the third tower. The tower has an inclination over an angle of 1/30. On the other hand, the superstructure of the building showed no significant structural damage. However, the large shear cracks on the non-structural walls of many housing units and large bending cracks at the center of the girder at the east edge of the building were observed.

(a) Location of Sendai (b) Building site and fill area distribution

Fig. 1 – Location of damaged building (processed from GSI Maps [4])



(a) Panoramic view (b) Schematic section

Fig. 2 – Damaged building [2]

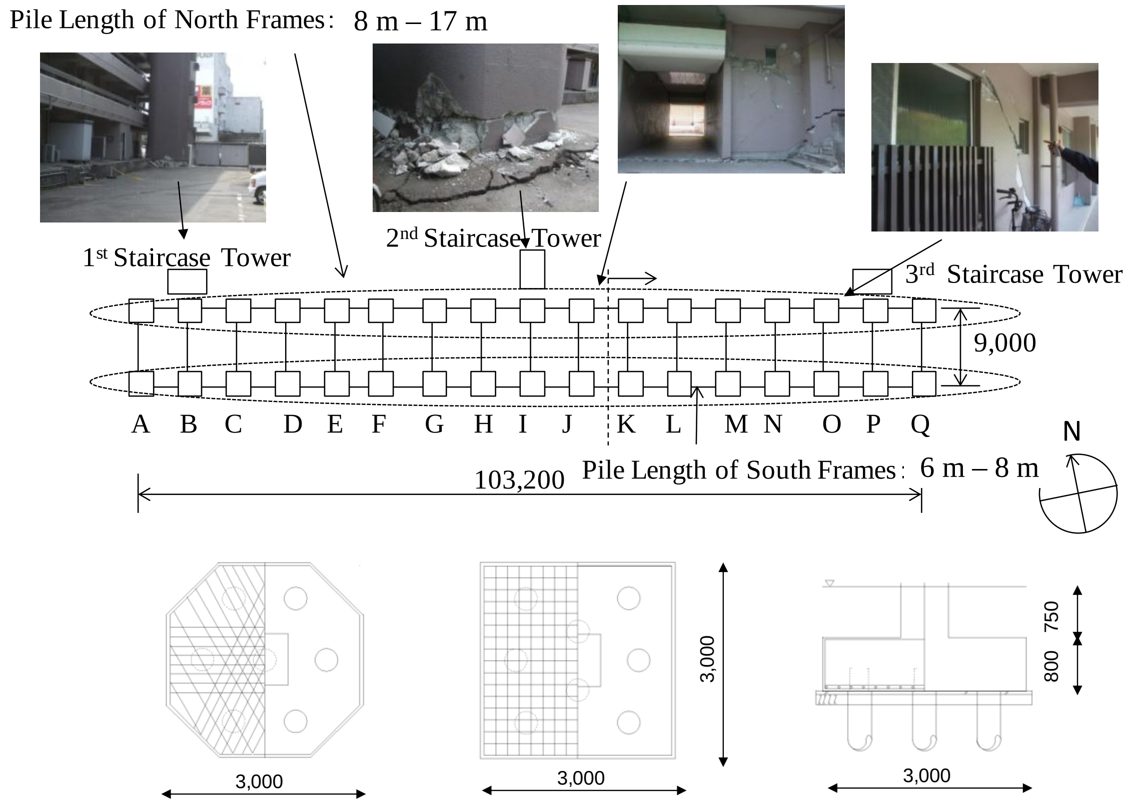


Fig. 3 – Schematic plan of building with damage and figures of the foundations

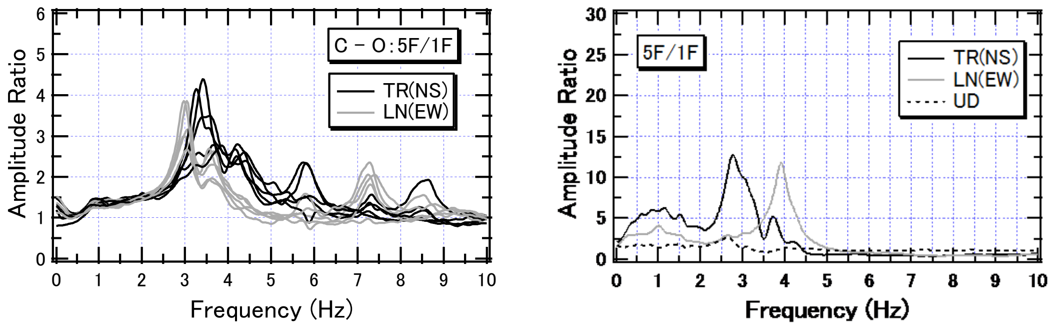
Table 1 – Pile lengths of each foundation (unit: m)

		1st. Str. Tower						2nd. Str. Tower						3rd. Str. Tower			
			19						15						18		
Frame	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q
North Pile	8	12	16	17	16	13	10	8	8	10	10	10	10	13	12	12	12
South Pile	7	10	10	8	6	6	6	6	6	6	6	6	6	6	6	6	6

3. Results of microtremor measurements and aftershock observations

3.1 Dynamic characteristics of the building and staircase tower from the microtremor measurements

Microtremor measurements were carried out after the earthquake to estimate the dynamic characteristics of the building. The results were reported, including a discussion of the dynamic characteristics of the building and the staircase tower [2]. Amplitude ratios between 5F and 1F of the superstructure of the building derived from the microtremor measurement data of the structural frames C, G, I, K, and O are shown in Fig. 4(a). The natural frequencies fluctuate depending on the frames. They are estimated at approximately 3.5 Hz for a fixed-base system (5F/1F) in the TR (NS) direction. The amplitude ratios between 5F and 1F of the third staircase tower are also shown in Fig. 4(b). The natural frequencies are estimated at 2.8 Hz in the TR (NS) direction, and 3.9 Hz in the LN (EW) direction, for the fixed-base system (5F/1F). While the amplitude ratio curves of the superstructure have multiple peaks, indicating torsional motion and higher modes, the amplitude ratio curve of the staircase tower has a single distinctive peak, indicating a bending-type first mode. The natural frequency in the TR (NS) direction is lower than that in the LN (EW) direction. This is the effect of the rectangle plan with a strong axis in the NS direction and a weak axis in the EW direction. A small peak is observed, indicating torsional motion in the TR (NS) direction immediately after the first mode and close to the natural frequency in the LN (EW) direction. It is also interesting that the UD direction has a spectral peak at almost the same range as the TR (NS) direction. This is because the tower inclined owing to the damage from the earthquake. Therefore, the spectral peak in the UD direction indicates a coupling effect of the TR (NS) direction from the inclination.



(a) Superstructure of the building (b) Staircase tower

Fig. 4 – Amplitude ratios between 5F and 1F [2]

3.2 Dynamic characteristics of the ground of the site, from aftershock observation records

To collect earthquake records, aftershock observations of the first floor of the building were conducted for approximately one year after October 2012. Two sets of seismometers were installed in the center and east parts of the long building to compare their response characteristics. The positions of the seismometers are illustrated in Fig. 5, where (a) is the seismometer at the center, and (b) is the seismometer in the east part of the building. Earthquake observation records are available for the vicinity of the site of the building. The site of "TRG" is governed by the International Research Institute of Disaster Science (IRIDeS) of Tohoku University. The site of TRG is approximately 500 m west of the site of the building. Soil conditions at the site of TRG are considered to be relatively rigid rock. Details of the aftershock observation records were reported in a previous paper [2].

Attempts to estimate the one-dimensional wave propagation characteristics of the reclaimed soft soil layers at the site of the building were conducted based on an analysis of the earthquake observation records. From the results of Fig. 6, the predominant frequency of the soft soil layer is assumed to be 2.5 Hz. According to the design documents for the building, the pile lengths of the north frames range from 8 to 17m. Two cases are prepared for the estimation, where the depth of the surface layer is assumed to be 15 m (Case 1) and 20 m (Case 2). Using the information for the predominant frequency and the depth of the surface layer, the shear wave velocity of the subsurface layer is estimated to be 180m/s for Case 1 and 240 m/s for Case 2 by Eq. (1). In the damage survey after the 2011 Tohoku Earthquake, the shear wave velocity of the damaged man-made reclaimed residential area of the hill side of Sendai is estimated to be 160 to 180 m/s by the surface wave method [5]. The results estimated from the observed earthquake records are in good agreement with the damage survey of the man-made reclaimed residential area.

$$f = Vs / 4H \tag{1}$$

The relative relations between the observation sites are discussed using the results of Fig. 6, where the average amplitude ratios between the center (a) and the east (b) of the building and TRG are compared in three directions. The amplitude ratios of a/TRG and b/TRG indicate clear peaks at approximately 2.5 Hz in both the TR (NS) and LN (EW) directions. This result leads to the assumption that the one-dimensional dynamic characteristics of the reclaimed soft soil layer at the site of building can be considered to be 2.5 Hz in the horizontal directions. Some small peaks appeared at 4 Hz, 5 Hz, and at higher frequencies that considered the effect of the irregularity of the pile-supporting stratum. The effect is more remarkable in the TR (NS) direction.

A specific spectral peak cannot be observed in the UD direction, while in the results of microtremor measurements, the motion of the foundation in the UD direction is relevant to that in the TR (NS) direction. Seismometers (a and b) were installed near the center of the plan in the NS direction, as shown in Fig. 5. By contrast, the sensors for the microtremor measurements were placed at the north side of the building. Therefore, the effect of the coupling in the TR (NS) and UD directions can be observed only in the results of the microtremor measurements and not in the results of the earthquake observation records. The difference in the amplitude ratio of the center (a) and the east (b) of the building can be found in the TR (NS) direction. A spectral peak is located from 3.0 to 3.5 Hz, which is close to the natural frequency of the superstructure, according to the microtremor measurements.

Fig. 5– Location of seismometers “a” and “b” of the building and TRG [4]

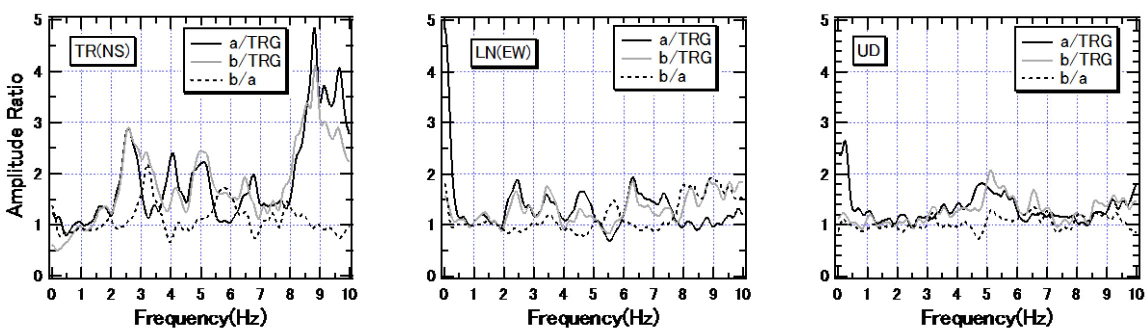


Fig. 6 – Average amplitude ratios between observation sites[2]

4. Numerical analysis of dynamic behavior of the building using 2D FEM model

4.1 Model and the parameters for numerical analysis

A numerical analysis using a 2D FEM model [6] is carried out to study the dynamic behavior of the building and the third staircase tower during the main shock of the 2011 Tohoku Earthquake. The dynamic behavior of the building and the staircase tower in the transverse direction is studied to discuss the effect of the irregular pile-supporting stratum. The structure of the surface layers of the FEM model, in particular, the angle of the pile-supporting stratum is organized based on the pile lengths described in the design documents. Severe damage is observed at the foot of the staircase tower. The serious damage is assumed to have occurred at the head of the piles, judging from the inclination of the tower and the damage conditions at the foot of the third staircase tower.

In order to develop a dynamic model of FEM, the soil properties of the model ground, and the parameters of the superstructure and the piles, are determined from the information described in the design documents and the results of the microtremor measurements and the aftershock observation records, as mentioned in the previous section. The model of the FEM is shown in Fig. 7. Viscous boundaries are assumed at both sides and at the bottom of the ground model. The soil properties of the model ground are listed in Table 2.

Because the practical soil properties of the site are not available, the *N* values of the SPT are assumed, and the shear wave velocities *Vs* and internal friction angles ϕ are estimated based on some assumptions. The shear wave velocities are determined by Eq. (2) as proposed by Ohta and Goto [7]. Here, *N* is the *N* value of the SPT, *H* is the depth, *E* and *F* are the coefficients representing the geological period and soil type, respectively. In this simulation, *E* = 1.00 indicates an alluvial layer, and *F* = 1.086 indicates fine sand.

$$V_s = 68.79N^{0.171} \cdot H^{0.199} \cdot E \cdot F \tag{2}$$

The nonlinear characteristics of the surface soil layers of the ground are expressed by the modified RO model [6] shown in Eq. (3), (4), and (5) where γ is the shear strain, τ is the shear stress, τ_f is the shear strength, G_{max} is the maximum of the shear modulus, α and β are the parameters of damping, R_f is the control coefficent of the shear strength, h_{max} is the maximum damping. The estimated soil properties are listed in Table 2, and the control coefficents for the shear strengths of R_f and h_{max} are assumed to be 0.03 and 0.3, respectively.

$$\gamma = \frac{\tau}{G_{max}}\left\{1 + \alpha\left(\frac{\tau}{\tau_f/R_f}\right)^{\beta-1}\right\} \tag{3}$$

$$\alpha = 2^{\beta-1} \tag{4}$$

$$\beta = \frac{2 + \pi h_{max}}{2 - \pi h_{max}} \tag{5}$$

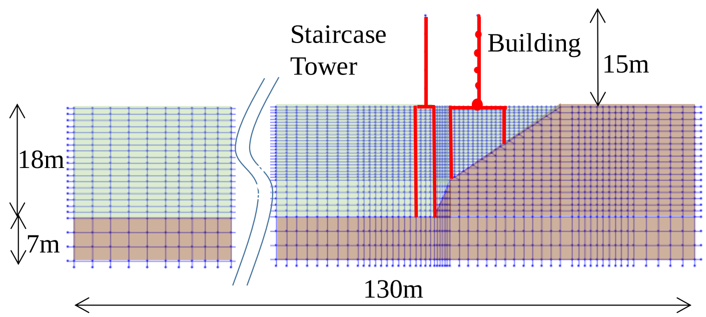


Fig. 7 – 2D FEM model

Table 2 – Soil properties of the ground model

GL-(m)	*N*	Vs(m/s)	φ(°)	ν
0 - 6	10	96 - 155	29	0.3
6 - 12	15	172 - 196	32	0.3
12 - 18	25	222	35	0.3
18 - 25	40	300	40	0.3

The parameters relevant to the dynamic characteristics of the superstructure of the building are shown in Table 3 and Fig. 8. The five-story superstructure of the building is modeled as an MDOF model with shear springs. The building investigated in this study is a long RC structure that is over 100 m in length; thus, the numerical model is organized as one structural frame in the transverse direction. The spans of the superstructure are 9 m in the transverse and 6m in the longitudinal directions. An area of 6 x 9 m is modeled as the superstructure for one floor. Furthermore, the area of the model is reduced to an area of 1 x 9 m because the FEM model assumes a plane strain condition. The weights of each layer are calculated from an assumption of 12 kN/m^2 for the RC structure. Shear stiffness properties of each floor are determined by the assuming that the drift angles of each floor respond in the same manner as the external force of the Ai distribution shape, which is evaluated based on the first natural frequency from the results of the microtremor measurements. This is shown in Fig. 4. The nonlinear characteristics of the story shear force and relative story displacement of the superstructure are assumed to be bi-linear. The story shear coefficients of the yield strength are assumed to be Cy = 0.5 on the first floor. The coefficients for other floors are determined based on the Ai distribution.

The parameters indicating the dynamic characteristics of the piles of the building and the staircase tower are shown in Table 4. All the piles are modeled by the beams with the bi-linear skeleton curves. The width of the foundation of the staircase tower is 5 m in the out-of-plane direction of the FEM model (Fig. 7). As the superstructure model is reduced from the width of 6 m to 1m, the width of the staircase tower is also decreased to 1 m. Numbers of the piles in one foundation are seven for the superstructure, 12 for the staircase tower. Considering the reduction effect of the width of the model, parameters on the piles for the numerical model are multiplied by 7/6 of the single pile for the superstructure, and 6/5 for the staircase tower, where the staircase tower is supposed to be supported by two foundations of six piles each. Fig.8 shows the relations of the bending moment resistance and curvature of the pile of this study, and a PC pile with a diameter of 350 mm [8]. Although the nonlinear relation should have practically the tri-linear type skeleton, that of the pile of this study is modeled as a bi-linear type because of the limitations of the FEM software. The curvature at the yield bending moment is 0.009, and that at the ultimate bending moment is 0.025. Table 5 lists the parameters for the staircase tower for the simulation to meet the results of the first natural frequency of the staircase tower from the microtremor measurements.

Table 3 – Parameters of supersturcture of the buliding

No.	W(kN)	K1(kN/m)	F1(kN)
5	70	0.18×10^{5}	65
4	120	0.32×10^{5}	137
3	120	0.44×10^{5}	193
2	120	0.55×10^{5}	239
1	120	0.65×10^{5}	275

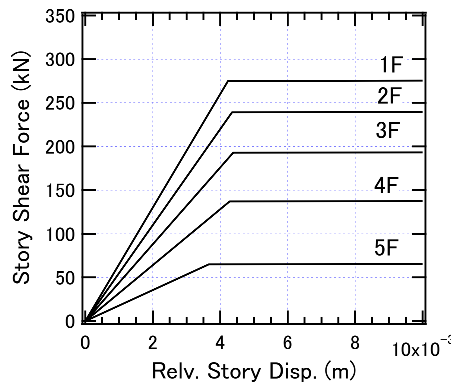


Fig. 8 – Story shear force and relative story displacement of superstructre of the bulding

Table 4 – Parameters of the piles of the building and staircase tower

	A(m^2)	I(m^4)	E1(kN/m^2)	E2(kN/m^2)	M1(kNm)
Pile of Building	6.53×10^{-2}	7.15×10^{-4}	1.82×10^{7}	$\rightarrow 0$	117.0
Pile of Str. Tower	6.70×10^{-2}	7.33×10^{-4}	1.82×10^{7}	$\rightarrow 0$	120.0

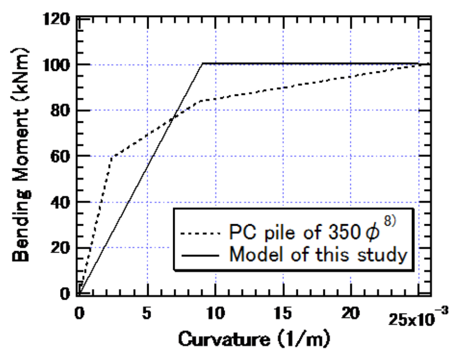


Fig. 8 – Bending moment resistance and curvature relation of the pile

Table 5 – Parameters of staircase tower

A(m^2)	I(m^4)	E1(kN/m^2)	E2(kN/m^2)	M1(kNm)
3.0	3.75×10^{-1}	2.40×10^{7}	$\rightarrow 0$	1093.5

The input motion of the simulation for the main shock is shown in Fig. 9. This time history is a record of the NS direction of the site of the Sumitomo Seimei Building (SU), governed by the IRIDeS of Tohoku University. The site of SU is located in the central area of Sendai. It can be considered the bedrock site based on the soil conditions and previously observed data. The duration of the time history of the main shock can be over 300 (s). The input motion for the simulation is reorganized by the two parts extracted from the two phases, as shown in Fig. 9. The time increment for the numerical integration is 0.001 (s), and the viscous damping of the soil is a Rayleigh type consisting of 2% for the first and second modes. The 2% of the Rayleigh-type viscous damping is also applied to the model of the superstructure, the staircase tower, and the piles.

Before discussing the results for the main shock, a linear case analysis is conducted to discuss the validity of the model parameters where one of the aftershocks recorded at the TRG site is adopted as the input motion (maximum acceleration is approximately 20 cm/^{2}). The results are shown in Fig. 10, where (a) is the amplitude ratio between 5F and 1F of the building and the staircase tower, and (b) is the amplitude ratio between the first floor of the building and the TRG site that corresponds to the amplitude ratio shown on the left side of Fig. 6. The first natural frequencies of the superstructure and the staircase tower are approximately 3.1 Hz and 2.8 Hz. The predominant frequency of the ground is approximately 2.1 Hz. A comparison of the results from the simulation and the microtremor measurements is shown in Fig. 4 and Fig. 6. This can lead to a discussion of the validity of the numerical model. Although the predominant frequency of the ground is shifted to a lower frequency range, the results are in good agreement with the dynamic characteristics estimated from the results of the microtremor measurements and aftershock observations.

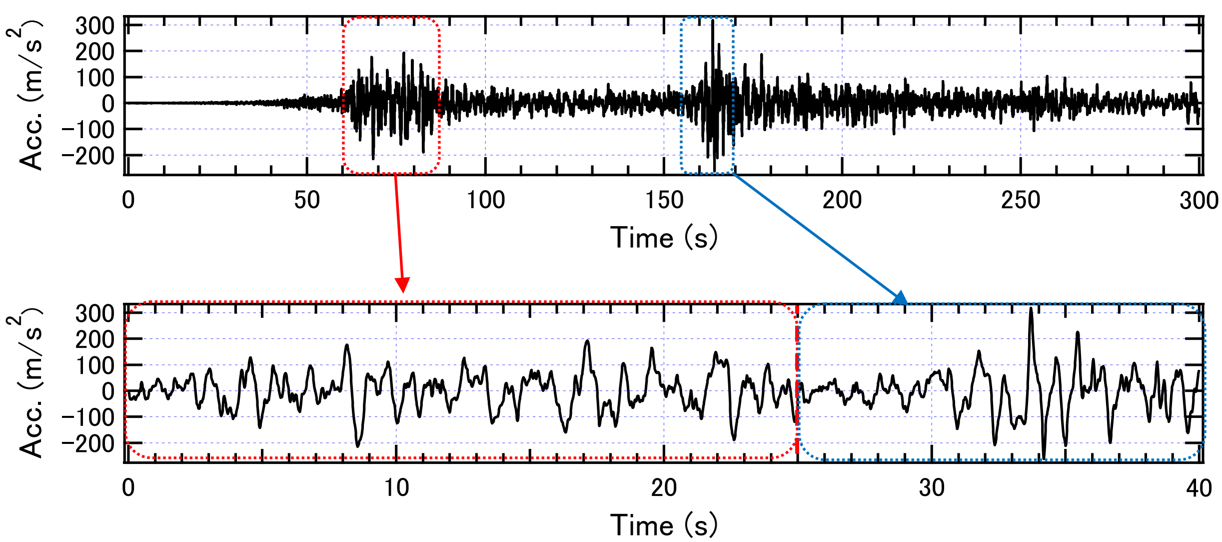
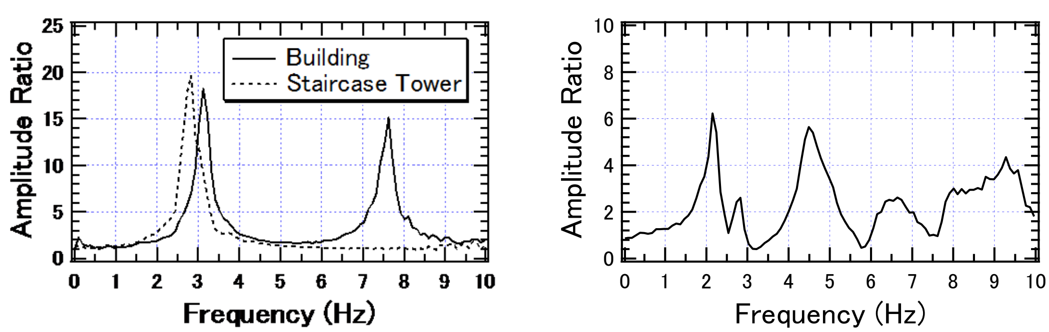


Fig. 9 – Input motion for numerical analysis of main shock



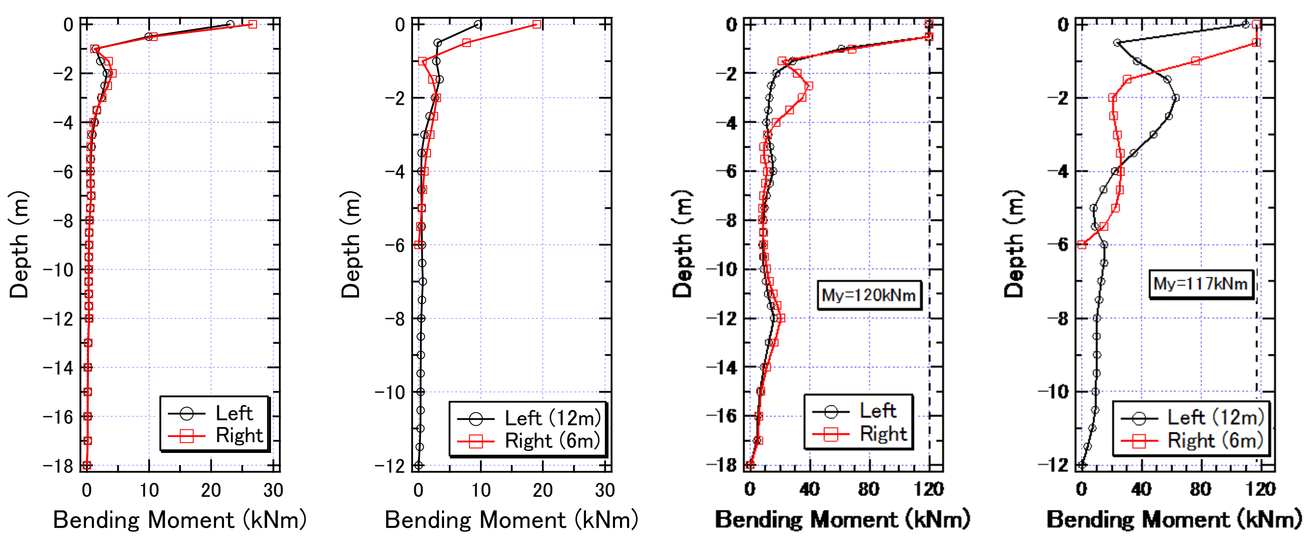
(a) 5F/1F of building and staircase tower

(b) 1F/TRG

Fig. 10 – Amplitude ratios by numerical analysis of aftershock record

4.2 Results and discussions

The results for the main shock are illustrated in Fig. 11 to 13 in order to discuss the dynamic response of the piles and examine the differences in the pile lengths and the effects of the irregular pile stratum. The distributions of the maximum bending moments of the piles of both the building and the staircase tower in depth are shown in Fig. 11 for (a) the linear analysis of the aftershock and (b) the nonlinear analysis of the main shock. It should be noted that Young's modulus of the pile is assumed to be 4.0 x 10^7 kN/m^2 for the linear analysis, which is different from that listed in Table 4. The figures of Fig. 11 (a) shows that in the linear analysis, the bending moments at the pile heads of the right piles are larger compared to those of the left piles for both the staircase tower and the building. In particular, in the results for the building, the bending moment at the right pile is much larger than that of the left pile at the pile head. The left pile is 9.6 kNm and the right pile is 19.0 kNm. In the results for the nonlinear analysis of the main shock [see Fig. 11 (b)], the bending moments at the pile heads exceed the yield bending moments for both the staircase tower and the building, except for the left pile of the building. The yield bending moments are also described in the results for the main shock (My = 120.0 kNm for the staircase tower and My = 117.0 kNm for the building). The bending moments at the pile heads of the staircase tower are larger than the yield bending moment for both the left and the right piles. For the building, the bending moment at the pile head of the right pile is larger than that of the left pile. The length of the right pile of the building is shorter than the left pile owing to the inclination of the pile-supporting stratum. The results reflect the difference in the pile lengths of the building. The bending moment of the left pile of the building tends to increase in the middle of the ground at approximately GL-2 m. This is considered to be an effect of the nonlinear responses of the soil layers. The same trend can be found in the results for the right pile of the staircase tower (approximately GL-3 m). The bending moments of the pile heads are in good agreement with the extent of the damage. However, the results for the right pile of the building seem too large compared with the damage to the building. The nonlinear responses of the soil layers are susceptible to the assumptions of the parameters of the dynamic characteristics. Therefore, the results are dependent on the assumptions of the nonlinear model of the soil. The process of determining the parameters should have more attention paid to it, and will be discussed in a future study.

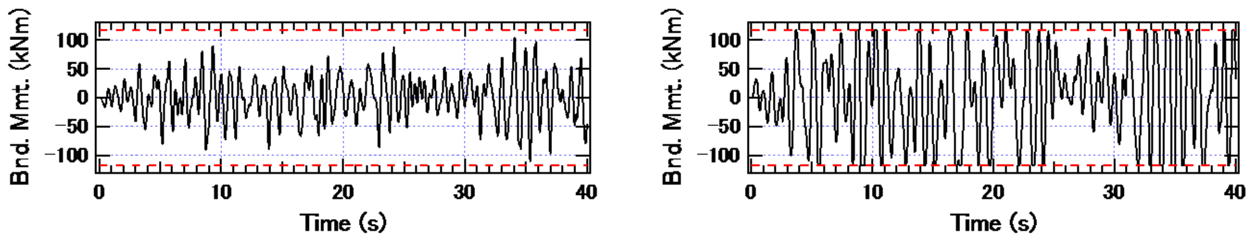


Str. Tower   Building   Str. Tower   Building

(a) Aftershock   (b) Main shock

Fig. 11 – Maximum bending moment distribution in depth

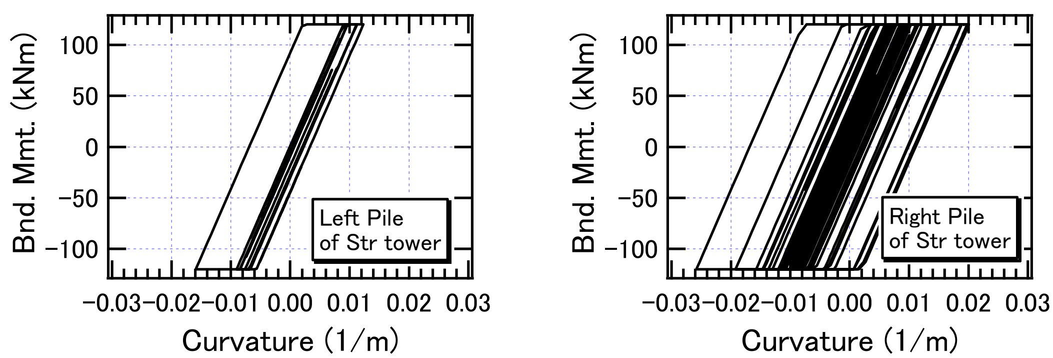
Fig. 12 shows the time histories of the bending moments at the pile heads. Fluctuations in the dynamic responses of the bending moments at the pile heads are clearly shown in the time histories. For the left pile, the response of the bending moment is less than the yield bending moment of 120.0 kNm. On the other hand, the response of the right pile of the building indicates a strong nonlinearity for almost all durations corresponding to the results observed in the maximum bending moment distribution of Fig. 11. Fig. 13 shows the relations of the bending moment and the curvature at the pile heads of the staircase tower and the building. As shown in Fig. 11 and 12, a nonlinear response strongly appears for the right pile. The ductility factors representing the extent of the structural damage are 1.8 for the left pile and 2.8 for the right pile in the staircase tower, and 2.6 for the right pile in the building. The relation of the left pile of the building does not show the nonlinearity. The results indicates that the nonlinear response of the pile head of the staircase tower is larger than that of the building. If the capacity of the ductility factor is assumed to be 3.0, the pile head of the staircase tower is considered to have been severely damaged, and the pile head of the right pile of the building is also close to severe damage. The results of the staircase tower are in good agreement with the extent of the observed damage in the previous section. However, the response of the building tends to be larger than the observed damage. It may be considered that there is still some more room for improving the model with regard to the response of the right pile of the building.



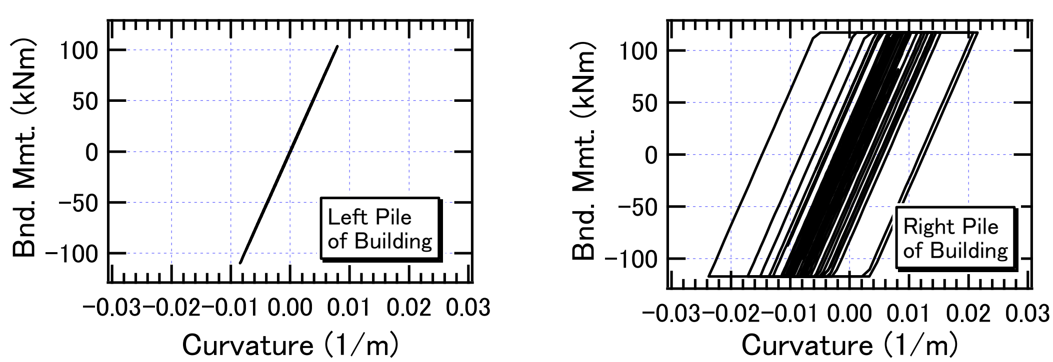
Left (12 m) Right (6 m)

Fig. 12 – Time histories of bending moment at pile heads of the building

(Dashed lines indicate yield bending moment of 117.0 kNm)



Left (12 m) Right (6 m)



Left (12 m) Right (6 m)

Fig. 13 – Bending moment and curvature relations at pile heads

5. Conclusions

The effect of an irregular pile-supporting stratum on the dynamic behavior of a pile-foundation building is discussed. Severe damage to the staircase tower was observed by a damage survey after the 2011 Tohoku Earthquake. The tower has an inclination over an angle of 1/30. Based on the findings of the damage survey, microtremor measurements, and aftershock observation records, a numerical analysis of the building and the staircase tower during the main shock of the 2011 Tohoku Earthquake is carried out using a 2D FEM model. This model is used to study the effect of the irregular pile-supporting stratum on the dynamic behavior of the building and the staircase tower. Our conclusions are as follows:

1) It is confirmed that the effect of the irregular pile-supporting stratum appears in the dynamic responses of the piles. In the response of the building, the bending moment of the right pile (6 m) becomes larger than that of the left pile (12 m). This can be considered an effect of the difference in the stiffness of the pile owing to the pile length, and the nonlinear response of the soil layers. The nonlinear response of the soil is also dependent on the irregular pile-supporting stratum. The bending moment of the left pile in the middle of the ground (GL-2 m) tends to increase. This could be a result of the nonlinear response of the surface soil layers.

2) The ductility factor of the right pile of the staircase tower is 2.8. If the capacity of the ductility is assumed to be 3.0, the result is in good agreement with the extent of the damage to the staircase tower by the main shock. By contrast, the ductility factor of the right pile of the building is 2.6, which is close to 3.0. The response of the building in this simulation can be considered too severe compared with the extent of the damage. The reason for this difference is mainly attributed to the estimation of the soil property, especially the nonlinear characteristics of the surface soil layers.

The results of the numerical analysis of the staircase tower are in good agreement with the extent of the damage and the results from the observed data. However, there are some points that do not correspond to the extent of the damage of the building. The numerical analysis model should be improved in a future study.

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References

[1] Architectural Institute of Japan (2012): Preliminary Reconnaissance Report of the 2011 Tohoku-Chiho Taiheiyo-Oki Earthquake (Geotechnical, Geological and Earthquake Engineering), Springer

[2] Mitsuji K, Ohno S, Motosaka M (2014): Damage survey and observed data analysis of long RC building with irregular pile supporting stratum after the 2011 Tohoku earthquake. *Second European Conference on Earthquake Engineering and Seismology*, Paper No.1176, Istanbul, Turkey.

[3] Fukken Gijutsu Consultant Co. Ltd. (2008): 1/25,000 Map of man-made residential area of Sendai (in Japanese)

[4] Geospacial Information Authority of Japan (GSI): http://www.gsi.go.jp

[5] Sendai City (2012) : Damage survey report of residential area (in Japanese)

[6] FORUM 8 (2014) : UWLC Ver.2 Operation Guidance, (in Japanese)

[7] Ohta Y, Goto N (1976): Estimation of S-wave velocity in terms of characteristic indices of soil, *Butsuri-Tanko*, **29** (4):34–41.

[8] COPITA (2014): Calculating program of N-M, M-ϕ and shear stress of piles, (in Japanese)