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Long-term Behaviour of Piled Raft with DMW Grid on Reclaimed Land

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ABSTRACT: This paper offers a case history of a friction piled raft, supporting a four-story parking garage on reclaimed land. The subsoil consists of filled sand and alluvial loose sand which have the potential for liquefaction. Hence, grid-form cement deep mixing walls were employed as a countermeasure of liquefaction with the piled raft. Below the sand layers, there are very-soft to medium alluvial clay layers, which are normally consolidated or under-consolidated, and the depth of the dense sand layer changes markedly near the center of the site. To reduce the differential settlement due to consolidation of the clay, 152 friction piles of different length were employed. To corroborate the foundation design, field monitoring on the foundation settlement and the load sharing between the piles and the raft was performed. The measured settlements and the maximum angular rotation of the raft about 12 years after the end of the construction were within acceptable limits. Furthermore, at the time of the 2011 off the Pacific coast of Tohoku Earthquake, no significant change in effective contact pressure between the raft and the unimproved sand was observed after the event, which confirms that the effectiveness of the grid-form DMWs as a countermeasure of liquefaction.

KEYWORDS: Piled raft, Grid-form DMWs, Consolidation settlement, Monitoring, Liquefaction, The 2011 Tohoku Earthquake

1. INTRODUCTION

This paper offers a case history of a friction piled raft combined with grid-form cement deep mixing walls (DMWs), supporting a four-story building on loose sand underlain by thick soft clay layers. To confirm the validity of the foundation design, field monitoring on the foundation settlement and the load sharing between the piles and the raft was performed. Performance of the friction piled raft at the time of the 2011 off the Pacific coast of Tohoku Earthquake is also discussed.

2. SOIL CONDITIONS AND FOUNDATION DESIGN

2.1 Building and soil conditions

A four-story parking garage is located in Urayasu City, Chiba Prefecture (Uchida et al., 2012). The site is in reclaimed land where the reclamation work was ended in 1975. Figure 1 shows the depth distribution of the soft sediments in the city where buried valleys exist (Tokimatsu et al., 2012). The depths of the soft sediments are more than 60 m.

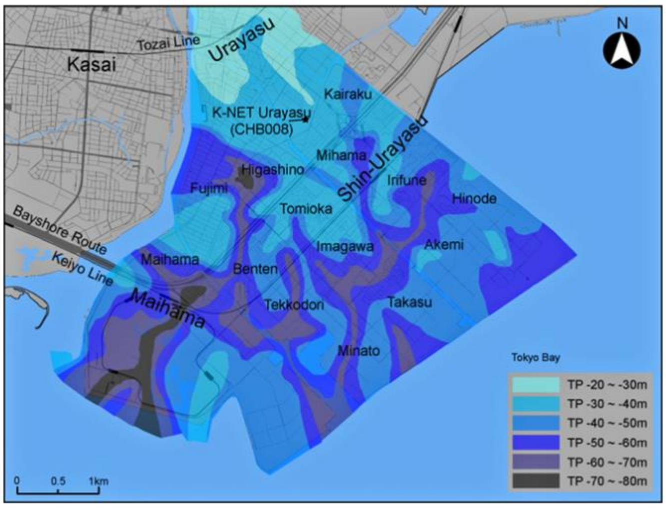


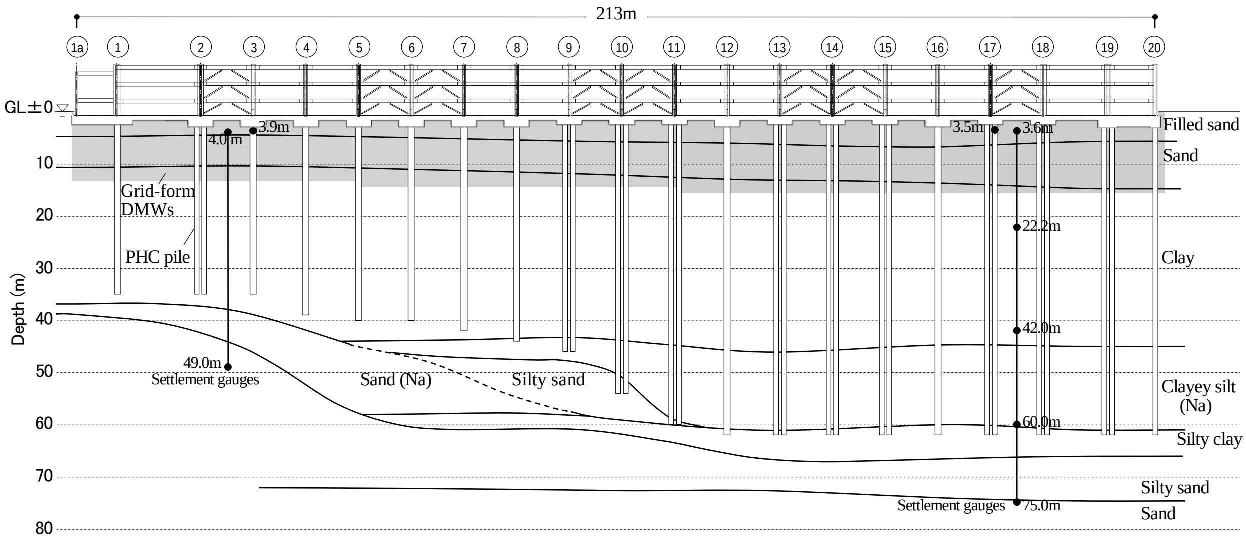
Figure 1 Map showing thickness of alluvial in Urayasu City (after Tokimatsu et al., 2012)

Figure 2 shows the building and foundation with the soil stratification. The four-story building measures 213 m by 71 m in plan; it was completed in 2006. The structure consists of steel reinforced concrete columns and steel beams. The average load per unit area of the raft was 45 kPa (which includes the live load of about 20 kPa). The raft consists of pile caps and mat slab. The foundation level of the pile cap is at a depth of 2.4 m, and partly at a depth of 2.0 m, while that of the mat slab (0.45 m in thickness) is at a depth of 1.2 m.

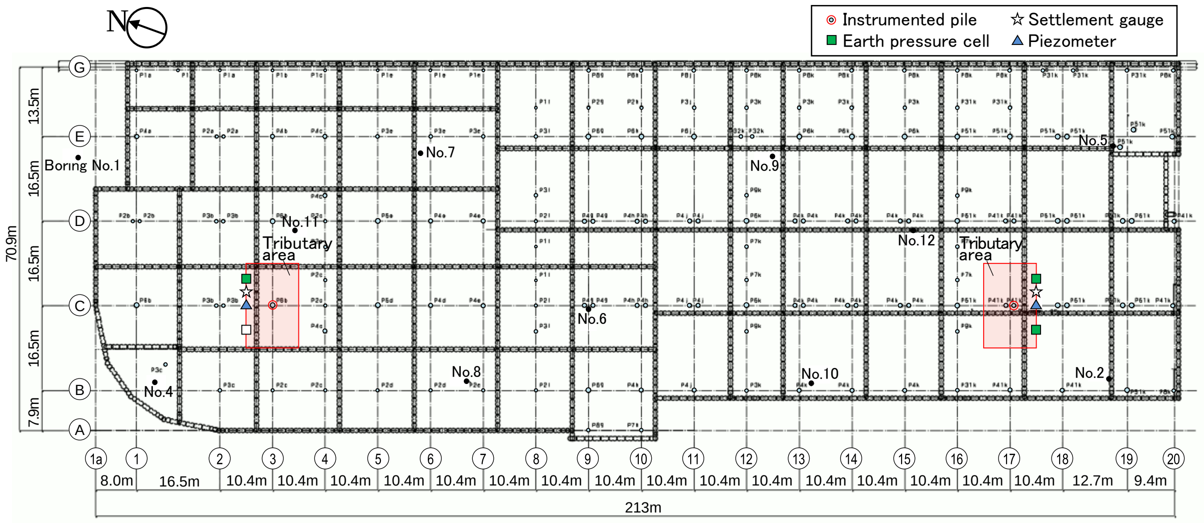
The soil stratification was estimated using 14 soil boring logs obtained in the site. Typical borehole logs over the north-south direction are shown in Figure 3. The soil profile down to a depth of 10-15 m from the ground surface consists of filled sand and alluvial loose sand. The ground water table appears around 1.5 m below the ground surface. Below the depth of 16 m, there are very-soft to medium alluvial clay layers which are normally consolidated or underconsolidated. The thickness of the alluvial clay layers changes markedly in the north-south direction near the center of the site since the buried valley exist below the site. Below the depth of around 60 m, there is a diluvial medium to stiff silty clay layer. The diluvial dense sandy layers appear at a depth of about 39 m in the northern part and 66 m in the southern part.

2.2 Foundation design

A liquefaction assessment made with a method specified in the Architectural Institute of Japan (2001) using a peak ground acceleration (PGA) of 2.0 m/s^2 indicated that the filled sand and alluvial sand had the potential for liquefaction. Hence, grid-form DMWs were employed as a countermeasure of soil liquefaction with piled raft foundation, as in the case histories reported by Yamashita et al. (2011, 2012, 2013, 2016). Figure 4 shows a schematic of the grid-form DMWs. The DMWs were constructed by the Cement Deep Mixing Machine equipped with two mixing shafts, and the spacing of the mixing shafts was 0.8 m for the mixing blades with a diameter of 1.0 m. Typical spacing of the element walls was 15 to17 m, relatively large, and the area replacement ratio (area of the DMWs in plan divided by the total area) was approximately 0.12, as shown in Figure 2(b). The specifications of grid-form DMWs were determined based on and the simplified method proposed by Taya et al. (2008). The design standard strength of the stabilized soil was 1.8 MPa. The bottom depth of the DMWs was set at 13 to 15 m depending on the depth to the alluvial sand.



(a) Profile of building and foundation



(b) Foundation plan

Figure 2 Building and foundation with soil stratification

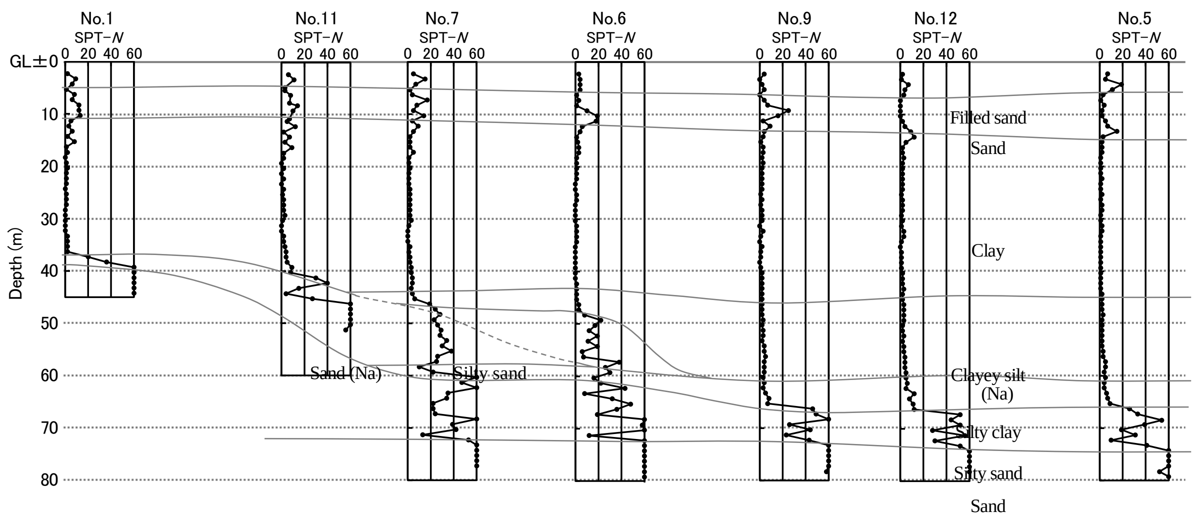
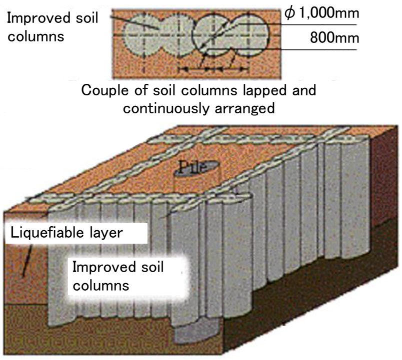
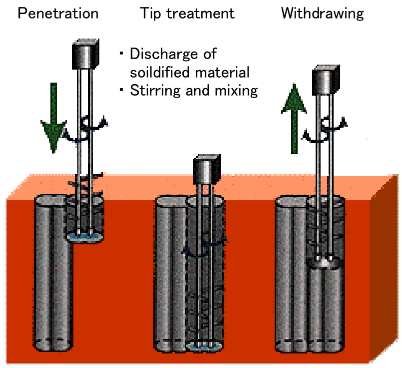


Figure 3 SPT-*N* values in typical borehole logs



(a) Grid-form DMWs



(b) Construction procedure (2-axle type)

Figure 4 Schematic of grid-form cement deep mixing walls

For the foundation design, if conventional pile foundation were used, the pile toe should be embedded in the very dense sand below a depth of about 75 m in the southern part. In addition, since significant residual subsidence due to the consolidation of the alluvial clayey layers was predicted (approximately 0.3 m), the piles should have been designed safely against large drag force due to the negative skin friction. Therefore, considering that the load of the structure is relatively small and a use of the steel–framed structure is a parking lot in which the average and differential settlement could be allowed to some degree, friction piles were employed as a cost-effective solution. To reduce the settlement due to consolidation of the clayey layers to an acceptable level, 152 PHC piles (0.5-1.0 m in diameter) were employed. The length of the piles was determined based on the estimated soil stratification. Thus, the pile toe in the northern part is at depths of 35-42 m from the ground surface ad in the southern part at a depth of 62 m while the pile toe in the central part is between depths of 44 and 60 m. It should be noted that a piled raft foundation, in which the raft resistance was considered in the design, was employed in the northern part. On the other hand, in the southern part, the foundation was designed substantially as a conventional friction pile foundation because long-term resistance of the raft could not be relied on due to the residual subsidence. The layout of the piles and the grid-form DMWs is shown in Figure 2(b).

3. INSTRUMENTATION

Field monitoring on the foundation settlement and the load sharing between the piles and the raft were performed 12 years after the end of the construction, October 2006 (denoted as E.O.C., hereafter). The locations of the monitoring devices are shown in Figure 2(b). Pile 3C, at the intersection point of Streets 3 and C, (35 m long, 1.0 m in diameter) and Pile 17C (60 m long, 0.8 m in diameter), were installed with a couple of LVDT-type strain gauge at the pile head, at a depth of 3.9 and 3.5 m, respectively. A pair of earth pressure cell together with a piezometer was installed in the filled sand (consisting of fine sand and silty sand) beneath the mat slab in the tributary area of columns 3C and 17C, as shown in Figure 2(b). The vertical ground displacements, at a depth of 4.0 m near 3C and at four depths of 3.6, 22.2, 42.0 and 60.0 m near 17C, were measured using differential settlement gauges as shown in Figure 2(a). The reference points were set in the very dense sand, at depths of 49.0 m and 75.0 m. These measurements began in March 2006. The settlements of the first floor were also measured at the selected column points using an optical level. The optical level measurements started in May 2006.

4. RESULTS OF LONG-TERM MONITORING

4.1 Observations related to the 2011 Tohoku Earthquake

The 2011 off the Pacific coast of Tohoku Earthquake, with an estimated magnitude of M_w= 9.0, struck East Japan on March 11, 2011. The distance from the epicentre to the building site was about 380 km. The peak ground surface acceleration of 1.57 m/s^2 in the east-west direction was recorded at K-NET Urayasu, and extensive soil liquefaction occurred in reclaimed land of Urayasu City where many sand boils, ground settlements as well as settlements and tilting of buildings and houses on spread foundations etc. were observed (Tokimatsu et al., 2012; Yasuda et al., 2012).

Photo 1 shows a view of the building two days after the earthquake. Note that the ground surface around the building had been temporally restored when Photo 1 was taken. Liquefaction induced settlement is seen in the ground surface adjacent to the building.

Photo 1 View of ground around the building two days after the event (March 13, 2011)

4.2 Settlement and load sharing between piles and raft

Figure 5 shows the development of the measured vertical ground displacements just below the mat slab. The ground displacement near 17C was significantly greater than that near 3C. The consolidation settlement near 17C was much greater than that near 3C due to the difference in thickness of the alluvial clayey layers. The ground displacement at 4.0 m depth near 3C was 16 mm in November, 2010 (four months before the seismic event), and increased notably to 23 mm in May 2011. It is likely that this incremental settlement of 7 mm was caused primarily by the settlement of the friction piles in a piled raft system subjected to cyclic axial loading due to the rotational moment from the superstructure during the event. Thereafter, the settlement was almost stable. On the other hand, the ground displacement at 3.6 m depth near 17C increased gradually after E.O.C. due to the continuing consolidation settlement. The settlement was 41 mm before the event, in November 2010, and increased to 63 mm in May 2014 at the end of the observation. Unfortunately, no data were obtained from the settlement gauges near 17C after that time. In contrast to the settlement near 3C, almost no increase in settlement was observed to have occurred due to the event.

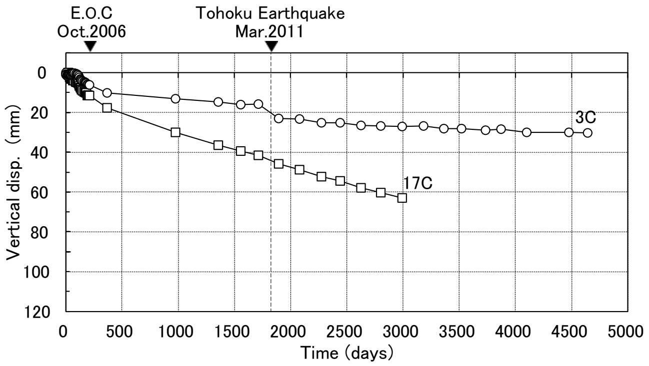


Figure 5 Measured vertical ground displacements just below mat slab vs. time

Figure 6 shows the development of the measured vertical ground displacement of each soil layer near 17C. At the end of the observation (May 2014), the largest displacement of 43 mm mostly occurred in the alluvial clayey silt layer between depths of 42 and 60 m. This layer, called the Nanagochi layer denoted as Na in Figure 2(a) (which was deposited 10000-20000 years ago and), was found to be undergoing consolidation. The consolidation settlement was likely to be caused by the structure load via the piles in addition to the residual land subsidence. In the silty clay and silty sand between the depths of 60 and 75 m, a consolidation settlement of 13 mm occurred that was generated almost entirely in the silty clay between the depths of 60 and 65 m. In contrast, the displacement of the alluvial clay layer between depths of 22.2 and 42 m was relatively small (7 mm at the end of the observation), and that of the filled sand and alluvial layers between depths of 3.6 and 22.2 m was also small, 7 mm. This indicates that the consolidation settlement of the clayey layer between depths of around 15 and 42 m have almost completed. Note that small settlement began to occur in the clayey layer after the seismic event.

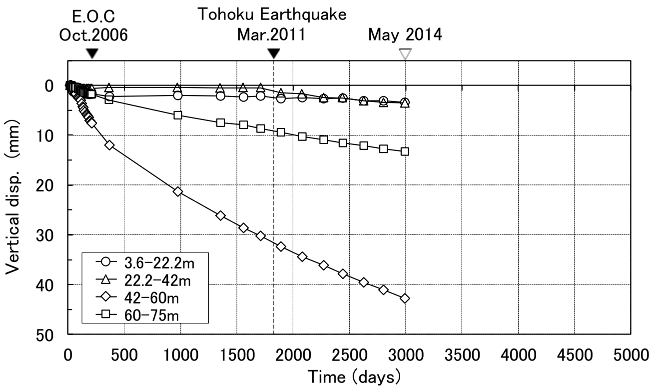
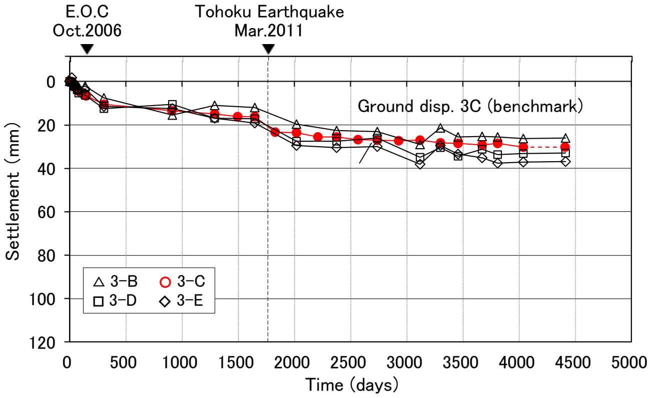
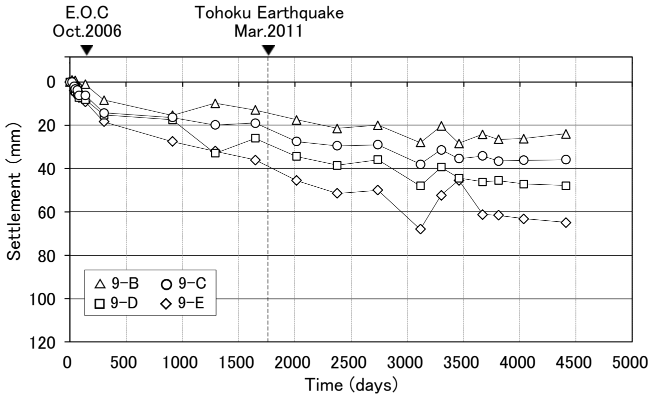


Figure 6 Measured vertical ground displacements of each soil layer vs. time (near 17C)

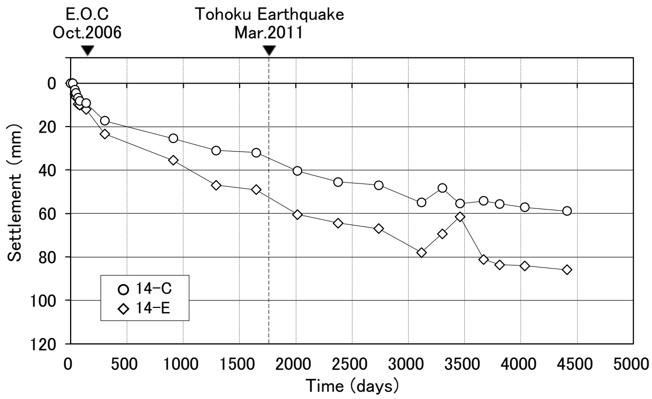
Figure 7 shows the development of the measured settlements of the first floor at the selected column points on Streets 3, 9, 14 and 17 from the beginning of the construction to June 2018 (end of the observation). A benchmark for the optical level measurement was set to column 3C, at which the settlement of the first floor was assumed to be equal to the vertical ground displacement at the depth of 4.0 m near 3C. The settlement of the benchmark after the casting of the mat slab is indicated by a red line in Figure 7(a). It is seen that the settlements become greater from the north to the south in



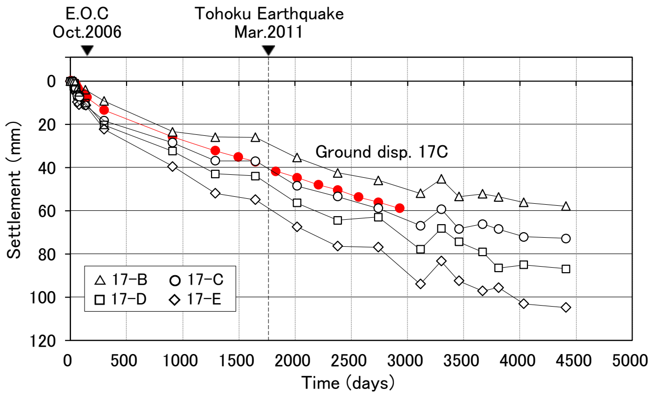
(a) Street 3



(b) Street 9



(c) Street 14



(d) Street 17

Figure 7 Measured settlements of 1st floor vs. time accordance of the thickness of the alluvial clayey layers. In addition, the settlements become greater from the west to the east. As is seen in Figure 7(d), the settlement of the first floor at 17C was consistent with the vertical ground displacement near 17C (indicated by a red line) before and after the 2011 earthquake. This suggests that no significant settlement of the first floor had occurred by the seismic load. At the end of the observation, the settlements were varied from 19 mm (1C) to 105 mm (17E).

It appears that the rate of increments in settlement on all the streets has been decreased in recent years. Future settlement was extrapolated applying a hyperbolic curve to the measured data. An example of the settlement versus time curve at 17E with the measured data is shown in Figure 8, where the initialization point of the hyperbolic curve corresponds to March 2007, five months after E.O.C. The settlement reaches about 190 mm in 2056 (50 years after E.O.C.), hence the residual settlement is 85 mm.

Figure 9 shows the measured settlement profiles of the first-floor after E.O.C. There was a major increase in settlement along Streets C and E from the north to the south, and also along Streets 9 and 17 from the west to the east. It is worth noting that the increase in settlement was almost linear without abrupt change, even near the center to the south where the thickness of the clayey layers changes drastically. This occurs probably because the potential differential settlements were mitigated by the bending or shear rigidity of the DMWs (which may behave as a simple beam beneath the raft). At the end of the observation, in June 2018, the maximum angular rotation was 1/1050 radian in the east-west direction (B-E along Street 17) and about 1/1690 radian in the north-south direction (9-17 along Street E), which were at present fairly less than an acceptable value of 1/500 radian in the design.

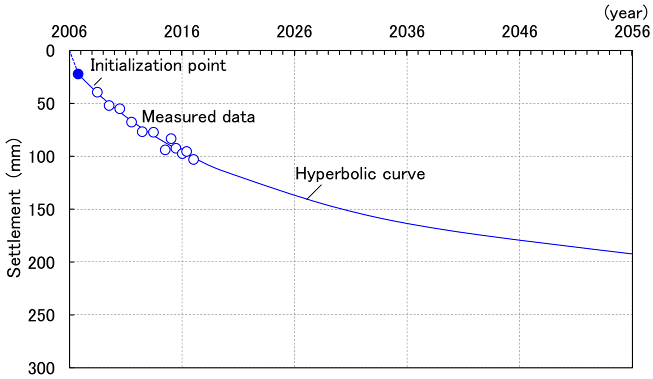


Figure 8 Predicted settlement vs. time curve with measured data (17E)

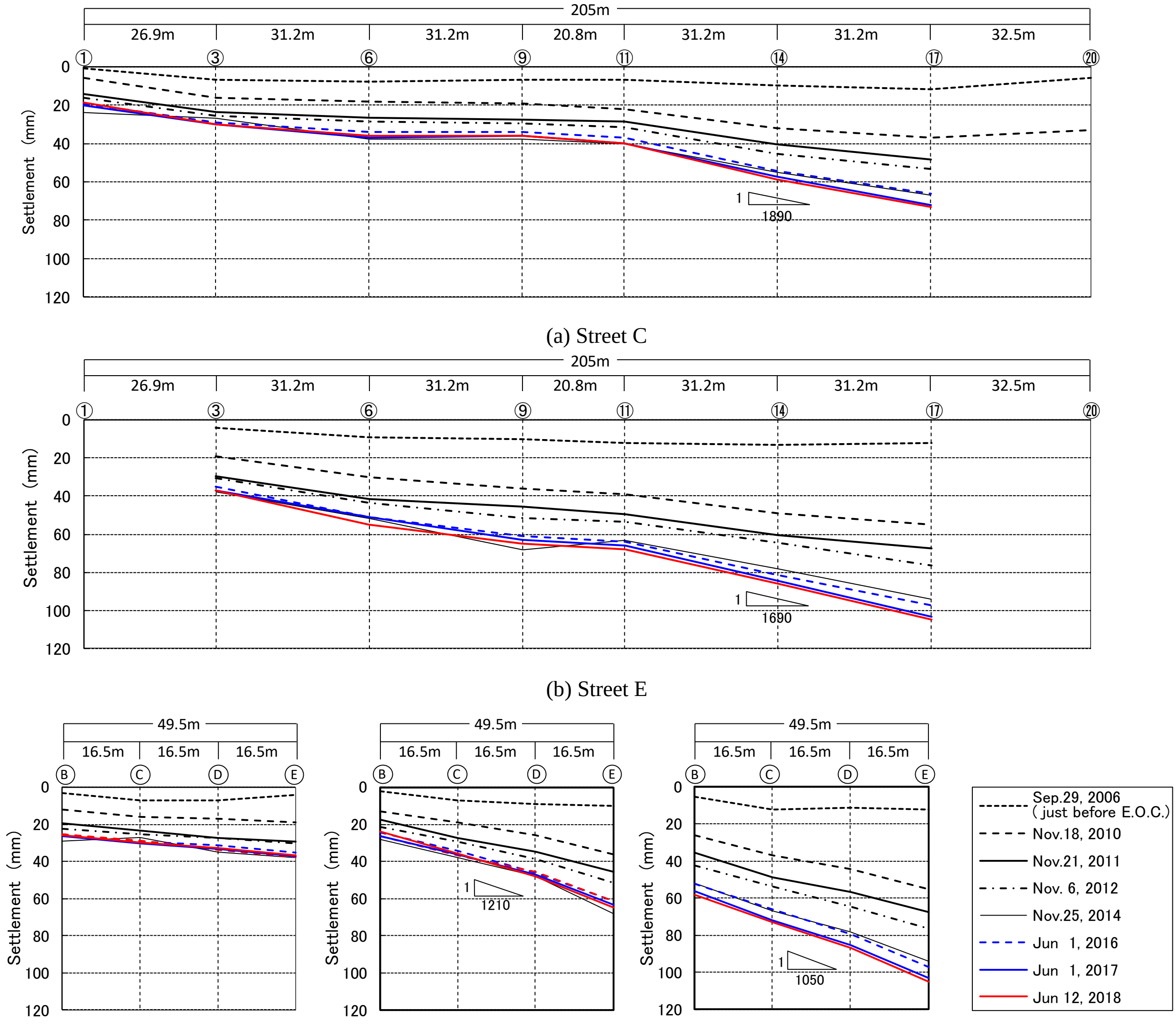


Figure 9 Settlement profiles at 1st floor

Figure 10 shows the average contact pressure from a pair of earth pressure cell and the porewater pressure beneath the mat slab near 3C and 17C. The contact pressure measured after E.O.C. was 5-14 kPa. The porewater pressure beneath the mat slab near 17C was ranged from 2 to 6 kPa. Unfortunately, the piezometer near 3C ceased functioning soon after the installation. At the time of the 2011 earthquake, no significant change in contact pressure between the raft and the unimproved soil was observed after the event. This confirms that the effectiveness of the grid-form DMWs as a countermeasure of soil liquefaction against seismic motion with PGA of 2.0 m/s^2 caused by the magnitude-9.0 earthquake (Uchida et Figure 11 shows the measured axial load at the pile head of Piles 3C and 17C. The axial load of Pile 3C, which was 3.0 to 3.9 MN after E.O.C., decreased significantly after the 2011 earthquake. By considering that no increase in contact pressure near 3C was observed after the event as shown in Figure 10, it is likely that considerable load transfer from the piles to the DMWs occurred due to the settlement of friction piles. The ratio of the load carried by the pile to the design load in the tributary area of column 3C (7.8 MN) reduced from 0.50 to 0.27 after the earthquake, however, the raft carried significant load. Thus, it was found that the load sharing behaviour was consistent with design concept of piled raft foundation. On the other hand, the axial load of Pile 17C increased gradually after E.O.C., and showed almost no change at the time of the 2011 earthquake. The ratio of the load carried by the piles (assumed to be twice the measured load of Pile 17C) to the design load in the tributary area of column 17C (8.0 MN) decreased only slightly from 0.70 to 0.67. The ratio was significantly larger than the ratio in Pile 3C, and the behaviour was similar to that of pile foundation. Subsequently, the axial loads of both piles increased gradually, and the ratio of the load carried by the Piles 3C and 17C to the design load reached 0.34 and 0.77, respectively, at the end of the observation (June 2018).

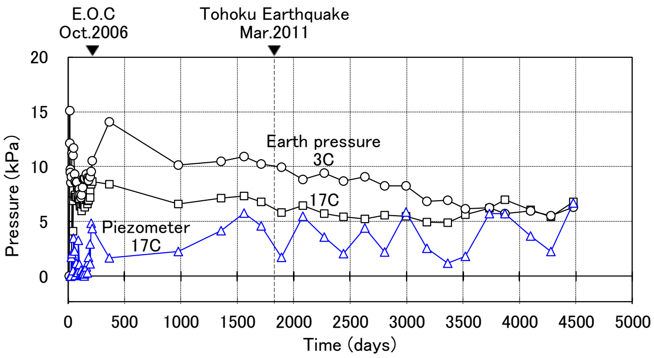


Figure 10 Measured contact pressure and pore-water pressure vs. time

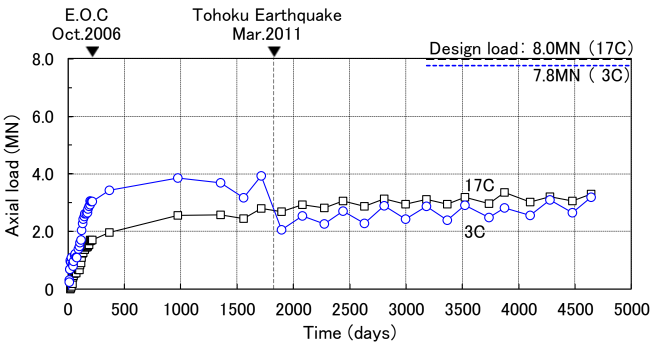


Figure 11 Measured axial loads at pile head vs. time

Fellenius (2016) suggested that an assessment of load sharing between piles and raft in terms of strain developed in the piles and in the soil underneath the raft is useful. Assessing the load sharing in terms of the measured strain would be a subject for future research.

5. CONCLUSION

The long-term behaviour of a friction piled raft foundation combined with the grid-form DMWs on the reclaimed land was investigated by monitoring the soil-foundation-structure system. Through the investigations, the following conclusions can be drawn:

(1) There was a major increase in settlement of the raft from the north to the south, and also from the west to the east. However, the increase in settlement was almost linear, even near the center to the south where the thickness of the clayey layers changes drastically. This occurs probably because the potential differential settlements were mitigated by the bending or shear rigidity of the DMWs.

(2) The measured settlements of the first floor about 12 years after E.O.C were 19-105 mm, and the maximum angular rotation of the raft was about 1/1050 radian which was less than an acceptable value in the design (1/500 radian).

(3) At the time of the 2011 off the Pacific coast of Tohoku Earthquake, no significant change in contact pressure between the raft and the unimproved soil was observed after the event. This confirms that the effectiveness of the grid-form DMWs as a countermeasure of soil liquefaction against seismic motion with PGA of 2.0 m/s^2.

Consequently, it was found that the friction piled raft system showed a good performance in grounds consisting of liquefiable sand underlain by thick soft clay layers.

6. ACKNOWLEDGEMENT

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