The paper was published in the proceedings of the 7th International Conference on Earthquake Geotechnical Engineering and was edited by Francesco Silvestri, Nicola Moraci and Susanna Antonielli. The conference was held in Rome, Italy, 17 – 20 June 2019.

Earthquake Geotechnical Engineering for Protection and Development of Environment and Constructions – Silvestri & Moraci (Eds)
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Development of hybrid foundation to mitigate the liquefaction-induced settlement of shallow foundation

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ABSTRACT: A series of dynamic centrifuge experiments are carried out to investigate the performance of hybrid foundation to mitigate the liquefaction-induced effects on shallow foundation. The proposed hybrid foundation is a combination of gravel drainage system and friction piles having spiral blades at the bottom employed under the footing as a hybrid mitigation technique. The evolution of excess pore water pressure (EPWP); specifically, in the vicinity of the foundation-structure system, dominatingly influence the settlement mechanism of shallow foundation during the seismic event. Centrifuge test results exhibit that the presence of gravel drainage is able to evade the post-liquefaction settlement of shallow foundation by means of rapid dissipation of EPWP. In addition, friction piles which are provided with gravel drainage system, rendered desirable functioning of minimizing the tilting/differential settlement of shallow foundation. It is found that the proposed hybrid foundation can serve the desired purpose of reducing overall liquefaction-induced effects on shallow foundation resting on liquefiable ground.

1 INTRODUCTION

Liquefaction is one of the most complex phenomena in geotechnical earthquake engineering. Devastating effects of liquefaction, sprang the attention of geotechnical engineers within a three-month period in 1964, because of the Good Friday earthquake in Alaska followed by the Niigata earthquake in Japan (Kramer, 1996). Liquefaction favorably occurs in saturated loose cohesionless soil during dynamic/cyclic loading. During liquefaction, soil loses its shear strength due to excessive build-up of pore water pressure leading to ground failure and sometimes even collapse of associated structures.

Soil liquefaction and related ground failures extensively studied by many researchers (Ishihara et al., 1992). Liquefaction has caused damage to build environment to a great extent. For instance, a significant area of Christchurch city in New Zealand was devastated by soil liquefaction during the 2011 earthquake in terms of structural settlement, tilting, and lateral spreading (Green et al., 2011). Most of the damaged buildings were two to four stories founded on shallow foundations and relatively thick and uniform deposits of clean sand in the 1964 Niigata and 1990 Luzon (Philippines) Earthquakes as reported by Cubrinovski & McCahon, (2012). Many researchers (Liu & Dobry, 1997, & Bray et al., 2000) described the role of liquefaction to the damage of buildings, specifically in the reclaimed land during the 2011 Tohoku earthquake.

The deformation mechanism of shallow foundation resting on the liquefiable ground is a complex phenomenon and presumed to be affected by several factors, e.g., induced cyclic stress, foundation configuration, and initial ground condition shear-induced deformation, development of excess pore water pressure, localized drainage, post-liquefaction reconsolidation, void re-distribution, inertial and kinematic interaction within soil-foundation-structure system (Tokimatsu & Seed, 1987, & Bird & Bommer, 2004).

Employment of any suitable mitigation technique is essential for any site which is prone to seismically induced liquefaction, during the commencement of any project. Existing mitigation techniques performs fairly well in case of a small earthquake, but during moderate or strong ground motion, the mitigation performance is still under a question (Tokimatsu & Seed, 1987, & Bird & Bommer, 2004). Hence, this research is intending to develop a hybrid foundation, which is supposed to perform satisfactorily even during the strong ground motion. The development of proposed hybrid foundation is carried out by performing a series of dynamic centrifuge experiments at 40g using Tokyo Tech Mark III centrifuge lab facility to investigate the intended performance of hybrid foundation to mitigate the liquefaction-induced effects on shallow foundation.

2 CENTRIFUGE EXPERIMENTAL PROGRAM

2.1 *Model ground preparation*

The suitability and effectiveness of the hybrid foundation are evaluated against temporary kind of structures such as Buffer Tank (BT) and Flare Stack (FS) mounted on distinct shallow foundations. BT and FS impose an average bearing pressure of 51.2 kPa and 71.2 kPa respectively at 0.8 m below the surface of the ground in prototype scale as shown in Figure 1 (detailed dimensions are shown in Figure 2). Silica no.3 is used to make the gravel drainage system and the liquefiable model ground is prepared using Toyoura sand (Table 1) with a relative density; of 50% by air pluviation method using sand hopper. The design charts reported by Seed and Booker (1977) in their seminal work and the revised guidelines presented by Bouckovalas et al. (2009) are used to design the gravel drainage system. The target excess pore water pressure ratio (r_u) equal to 0.7 is assumed for strong earthquake (N_{eq}/N_L = 3). For detailed design procedure readers are suggested to refer Seed and Booker (1977). To ensure the consistency of relative density for different model grounds, the sand hopper is precisely calibrated in terms of falling height and pouring rate of Toyoura sand. A flexible laminar container with inner dimensions of 600 x 250 x 438 mm (in model scale) in length, width and height respectively, is used to frame the models. It is to be noted that certain boundary interaction between the structures is inevitable due to limited size of the laminar container.

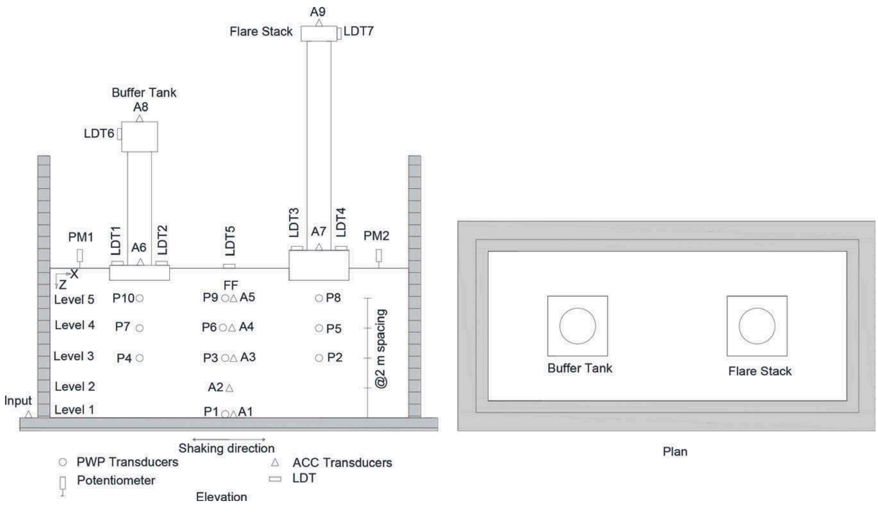


Figure 1. Typical centrifuge model layout

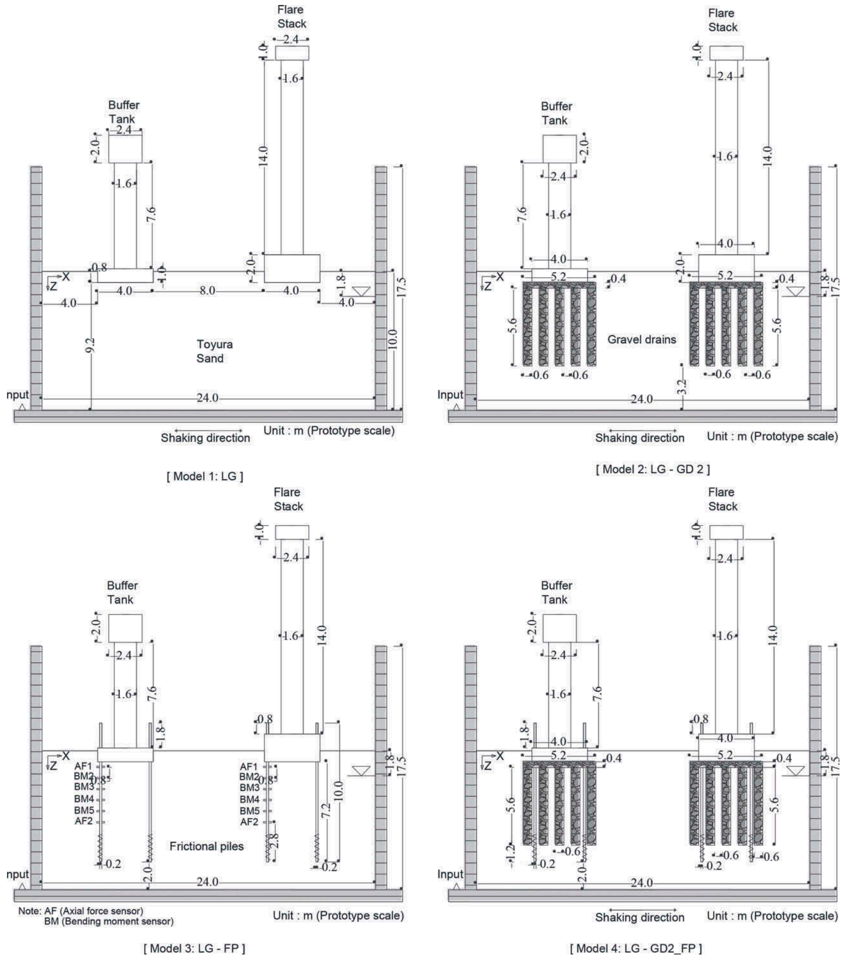


Figure 2. Centrifuge model configurations in prototype scale

Table 1. Index properties of Toyoura sand and silica no. 3

Property	Toyoura sand	Silica no. 3
Specific gravity, G_s	2.65	2.63
D_{50} (mm)	0.19	1.72
D_{10} (mm)	0.14	1.37
Maximum void ratio, e_{max}	0.973	1.009
Minimum void ratio, e_{min}	0.609	0.697
Permeability, k (m/s)	2.00E-04	6.60E-03
Relative density, D_r	50%	30%

The model ground is saturated with the viscous fluid, i.e., a mixture of water and 2 % Metolose (Hydroxypropylmethyl cellulose) by weight of water, to achieve a viscosity about 40 times that of water. The viscous fluid ensured the permeability in prototype scale as mentioned in Table 1. Gravel drainage system is employed during the ground preparation (before saturation) using the special guide frames whereas, the friction piles are installed after the saturation by screw driving method in the respective model grounds as shown in Figure 2.

Table 2. Description of centrifuge models

Test code	Model description	Model ground	Gravel drainage*	Friction piles
Model 1	LG: Liquefiable ground	Toyoura sand (D_r = *52.8 %*)	No	No
Model 2	LG-GD2: Liquefiable ground with gravel drainage	Toyoura sand (D_r = *54.0 %*)	Yes	No
Model 3	LG-FP: Liquefiable ground with friction piles	Toyoura sand (D_r = *53.1 %*)	No	Yes
Model 4	LG-GD2_FP: Liquefiable ground with gravel drainage and friction piles	Toyoura sand (D_r = *55.2 %*)	Yes	Yes

* Group of 5 x 5 gravel drains with 0.6 m diameter of each drain and 0.6 m clear spacing (prototype scale)

2.2 *Testing scheme*

The layout of the physical models is shown in Figure 2. The detail description of centrifuge test conditions is tabulated in Table 2. Initially, liquefaction-induced effects on shallow foundation are investigated in Model 1. As the proposed hybrid foundation consist of gravel drainage and friction piles, the individual performance of the same is evaluated in Models 2, and 3 respectively. Finally, the centrifuge test for Model 4 is performed to evaluate the effectiveness of hybrid foundation.

After model preparation, the instrumented model is placed in the centrifuge and spun at the centrifugal acceleration of 40g. All models (Models 1-4) are tested under the earthquake ground motion recorded at the Hachinohe Port in 1968 Tokachi-Oki Earthquake (NS component) followed by design earthquake motion for highway bridges in Japan (2-I-I-3, NS component) recorded at the ground surface near the New Bansuikyo Bridge, Tochigi during 2011 Tohoku earthquake. Before applying the Tokachi-Oki ground motion, models are subjected to white noise to evaluate the system behavior. Experiment results related to Tokachi-Oki ground motion only, are presented herein for the sake of brevity. Figure 3 depicts the acceleration time histories, Arias intensity, and Fourier spectra of input base motion (simulated wave of Tokachi-Oki) for all the models. It should be noted that the motion for Model 4 is rather larger than the other cases. Base motions shown here are presented after having baseline correction and filtering. Filtering is performed in the frequency domain using the bandpass Butterworth filter with corner frequencies of 0.3Hz and 10 Hz respectively in prototype scale.

3 RESULTS AND DISCUSSION

3.1 *Evolution of pore water pressure*

Excess pore water pressure (EPWP) time histories of different pore pressure transducers (PPTs) at Levels 5 and 3 (Figure 1) for all the Models during Tokachi-Oki ground motion are shown in Figure 4. The ground is said to be liquefied at any depth if the excess pore water pressure ratio-(EPWPR or r_u) progressed to 1. EPWPR is the ratio of EPWP and the initial effective stress before shaking at corresponding depth. The dissipation rate of EPWP in case of Models 2 and 4 is quite fast in comparison with the one observed for Models 1 and 3. This significant increase in the dissipation rate of EPWP justifies the effectiveness of gravel drainage provided in case of Models 2 and 4. At level 5 (P8, P9, and P10), dissipation of EPWP did not start for quite a long period of time in case of Models 1 and 3 in comparison with the trend observed for Models 2 and 4. The reason for this is the availability of migrated pore fluid from the deeper portion even after shaking. It is to be noted that the EPWP can only dissipate from top of the model ground because of boundary constraints. As the pore fluid keep coming towards the shallow portion of the model ground because of pore pressure

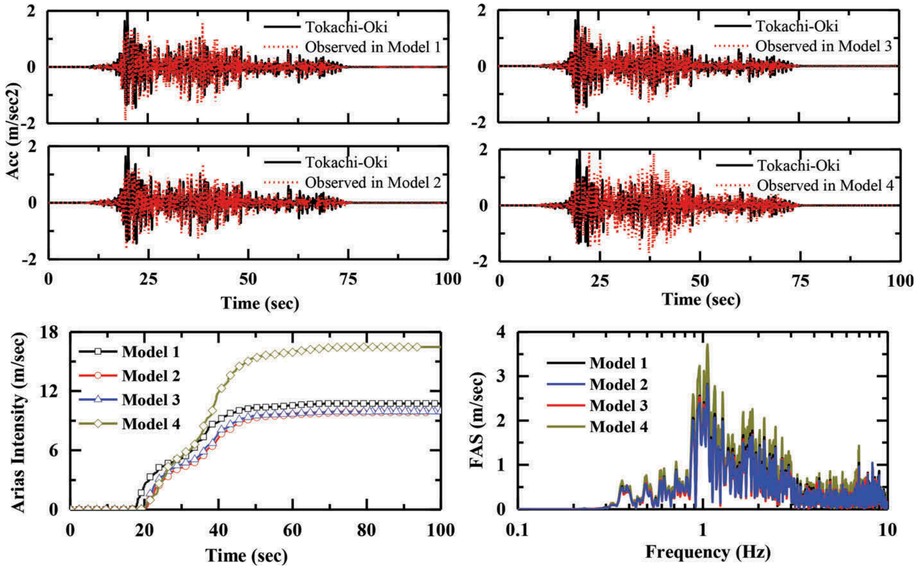
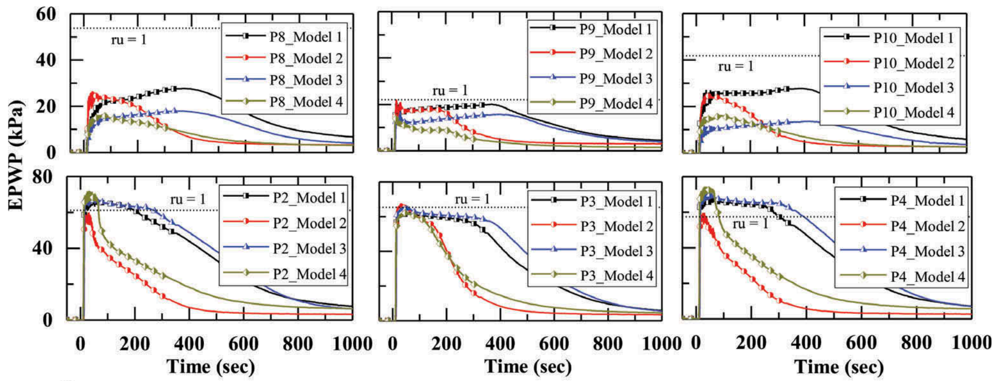


Figure 3. Acceleration time histories, Arias intensity and Fourier amplitude spectra (FAS) of input waves for all Models 1-4



*P8, P9, and P10 at Level 5 (@ 2 m depth); P2, P3, and P4 at Level 3 (@ 6 m depth)

Figure 4. EPWP time histories of all the Models during Tokachi-Oki ground motion

difference and boundary constraints, the traces of delayed dissipation (dissipation starts at around 400 seconds) for Models 1 and 3 (at P8, P9, and P10) can be justified. However, this is not the case for models having gravel drainage system (Models 2 and 4). The reason for this is that the gravel drainage is able to dissipate the EPWP as desired which devoid the pore fluid migration from the deeper portion towards the shallower portion of the ground.

Similar behavior is observed at Level 3 (at P2, P3, and P4); however, late development of EPWP at around 400 seconds is not traced for Models 1 and 3 at Level 3 as it is the case for PPTs at Level 5. Surprisingly, the maximum magnitude of EPWP at Level 5 for Model 2 is considerably large in comparison with Model 4 even though the gravel drainage system is alike for both the models. There might be several reasons associated to this behavior such as placing of PPTs very close to gravel drains, better performance of gravel drainage system in case of Model 4, and/or comparatively deeper positioning of PPTs at Level 5 in case of Model 2. P8 and P10 exhibits considerable large hydrostatic pressure before shaking in case of Model 2 in comparison with Model 4 which consolidate the assumption of relative deeper positioning

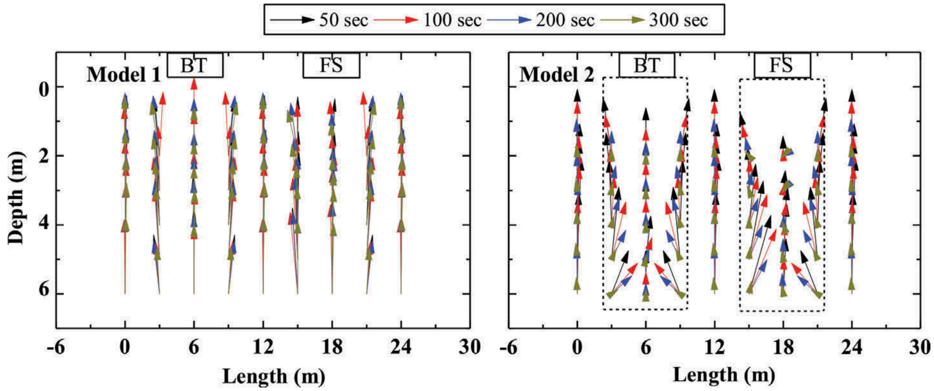


Figure 5. Pore fluid streamlines at different times for Models 1 and 2 during Tokachi-Oki ground motion

of the same. Whereas at P9, less EPWP for Model 4 in comparison with Model 2, seems to be because of better performance of gravel drainage system.

Pore fluid velocities during and after the shaking are deduced from the EPWP time histories recorded at several locations within the model ground to get an insight of pore fluid flow and drainage effects in case of Model 2 in comparison with Model 1. Initially the hydraulic gradient field is estimated by pore pressure distribution and then the flow direction (velocities) is estimated by using the same. Figure 5 depicts the velocities of pore fluid for Models 1 and 2 during Tokachi-Oki ground motion. It is to be noted that the dissipation of pore pressure is only permitted from the surface of the model ground because of boundary constraints in the experiment, hence dominatingly the flow of pore fluid is always upward even after the shaking and continued until pore fluid pressure reached the equilibrium state throughout the model ground (Zeybek & Madabhushi, 2017) which could be justified from the velocities deduced in case of Model 1. Drainage effects of gravel drainage system is apparent as the radial flow (towards the gravel drainage zone) is formed as indicated by velocities in case of Model 2 as shown in Figure 5. However, at the shallower depth, the radial flow of pore fluid is not significant under both BT and FS footing. In addition, the influence of gravel drains on the flow at model centerline seems negligible as the flow velocities are very much alike for both Models 1 and 2.

3.2 *Settlement behavior*

Settlement time histories of both BT and FS footings are shown in Figure 6 for all the Models during Tokachi-Oki ground motion. Total settlement of both BT and FS footings

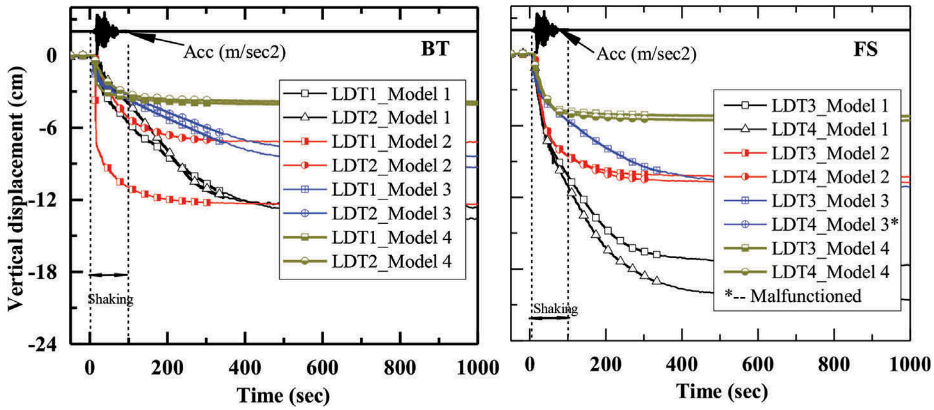


Figure 6. Settlement time histories of BT and FS footings during Tokachi-Oki ground motion

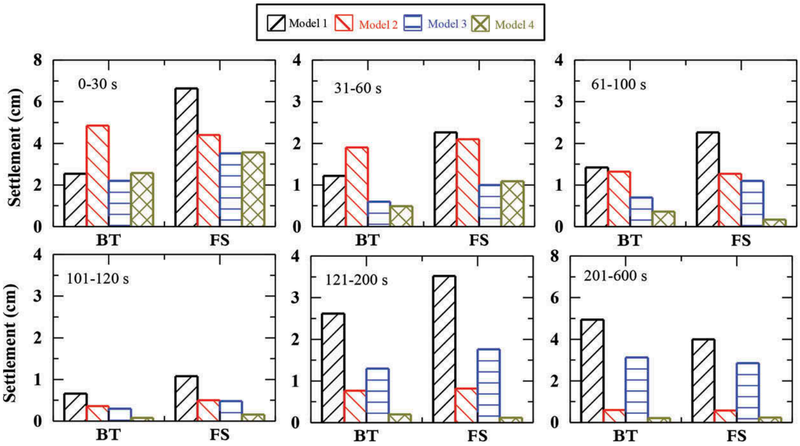


Figure 7. Average settlement of BT and FS footings for different time periods during Tokachi-Oki ground motion

can be divided into two phases. The first phase is the co-shaking settlement which occurred during the time period of shaking and the second is the post-shaking settlement which takes place after the shaking. Co-shaking settlement mechanism of shallow foundation resting on liquefiable ground is a complex phenomenon dominatingly affected by several factors, e.g., the cyclic shearing-induced evolution of EPWP, and inertial and kinematic interaction within of soil-foundation-structure system. Whereas, post-shaking settlement dominatingly governed by localized drainage, and post-shaking/liquefaction reconsolidation (Dashti et al., 2010).

It is evident that in case of Model 1, both BT and FS experienced excessive settlement which cumulatively took place during both co-shaking and post-shaking time period. This also stipulate that the shallow foundation resting on liquefiable ground is prone to undergo severe liquefaction-induced deformation during the earthquake. In case of Model 2, the observed overall settlement is less compared to Model 1. Presence of gravel drainage in Model 2 is able to minimize the settlement after shaking. However, BT exhibits significant differential settlement (relative difference in LDT1 and LDT2) in case of Model 2. The reason for this is the change in initial condition before shaking under BT for Model 2. BT foundation experienced a cumulatively considerable amount of differential settlement (2.3 cm in prototype scale) during spinning up the centrifuge up to 40g and due to the white noise shaking. This uneven settlement of BT before shaking exaggerated the differential settlement during Tokachi-Oki ground motion as shown in Figure 6.

The overall settlement of BT and FS in case of Model 3 is less in comparison with Model 1. Recorded bending moment and axial force (not presented in this paper) at several locations as shown in Figure 2, exhibited that the friction piles having spiral blades at bottom serve a good means of frictional/bending resistance against the settlement of shallow foundation. This resulted in overall less deformation of shallow foundation because of combined inertial and kinematic interaction with foundation-superstructure and ground respectively. However, the presence of friction piles could not avoid the post-shaking settlement in case of Model 3. In case of Model 4, total settlement of both BT and FS is smaller among all the models and post-shaking settlement is almost diminished. Figure 7 shows the average settlement of BT and FS for different time periods during Tokachi-Oki ground motion. It is evident that the presence of gravel drainage minimized the post-shaking settlement (after 100 seconds) in Models 2 and 4. Though the settlement during shaking is small in case of Model 3 in comparison to Model 1 and 2; however, comparatively large amount of settlement occurred after shaking in case of Model 3. The combined effect of both friction pile and gravel drainage is evident in case of Model 4 as the settlement occurred in all time intervals is remarkably less compared to Models 1, 2, and 3.

4 CONCLUSIONS

A series of dynamic centrifuge experiments are carried out to investigate the performance of proposed hybrid foundation to mitigate the liquefaction-induced effects on shallow foundation. The research is carried out in three phases. In the first phase, an attempt is made to understand the behavior of shallow foundation resting on liquefiable ground during Tokachi-Oki ground motion. In the second phase, the performance of gravel drainage system and friction piles are investigated. The explicit conclusions based on the subject matter presented herein are as below:

1. Liquefaction mitigation is necessary for desirable serviceability of shallow foundation resting on liquefiable ground. Test results indicate that shallow foundation resting on untreated liquefiable ground undergo excessive settlement during the dynamic event.
2. Presence of gravel drainage increase the dissipation rate of generated EPWP and reduced the post-shaking/liquefaction settlement.
3. Friction piles having spiral blades at bottom reduced the overall deformation of shallow foundation significantly by means of combined inertial and kinematic interaction with foundation-superstructure and ground respectively.
4. The combination of gravel drainage and friction piles performs as desired to mitigate the liquefaction-induced effects on shallow foundation.

ACKNOWLEDGEMENT

The work presented in this paper is part of the collaborative research with the Nippon Steel & Sumikin Engineering. First author sincerely acknowledges the support provided by MEXT for graduate students. The authors are also indebted to Mr. Sakae Seki, lab technician, dept. of Civil and Environmental Engineering, Tokyo Institute of Technology for his tremendous contribution to the successful completion of centrifuge experiments.

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