Long-term behavior of friction piled raft with grid-form DMWs in loose sand underlain by thick clay layers in reclaimed land

Comportement à long terme de frottement empilés radeau avec grille-forme DMWs dans le sable meuble reposant sur des couches d'argile épais en terres récupérées

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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 underconsolidated 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 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. The measured settlements and the maximum angular rotation of the raft about ten years after the end of the construction were less than an acceptable value. 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.

RÉSUMÉ : Nous rapportons le cas d'un radier sur pilotis à friction qui supporte un parking de 3 étages. Le sous-sol est constitué de sable rapporté et de sable alluvial meuble qui ont tendance à se liquéfier. De ce fait, on a mis en oeuvre des murs de ciment composite profonds en réseau, avec un radier sur pilotis, pour contrer la liquéfaction. Sous les couches de sable, on trouve des couches d'argile alluviale de densité moyenne ou très meuble, normalement compactées ou compactées par-dessous et la densité de la couche de sable compact change remarquablement. Pour obtenir une réduction du tassement différentiel, par le compactage de la couche d'argile, on a employé 152 pieux à friction, de longueurs différentes. Pour l'assessement de la validité des fondations, on a effectué des observations sur le terrain du tassement des fondations et de la répartition des forces entre les pieux et le radier. Les tassements mesurés et l'angle maximum de rotation du radier environ 10 ans après la construction de l'ouvrage sont des valeurs négligeables. Par ailleurs suite au Grand Séisme du Tohoku en 2011, aucun changement notable, suite à l' événement. ne fut observé dans les pressions de contact effectives entre le radier et le sable. Ainsi nous confirmons l'efficacité du réseau comme contre-mesure de la liquéfaction.

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 BUILDINGS AND SOIL CONDITIONS

A four-story parking garage is located in Urayasu City, Chiba Prefecture (Uchida et al., 2012). Figure 1 shows a schematic view of the building and the foundation with soil stratification in the site. The site is in reclaimed land where the reclamation work was ended in 1975 (Tokimatsu et al., 2012). The soil stratification was estimated using totally 14 soil boring logs in the site. 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 depth of the alluvial clay layers changes markedly in the north-south direction near the center of the site since buried valleys exist below the site. The diluvial dense sandy layers appear at a depth of about 39 m in the northern part and about 66 m in the southern part.

Figure 2 shows the foundation plan. The building measuring 213 m by 71 m in plan is a steel-frame structure; it was completed in September, 2006. The average load per unit area of the raft was 45 kPa. The raft consists of pile caps and 0.45-m thick 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 is at a depth of 1.2 m.

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 (Yamashita et al., 2012; Yamashita et al., 2016). Typical center-to-center spacing of element walls was 15 to17 m, relatively large, and the area replacement ratio (ratio of area of walls to total plan area) was 0.12. The specifications of grid-form DMWs were determined based on and the simplified method (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 that of the alluvial sand.

In foundation design, considering that the structure load is relatively small and the dense sand layers appear below about 40 m depth, friction piles were adopted as cost-effective foundation based on preliminary study. To reduce the average and differential settlement due to consolidation of the clay layers to an acceptable level, 152 friction piles (PHC piles, 0.5-1.0 m in diameter) were employed. The length of the piles was determined based on the estimated soil stratification. Namely, the pile toe in the northern part is at depths of 37-40 m from the ground surface and that in the southern part is at a depth of 62 m while the pile toe in the central part is between depths of 42 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, a conventional friction pile foundation was used in the southern

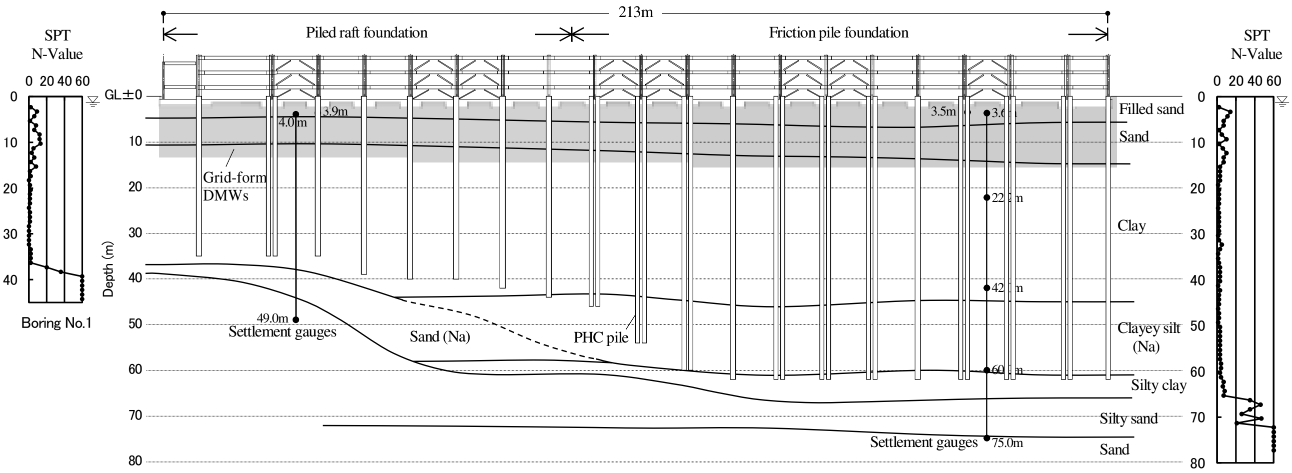


Figure 1. Schematic view of building and foundation with soil profiles

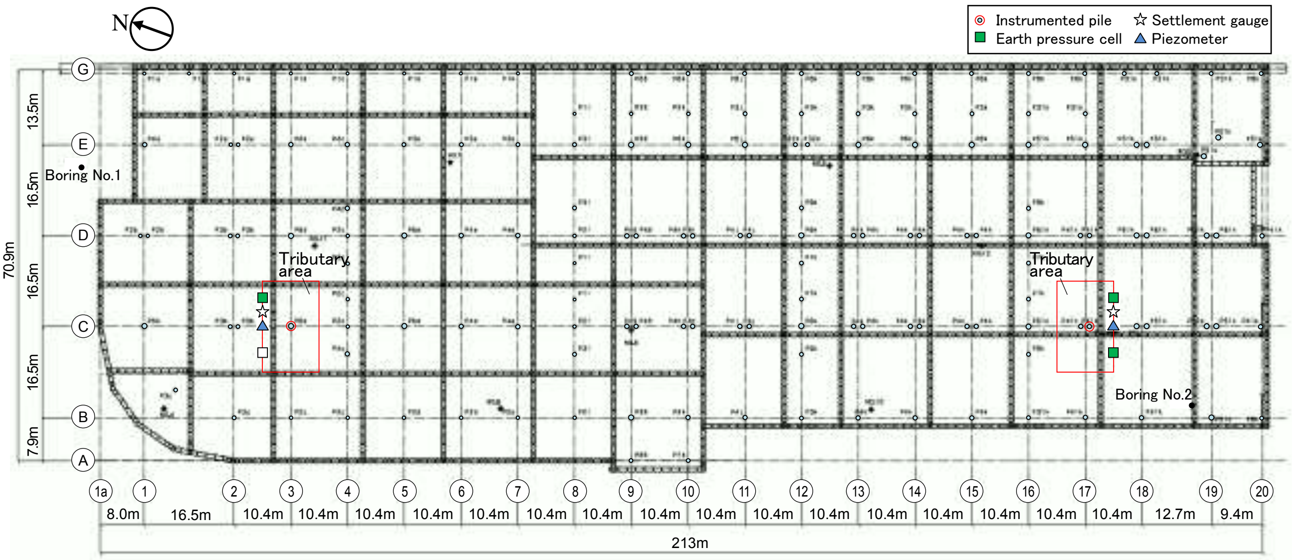


Figure 2. Layout of piles and grid-form DMWs with locations of monitoring devices

part because long-term resistance of the raft could not be expected due to the considerable residual subsidence due to consolidation settlement of the alluvial clayey layers (which was predicted about 0.3 m). The layout of the piles and the grid-form DMWs is shown in Fig. 2.

3 INSTRUMENTATION

Field monitoring on the foundation settlement and the load sharing between the piles and the raft were performed ten years after the end of the construction (denoted as E.O.C., October 2006). The locations of the monitoring devices are shown in Figs. 1 and 2. Pile 3C (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 unimproved soil beneath the mat slab in the tributary area of columns 3C and 17C, as shown in Fig. 2. 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 Fig. 1. 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 selected column points using an optical level. The optical level measurements started in May 2006.

Photo 1. View of ground around the building (March 13, 2011)

4 RESULTS OF LONG-TERM MONITORING

4.1 *The 2011 off the Pacific coast of Tohoku Earthquake*

On March 11, 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. Liquefaction induced settlement is seen in ground surface adjacent to the building, though the ground surface around the building had been temporally restored when Photo 1 was taken.

4.2 *Settlement and load sharing between piles and raft*

Figure 3 shows the development of the measured vertical ground displacements just below the mat slab. The ground displacement near 3C was 16 mm in November, 2010, and increased notably to 23 mm in May, 2011. It is likely that the incremental settlement of 7 mm was caused primarily by settlement of the friction piles in piled raft system due to the rotational moment of the superstructure during the earthquake. Thereafter, the settlement was almost stable. On the other hand, the ground displacement near 17C increased gradually after E.O.C. due to the consolidation settlement of the reclaimed ground. The settlement was 41 mm before the event, in November, 2010, and increased to 63 mm in May, 2014. 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 after the event.

Figure 4 shows the measured vertical ground displacement of each soil layer versus time near 17C. The largest vertical displacement of 43 mm occurred in a soil layer between depths of 42 and 60 m in May, 2014. This alluvial soil layer is called the Nanagochi layer (Na) which was deposited 10000-20000 years ago (Yasuda et al, 2012). The ground displacement of 13 mm occurred in the diluvial soil layers between depths of 60 and 75 m. In contrast, the ground displacement of the filled sand and alluvial layers between depths of 3.6 and 42 m was relatively small, 7 mm.

Figure 5 shows the development of the measured settlements of the first floor at the selected column points. A benchmark for the optical level measurement was set to column 3C, at which the settlement was assumed to be equal to the vertical ground displacement at the depth of 4.0 m near 3C after the casting of the mat slab (indicated by a red line in Fig. 5 (a)). The settlement of the first floor at 17C was almost consistent with the vertical ground displacement near 17C shown in Fig. 5(b) before and after the 2011 earthquake. This suggests that almost no increase in settlement of the first floor in the southern part occurred due to the seismic load. It is seen that the rate of the measured settlements on Street 17 decreased in recent years. Hence, the residual settlements were calculated using Asaoka method (Asaoka, 1978), and estimated to be less than 60 mm.

Figure 6 shows the settlement profiles of the first floor after E.O.C. The settlements of the first floor reached 22 mm (at 1C) to 96 mm (at 17E) in October, 2016. At that time, the maximum angular rotation was 1/1367 radian in the east-west direction (B-E along Street 17) and about 1/2600 radian in the north-south direction (9-17 along Street C), which were less than an acceptable value of 1/500 radian in Figure 7 shows the average contact pressure from a pair of earth pressure cell and the porewater pressure beneath the mat slab. The contact pressure after E.O.C. was seen clearly, 5-14 kPa, from both earth pressure cells. The porewater pressure beneath the mat slab near 17C was varied from 2 to 6 kPa. Note that no data were obtained from the piezometer near 3C. 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 al., 2012; Tokimatsu et al., 2012).

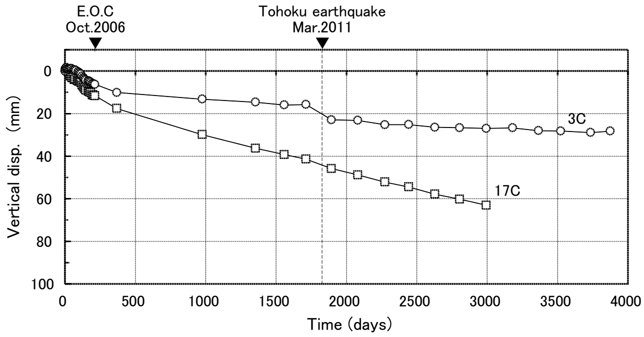


Figure 3. Measured vertical ground displacements just below mat slab vs. time

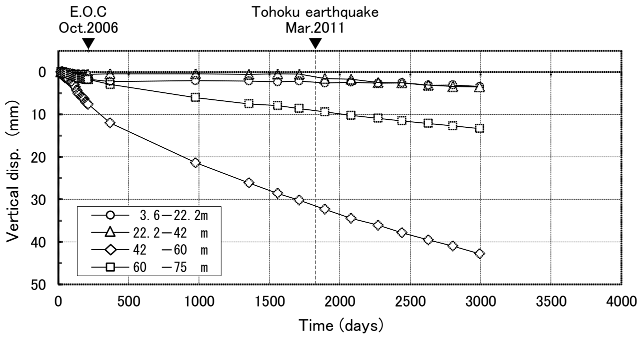
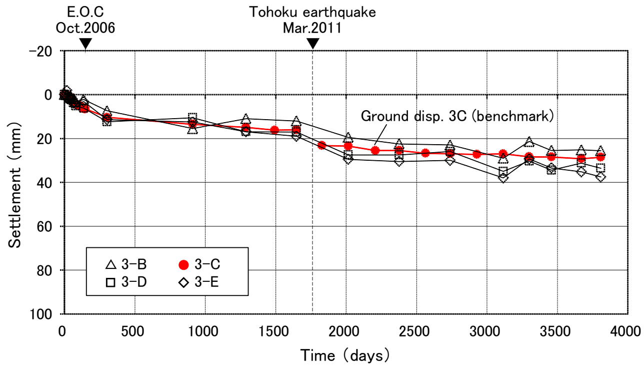
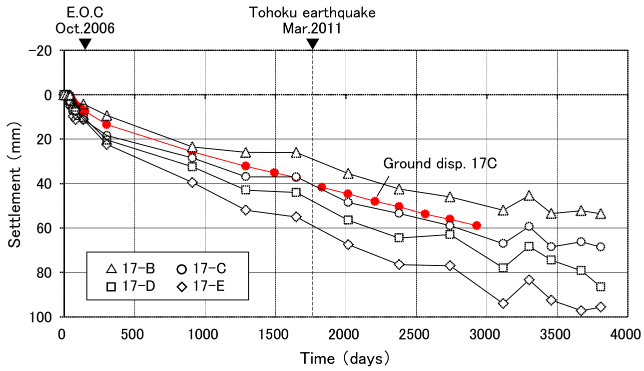


Figure 4. Measured vertical ground displacements of each soil layer vs. time (near 17C)



(a) Street 3



(b) Street 17

Figure 5. Measured settlements of the 1st floor vs. time

Figure 8 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

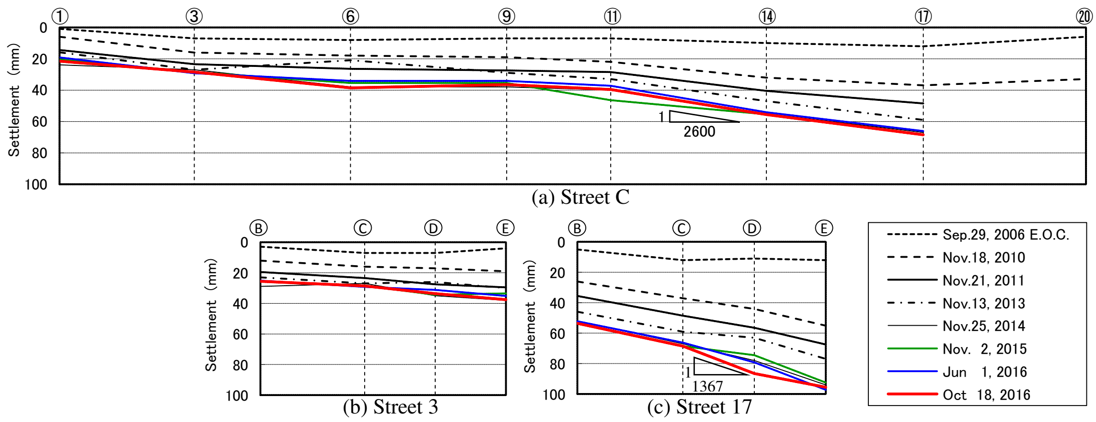


Figure 6. Time dependent settlement profiles of the 1st floor

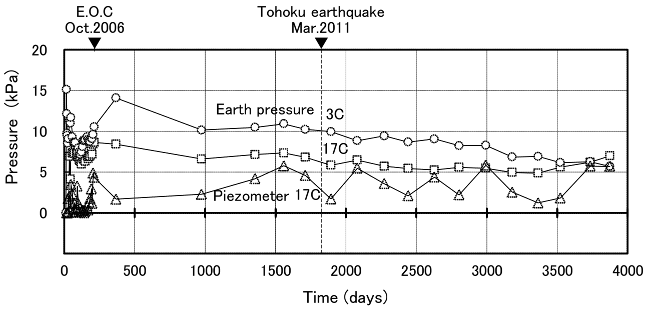


Figure 7. Measured contact pressure and porewater pressure vs. time

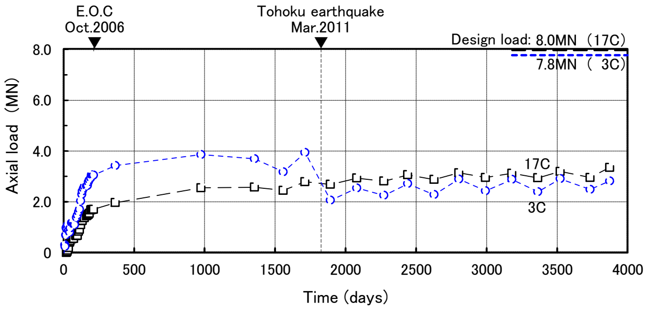


Figure 8. Measured axial loads at pile head vs. time

near 3C was observed after the event as shown in Fig. 7, it is likely that significant 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 considerably larger than that 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 in the tributary area of columns 3C and 17C reached 0.37 and 0.80, respectively, in October 2016.

5 SUMMARY

The long-term behavior of a friction piled raft foundation combined with the grid-form DMWs in the reclaimed land was investigated by monitoring the soil-foundation-structure system. The measured settlements of the first floor about ten years after E.O.C were 21-93 mm, and the maximum angular rotation of the raft was about 1/1300 radian, which was less than an acceptable value of 1/500 in design. 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.

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