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Performance of Piled Raft Foundation Subjected to Strong Seismic Motion

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ABSTRACT: This paper offers a case history of a low-rise building, located in Ibaraki Prefecture, supported by a piled raft in medium to dense sand underlain by over consolidated silty soil. To confirm the validity of the foundation design, field measurements were performed on the foundation settlement and the load sharing between the piles and the raft from the beginning of the construction to 80 months after the end of the construction. During the monitoring period, 44 months after the end of the construction, the 2011 off the Pacific coast of Tohoku Earthquake struck the site of the building. The peak horizontal ground acceleration of 3.24 m/s^2 was observed in the neighbourhood of the building. Although the foundation settlement near the centre of the raft increased by 4 mm from the pre-earthquake value of 21 mm to 25 mm, no significant changes in the load sharing between the piles and the raft were found after the strong seismic motion.

1. INTRODUCTION

In recent years there has been an increasing recognition that the use of piles to reduce raft settlements can lead to considerable economy without compromising the safety and performance of the foundation (Poulos 2001). Detailed investigations of many high-rise buildings founded on piled rafts in Germany have been performed (Katzenbach et al. 2000; Mandolini et al. 2005). Piled raft foundations have been used in Japan for many buildings, including tall buildings in excess of 150 m in height, since a piled raft was first used for a four-story building in 1987 and a basic design framework has been established early in the 2000s (Yamashita and Kakurai 1991; Yamashita 2012). The effectiveness of piled raft foundations in reducing average and differential settlements has been confirmed by field monitoring of a number of piled rafts, not only on favourable ground conditions, but also on unfavourable ground conditions with ground improvement techniques (Yamashita et al. 2011a; Yamashita et al. 2011b; Yamashita et al., 2013).

It has become necessary to develop more reliable seismic design methods for piled rafts, particularly in highly seismic areas such as Japan. Mendoza et al. (2000) reported on the static and seismic behaviour of a friction pile-box foundation supporting an urban bridge in Mexico City clay. Yamashita et al. (2012b) reported the behaviour of a piled raft foundation supporting a 12-story base-isolated building in soft ground in Tokyo during the 2011 off the Pacific coast of Tohoku Earthquake, where a peak horizontal ground acceleration of 1.75 m/s^2 was observed. However, only a few case histories exist on the monitoring of soil-foundation interaction behaviour before and after large earthquakes.

This paper offers a case history of a low-rise building supported by a piled raft foundation on medium to dense sand underlain by over consolidated silty soil. To confirm the validity of the foundation design, field measurements were performed on the foundation settlement, the axial loads of the piles, the contact pressure of the raft and the pore-water pressure beneath the raft. During the monitoring period, 44 months after the end of construction, the 2011 off the Pacific coast of Tohoku Earthquake struck the building site. In this paper, the results of the monitoring on the settlement and the load sharing of the piled raft subjected to the strong seismic motion are presented and the effects of the strong motion on the foundation behaviour are discussed.

In addition, a part of the results of the field monitoring presented in this paper have been previously published (Yamashita et al., 2011a; Yamashita et al., 2012a).

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2. BUILDING AND SOIL CONDITIONS

The hadron experimental hall is located at J-PARC (Japan Proton Accelerator Research Complex) in Ibaraki Prefecture (Photo 1). The J-PARC is a joint project between KEK (High Energy Accelerator Research Organization) and JAEA (Japan Atomic Energy Agency) to study material and life science, hadron and particle physics using high-intensity and high-energy proton beam. Figure 1 shows a schematic view of the structure with a soil profile, together with the results of shear wave velocity measurements of soil. The hadron experimental hall, 19 m in height above the ground surface, is a steel reinforced concrete structure. The subsoil consists of loose to dense sand with SPT *N*-values of 7 to 40 to a depth of 6 m from the ground surface, underlain by diluvial dense sand-and-gravel with SPT *N*-values of 60 or higher and medium to dense sand with SPT *N*-values of 20 to 60 to a depth of 16 m. Between the depths of 16 to 23 m, lie medium sandy silt, loose silty sand and dense sand. Between the depths of 23 and about 40 m, lie stiff sandy silt and silt layers with unconfined compressive strengths of 180 to 480 kPa, underlain by a weathered sandy mudstone.

Photo 1 Hadron experimental hall in J-PARC

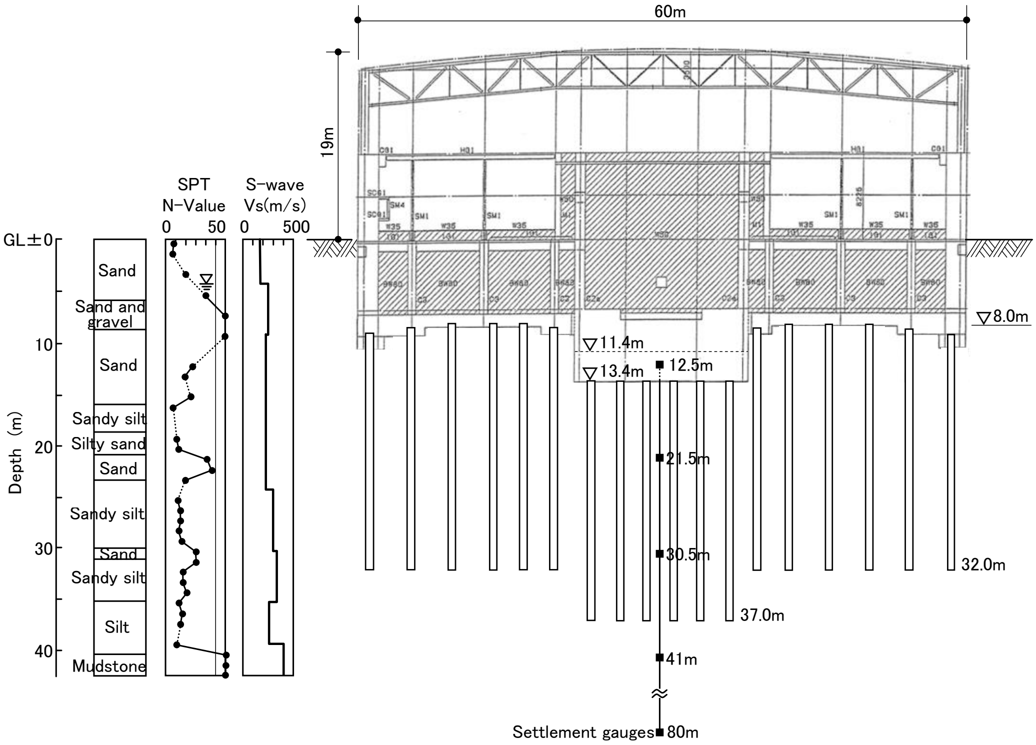


Figure 1 Schematic view of structure with soil profile

The groundwater table appears about 4 m below the ground surface. The shear wave velocities derived from the P-S logging using down-hole technique, carried out at a neighbouring site, were 230 to 250 m/s in the medium to dense sand just below the foundation levels and 400 m/s in the weathered mudstone.

3. FOUNDATION DESIGN

Figure 2 shows the foundation plan with a layout of the piles. The hadron experimental hall consists of a beam line, an experimental line and a beam dump. The average pressure over the raft was 259 kPa (196 kPa) in the experimental line, 350 kPa (294 kPa) in the beam line and 442 kPa (294 kPa) in the beam dump: the figures in the parentheses were the live loads. The live loads were relatively large, 67 to 84% of the total load, because a lot of iron and concrete shielding blocks were to be set up after the end of the construction. The foundation level of the beam line is at a depth of 11.4 m, partly 13.4 m, and that of the beam dump is at the depth of 13.4 m, while the foundation level of the experimental line is at a depth of 8.0 m from the ground surface (Photo 2). A reinforced concrete mat was founded on the dense sand-and-gravel and medium to dense sand, so that the allowable bearing capacity at the foundation levels was much higher than the average contact pressure. If a raft foundation were used, the maximum settlement would be estimated to be greater than the allowable value of 40 mm due to the compression of the cohesive soil layers below the depth of 23 m. Therefore, to reduce the settlement of the raft foundation, a piled raft foundation consisting of 371 PHC (pre-tensioned spun high-strength concrete) piles was proposed. The PHC piles are 22.0 to 25.5 m in length and have diameters varying from 0.60 to 0.80 m (330 piles of 0.60 m, 35 piles of 0.70 m and 6 piles of 0.80 m).

For designing a piled raft, the settlements and the load sharing between the piles and the raft were computed by the simplified method of analysis developed by Yamashita et al. (1998). The results of the analysis indicated that the maximum settlement was 32 mm and the ratio of the load carried by the piles to the effective structure load was 0.6 to 0.8 (Yamashita et al., 2011a). The influence of lateral loading on a piled raft was also considered, i.e., the maximum bending moment and shear force acting on the cross sections of the piles were computed using a simplified method of analysis based on Mindlin's solution proposed by Hamada, et al. (2012).

The piles were constructed by inserting a couple of 9 to 15 m long PHC piles into a pre-augered borehole filled with mixed-in-place soil cement.

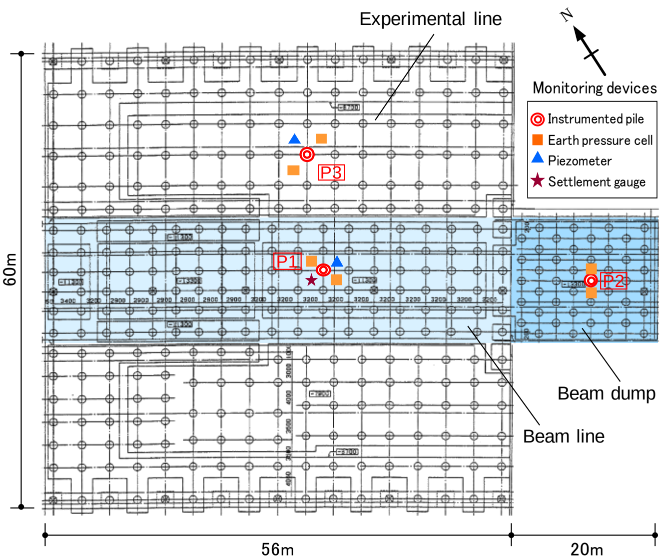


Figure 2 Foundation plan with locations of monitoring devices

4. INSTRUMENTATION

Field measurements were performed on the vertical ground displacements below the raft, the axial loads of the piles and the contact pressure between the raft and the soil, as well as the pore-water pressure beneath the raft from the beginning of the construction to 80 months after the end of the construction. The locations of the monitoring devices are shown in Figure 2. The LVDT-type transducers were installed near the centre of the raft at depths of 12.5, 21.5, 30.5 and 41.0 m to measure the relative displacements to a reference point at a depth of 80 m from the ground surface, as shown in Figure 1. Three piles, P1, P2 and P3, 0.6 m in diameter were installed with a couple of LVDT-type strain gauges at the pile head and the pile toe. A pair of LVDT-type earth pressure cells was installed beneath the raft near the instrumented piles. The LVDT-type piezometers were installed near Piles P1 and P3.

Photo 2 View of ground surface at foundation level in beam line

5. RESULTS OF MEASUREMENTS

Figure 3 shows the measured vertical ground displacements relative to the reference point near the centre of the raft. The ground displacement at the depth of 12.5 m was approximately equal to "foundation settlement" when it was initialised just before the casting of the foundation mats. The foundation settlement was 12.4 mm at the end of the construction. Thereafter, the settlement increased due to the setting up of the shielding blocks and reached 20.7 mm on March 11, 2011, just before the 2011 off the Pacific coast of Tohoku Earthquake. Figure 4 shows the measured axial loads of Pile P1 and the contact pressure and the pore-water pressure beneath the raft near Pile P1. Figure 5 shows those of Pile P2 and the contact pressure and the pore-water pressure beneath the raft near Pile P2 where the values of the pore-water pressure were estimated from those near Pile P1. The measured pile-head load of Pile P1 increased considerably after the end of the construction due to the setting up of the shielding blocks. The measured contact pressure increased gradually after the end of the construction, while the measured pore-water pressure was almost constant.

As for Pile P3, the measured pile-head load was approximately zer**o** even after the end of the construction. This was likely that the actual load was much smaller than the design load in the experimental hall and the load was carried mostly by the buoyancy force.

However, further consideration about the measured data would be required to clarify the behaviour of Pile P3.

Figure 6 shows the time-dependent load sharing among the piles, the soil and the buoyancy and the ratio of the load carried by the pile to the effective structure load versus time, together with that to the total structure load versus time, in the tributary area of Pile P1.

Here, the total load was the sum of the measured pile-head load and the raft load, which was derived from the mean value of the measured contact pressures, and the effective load was the total load minus the buoyancy. Figure 7 shows those in the tributary area of Pile P2. The ratios of the load carried by the piles to the effective load were estimated to be 0.85 in Pile P1 and 0.67 in Pile P2 just before the earthquake. The ratios of the load carried by the piles to the total load were estimated to be 0.67 in Pile P1 and 0.45 in Pile P2.

6. EFFECT OF EARTHQUAKE ON FOUNDATION BEHAVIOUR

6.1 The 2011 off the Pacific coast of Tohoku Earthquake

The 2011 off the Pacific coast of Tohoku Earthquake, with an estimated magnitude of M_w= 9.0 on the Moment Magnitude Scale, struck East Japan at around 14:46 (local time) on March 11, 2011. Figure 8 shows peak ground acceleration map derived from strong motion records of K-NET and KiK-net (Kunugi et al., 2012). The distance from the epicentre to the building site was about 270 km and the seismic intensity in the site was estimated to be "6 weak" according to JMA (Japan Meteorological Agency).

Hashimura et al. (2011) has reported about the response characteristics of a five-story base-isolated building located 0.4 km south from the hadron experimental hall. Figure 9 shows the time histories of accelerations observed at a depth of 6 m below the ground surface near the five-story building. The peak horizontal ground acceleration was 3.24 m/s^2 and the peak vertical one was relatively large value of 2.77 m/s^2. Figure 10 shows the response spectra with a 5% damping ratio of the horizontal acceleration of the ground and that of the basement. It can be seen that the amplitudes of the short-period components are larger and the predominant period was around 0.2 s.

Photo 3 shows the ground subsidence along the northeast side of the hadron experimental hall near the north corner. The ground subsidence reached a maximum of 1.2 m, which was presumed to be caused by compaction of the backfill sand due to the strong horizontal and vertical ground motion. Although the monitoring system was suspended immediately after the earthquake because of power outage, the system was restored 28 days after the earthquake.

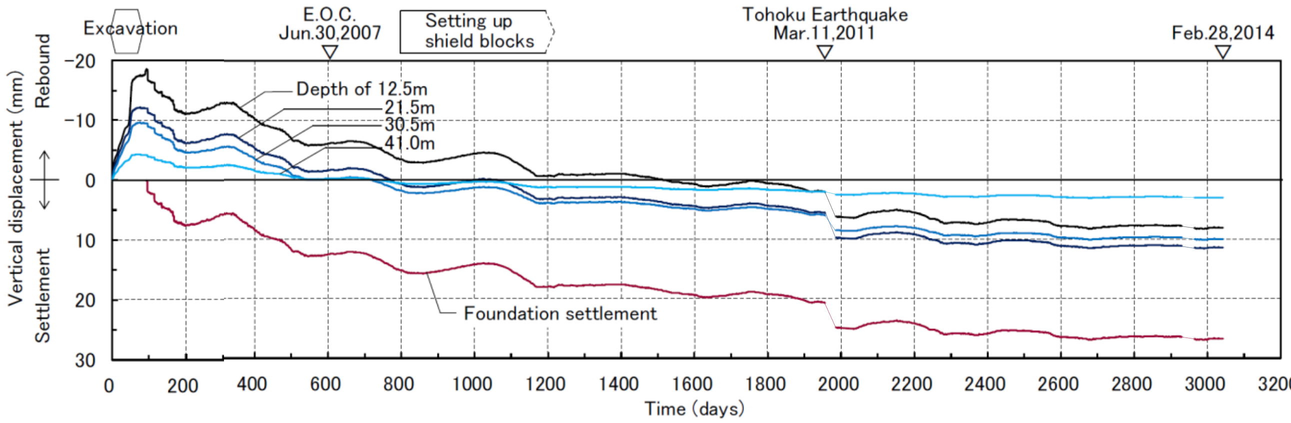
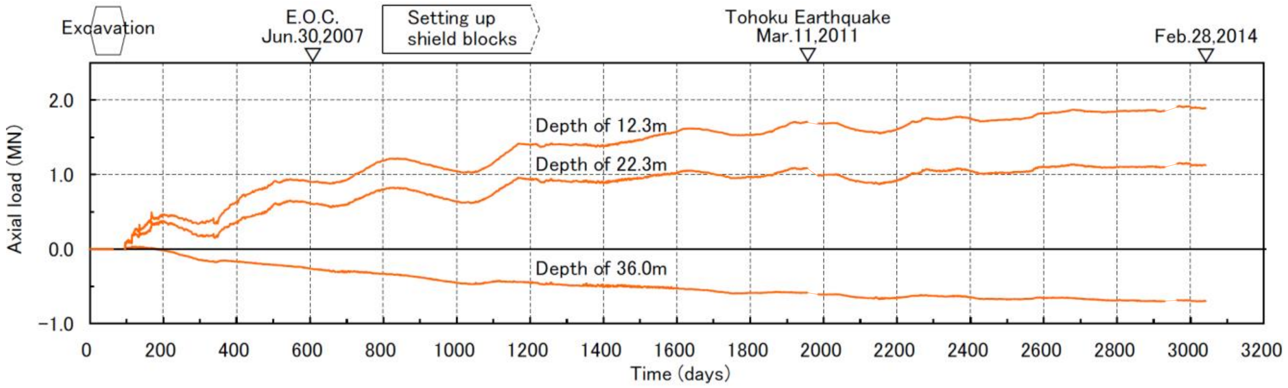
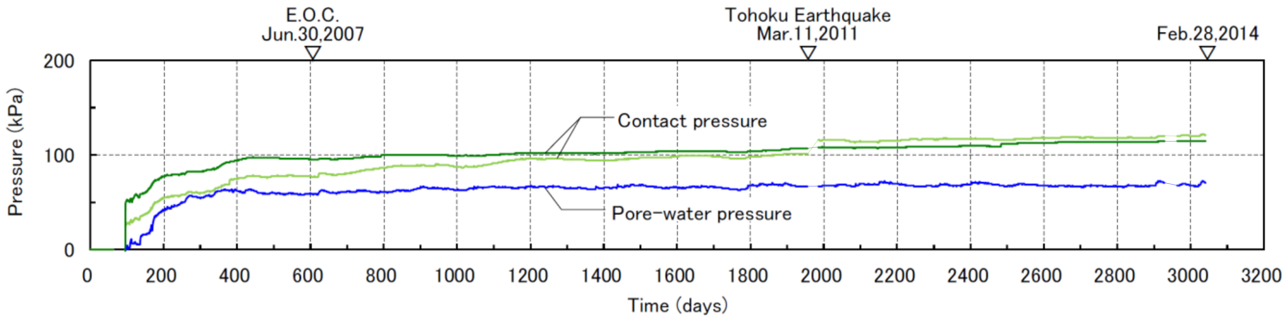


Figure 3 Measured vertical ground displacements in beam line

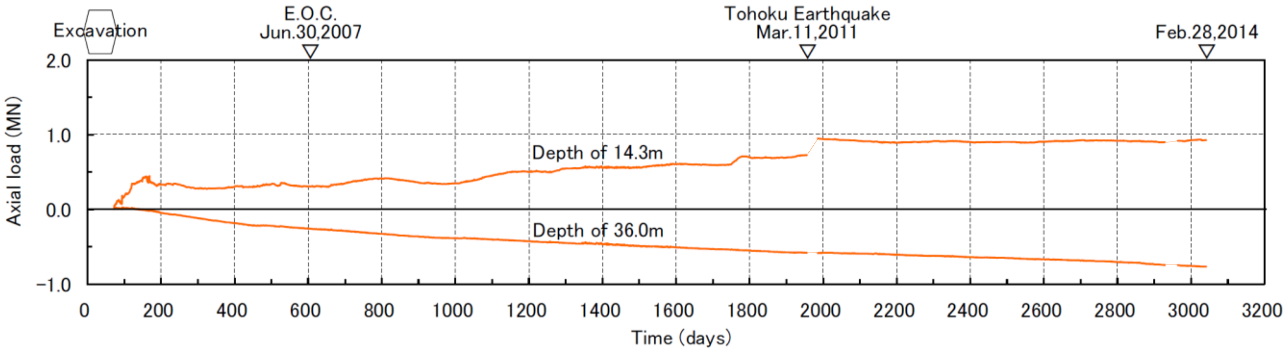


(a) Axial loads of Pile P1

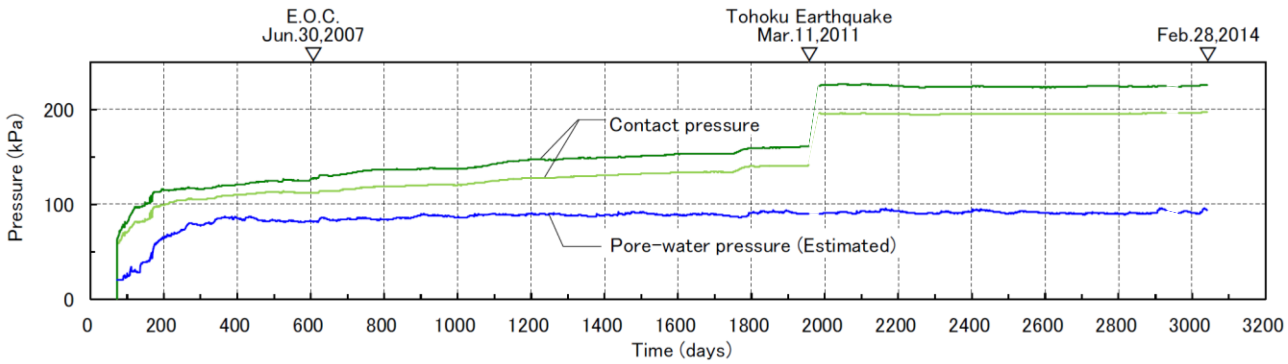


(b) Contact pressure and pore-water pressure

Figure 4 Measured axial loads of pile and contact pressure in beam line

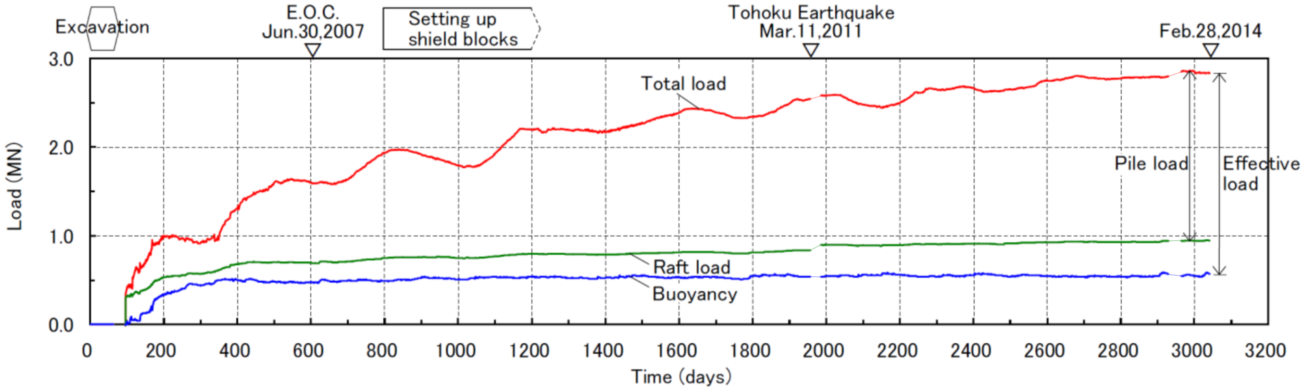


(a) Axial loads of Pile P2

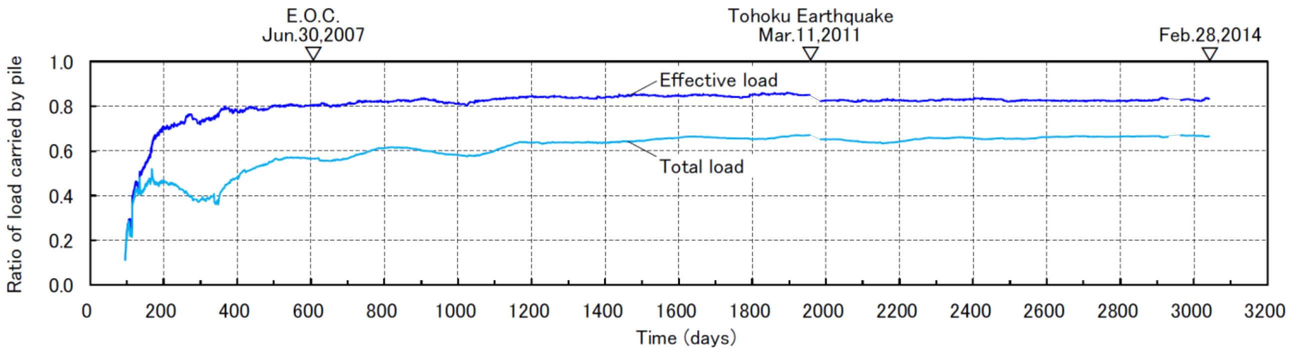


(b) Contact pressure and pore-water pressure

Figure 5 Measured axial loads of pile and contact pressure in beam dump

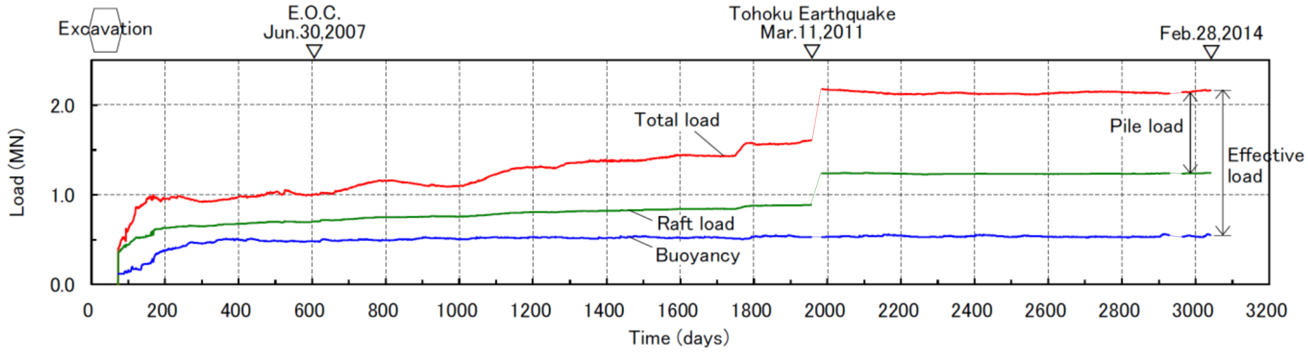


(a) Load sharing between pile and raft

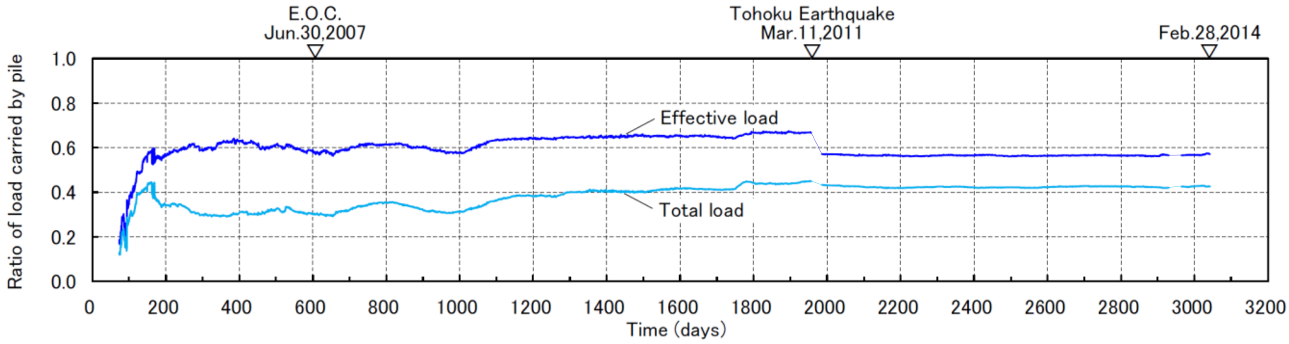


(b) Ratio of load carried by pile

Figure 6 Load sharing between piles and raft in tributary area of Pile P1



(a) Load sharing between pile and raft



(b) Ratio of load carried by pile

Figure 7 Load sharing between piles and raft in tributary area of Pile P2

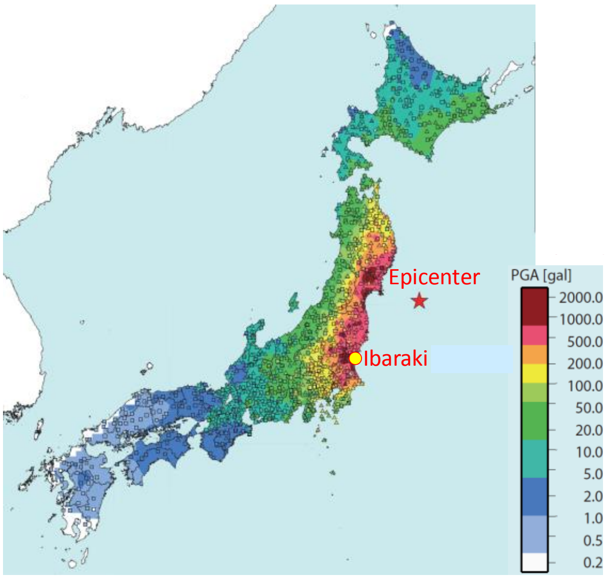
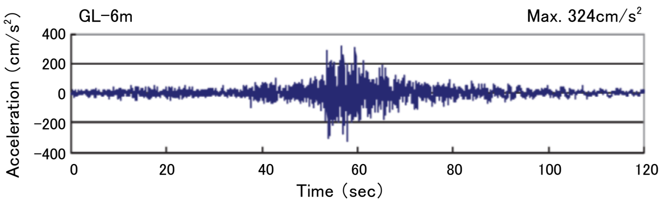
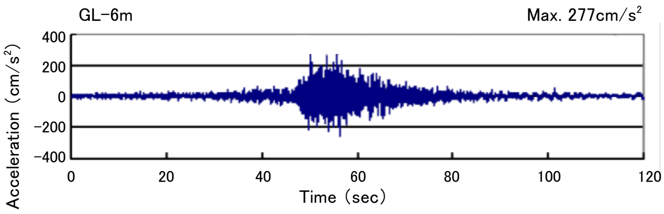


Figure 8 PGA map derived from strong motion records (Kunugi et al., 2012)



(a) Horizontal direction



(b) Vertical direction

Figure 9 Time histories of accelerations at Tokai (Hashimura et al., 2011)

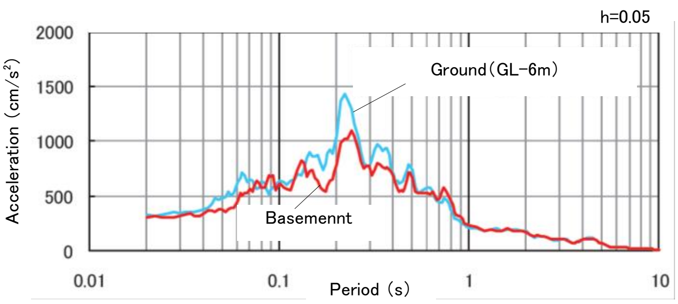


Figure 10 Response spectra of horizontal accelerations at Tokai (Hashimura et al., 2011)

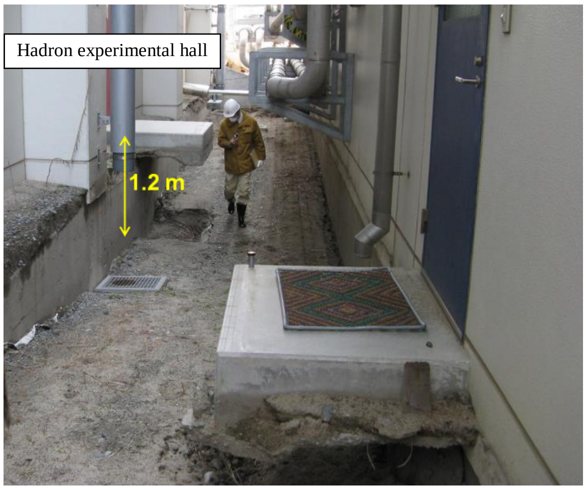


Photo 3 Ground subsidence along building

6.2 Effect of earthquake on settlement and load sharing

Figure 11 shows the profiles of the measured vertical ground displacements near the centre of the raft just before and after the earthquake, where the measured displacements were initialised just before the casting of the foundation mats. The foundation settlement increased by 4.1 mm from the pre-earthquake value of 20.7 to 24.8 mm 28 days after the earthquake. It can be seen that the increments in the ground displacements occurred mostly by the compression of the stiff silty soil layers between depths of 23 and 41 m. It is likely that the compression of the silty soil was caused by the vertical cyclic loading due to the inertial force acting on the structure, mostly via the pile group.

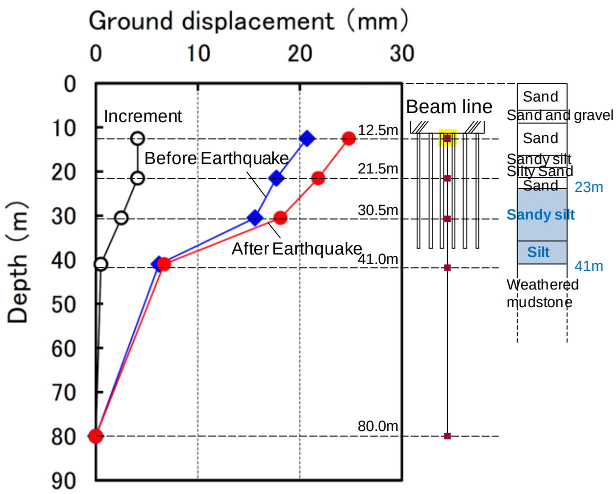


Figure 11 Profiles of vertical ground displacements

After the earthquake, a part of the shielding blocks were removed to align experimental devices in the beam line. This caused the decrease in the total load and the foundation settlement in the beam line, as shown in Figure 6(a) and Figure 3. The shielding blocks were set up again about six months after the earthquake and the foundation settlement increased. Thereafter, the foundation settlement was stable and reached 26.7 mm at the end of the observation, 36 months after the earthquake. The hadron experimental hall resumed operation in January 2012, after the alignment of the experimental devices.

The axial loads of Pile P1 decreased only slightly and the contact pressure near Pile P1 increased slightly after the earthquake as shown in Figure 4. On the other hand, the axial load of Pile P2 at pile head increased 30% and the contact pressure near Pile P2 increased 39% after the earthquake as shown in Figure 5. The increase in the loads of both the pile and the raft seemed to be caused by the loss of the vertical frictional resistance on the basement walls due to the subsidence of the backfill sand induced by the strong seismic motion. So, a part of the structure load, which was supported by the frictional resistance, was transferred to the bottom of the raft and distributed to the soil beneath the raft and the piles. The increase in the pile load and the contact pressure appeared remarkably in the beam dump because the ratio of the perimeter to the planar area was relatively large. The measured pore-water pressure was not affected by the seismic motion as shown in Figures 4(b) and 5(b).

The ratio of the load carried by the piles to the effective load in the tributary area of Pile P1 decreased only slightly from 0.85 to 0.82 and that of Pile P2 decreased slightly from 0.67 to 0.57 28 days after the earthquake. Thereafter, the ratios of the load carried by the piles to the effective load were quite stable, i.e., 0.83 for the former and 0.57 for the latter at the end of the observation, 36 months after the earthquake. Incidentally, though the monitoring system was suspended again at the beginning of November, 2013 because of data storage capacity trouble, the system was restored in December, 2013 as shown in Figures 3 to 7. Meanwhile, the ratios of the load carried by the piles to the total load decreased only slightly from 0.67 to 0.65 in Pile P1 and from 0.45 to 0.43 in Pile P2 28 days after the earthquake. Thereafter, the ratios of the load carried by the piles to the total load were quite stable, i.e., 0.67 for the former and 0.43 for the latter at the end of the observation.

Therefore, no significant changes in foundation settlement or load sharing were observed after the earthquake.

7. CONCLUSIONS

The settlement and the load sharing behaviour of a piled raft supporting a low-rise building was investigated by monitoring the soil-foundation system. During the monitoring period, 44 months after the end of the construction, the 2011 off the Pacific coast of Tohoku Earthquake struck the site of the building. Through the investigation before and after the earthquake, the following conclusions can be drawn:

1) The measured foundation settlement near the centre of the raft reached 21 mm just before the earthquake. The ratios of the load carried by the piles to the effective structure load in the tributary area were estimated to be 0.85 in Pile P1 and 0.67 in Pile P2.
2) The foundation settlement increased by 4 mm from the pre-earthquake value to 25 mm 28 days after the earthquake. The ratio of the load carried by the piles to the effective load in the tributary area of Pile P1 decreased only slightly from 0.85 to 0.82 and that of Pile P2 decreased slightly from 0.67 to 0.57. Therefore, no significant changes in foundation settlement or load sharing were observed after the strong seismic motion. Thereafter, the behaviour of the settlement and the load sharing was found to be quite stable.

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