LARGE-SCALE SHAKE TABLE EXPERIMENT ON THE PERFORMANCE OF HELICAL PILES IN LIQUEFIABLE SOILS

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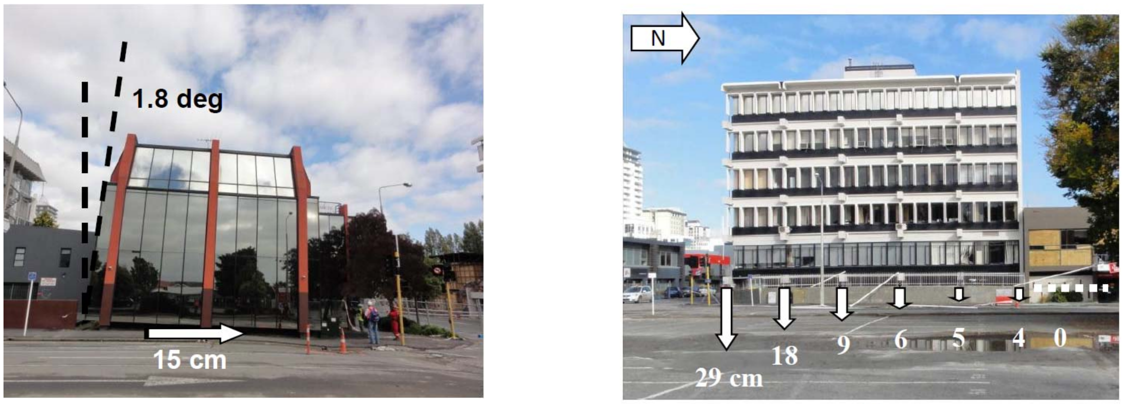
ABSTRACT

Documented case histories from past earthquakes have exhibited the devastating effects of the liquefaction phenomenon on the performance of buildings that are often presented in the form of excessive settlement of shallow foundations. Example earthquakes include the 1964 Niigata earthquake in Japan, and more recently the 2010-2011 Canterbury Earthquake Sequence in New Zealand, and the 2011 Tohoku earthquake in Japan. All of the listed events comprised several examples of severe liquefaction-related damage to the buildings founded on shallow foundations. Although the liquefaction-induced building settlements have been experimentally studied extensively since 1964 using shake table tests and dynamic centrifuge experiments, studies focusing on cost-effective mitigation strategies are still limited. In this study, the performance of helical piles as a cost-effective countermeasure to minimize the damaging effects of liquefaction-induced ground settlements on buildings founded on shallow foundations is studied. Two large-scale shake table experiments were conducted at the University of California, San Diego (UCSD) to investigate the effectiveness of helical piles in liquefiable grounds. Results from these two experiments highlight a highly improved performance of a shallow foundation supported on helical piles. These large-scale shaking table experiments demonstrate the efficiency of using helical piles as a cost-effective countermeasure to reduce the liquefaction-induced building settlement.

Keywords: liquefaction, shake table test, helical pile, liquefaction-induced settlement

PREVIOUS WORK

Documented case histories involving widespread liquefaction have been used for further understanding the devastating effects of liquefaction on foundation systems in past earthquakes. Recent examples of these earthquakes were the 2010-2011 Canterbury Earthquake Sequence (CES) in New Zealand and the 2011 Tohoku earthquake in Japan, which resulted in significant liquefaction-induced damage to buildings and their foundations (Cubrinovski 2011; Yasuda et al. 2012). Figure 1 presents some of the examples of shallow-founded buildings undergoing excessive settlement during the 2011 Christchurch earthquake. The liquefaction-induced building settlement phenomenon consists of three major mechanisms: 1- shear-induced, 2- volumetric-induced, and 3- ejecta-induced, where each mechanism is further sub-categorized to its contributing effects. Further details concerning mechanisms of liquefaction-induced building settlement were presented in Bray and Dashti (2014). The contribution of each mechanism in liquefaction-induced foundation settlement is thoroughly evaluated in previous works; however, there is still a lack of information about the ejecta-induced mechanism and its contribution. Jahed Orang et al. (2019a) elaborated on some preliminary aspects on the contribution of ejecta in liquefaction-induced foundation settlements. The results of their study demonstrated a linear relationship between total foundation settlement and volume of the ejecta up to a point, beyond which there was no pronounced impact of ejecta volume observed in liquefaction-induced foundation settlements.



(a) (b)

Fig. 1. (a) Liquefaction-induced differential settlement and sliding of building of shallow-founded three-story building, (b) liquefaction-induced differential settlement in Christchurch, New Zealand (Cubrinovski et al. 2011).

In addition to better understanding the key mechanisms of liquefaction-induced foundation settlement, it is essential to evaluate different ground improvement and foundation underpinning techniques. Past ground improvement studies included different methodologies such as soil densification, use of underground columns, improved drainage techniques (i.e. prefabricated vertical drains), and underground walls around building premises (Adalier et al. 2003; Olarte et al. 2017; Rasouli et al. 2018). All of the above-mentioned mitigation techniques varied depending on the method used for the ground improvement, yet all demonstrated some extent of effect in reducing the liquefaction-induced settlement. In a recent study, synthetic polymer was evaluated to mitigate the liquefaction-induced foundation settlement (Prabhakaran et al., 2020a). Two shake table tests were conducted at UCSD Powell laboratory to examine the efficiency of polymer injection method. Considerable amount of mitigation was achieved by injecting an expansive polymer into a shallow liquefiable soil deposit. Details about the polymer injection method and its efficacy can be found in Prabhakaran et al. (2020b). However, there is still a lack of research on the efficiency of mitigation measures considering the associated cost of the applied remedial measure to reduce liquefaction-induced settlements.

Helical piles are a type of deep foundation elements which are used for underpinning foundations in existing and new construction, especially in areas with limited access and low headroom. The main components of helical piles consist of a lead section, an extension part, helical plates, and coupling connections (Perko, 2009). Although satisfactory performance of helical piles have been observed during past earthquakes in New Zealand, Japan and the U.S., design codes do not address the use of helical piles in high seismic zones (Cerato et al., 2017). Recent large-scale shake table test at UCSD outdoor shake table facility (Elsawy et al., 2019) and moderate-scale 1g shake table test at UNR (Orang et al., 2019b) shed some light on the adequate performance of helical piles in dry sand. In this study, helical piles were used to evaluate the performance of these elements in surficial liquefiable deposits while supporting a shallow foundation. The overall scope of this study is to assess cost-effective remedial measure for liquefaction-induced foundation settlement including helical piles and their efficiency in reducing liquefaction-induced foundation settlements.

SHAKE TABLE EXPERIMENTAL SETUP

Two series of large-scale shake table tests were conducted at Powell Laboratory at the University of California, San Diego (UCSD), using a 2.9 m tall laminar soil box. Each series included two shaking sequences with varying peak acceleration. In this study, only the first shaking sequence for both series of tests is reported. The first test series, referred as 'baseline' in this paper, presents the test configuration without any mitigation, while the second series, referred as the 'helical pile' test, corresponds to the experiment in which helical piles were installed to underpin the foundation. Both test series were constructed in similar ground conditions and structural elements (i.e. shallow foundation). Figure 2 presents the completed test configurations for the helical pile test as well as the instrumented helical piles prior to the installation. The aluminum tapes around each of the helical piles at different levels in Fig. 2(b) indicate the strain gauge locations. Further details regarding model configuration, instrumentation, soil properties, and ground motion characteristics will be discussed in the following sections.

(a) (b)

Fig. 2. (a) Model shallow foundation configuration for helical pile test before shaking, (b) Helical piles configuration after instrumentation (Helical piles provided by Ram Jack).

EXPERIMENTAL PROCEDURE

The ground models for both experiments were built in a three-layer deposit configuration, using Ottawa F-65 sand. Further details about Ottawa F-65 sand can be found in Bastidas (2016). A saturated 1.00 m dense sand layer was overlain by a 1.30 m thick saturated liquefiable deposit. Finally, the 0.60 m top layer was constructed as a crust medium. The schematic illustration of the model is shown in Fig. 3. The construction method of all three layers is described in detail in the following sections.

Dense Layer

The dense layer was compacted in a moisture-conditioned state in three sublayers, using a plate compactor. Once built, saturation was achieved by adding water through a horizontally-connected perforated piping system installed at the base of the soil box after the construction of the dense deposit. The relative density of each layer was calculated based on the weight-volume relationship. The achieved relative density of the dense layer was about 85-90%.

Loose Layer

The loose liquefiable layer was built by hydraulic pluviation of dry Ottawa F-65 sand using a hopper. The pluviated sand was passed through two sets of screens into the water, allowing a full saturation. The final thickness of the liquefiable stratum was 1.30 m with a relative density of about 40-45%, based on the weight-volume calculations.

Crust Layer

Finally, the top crust layer was built through the air pluviation method in two stages. Only one screen was used to deposit the soil into the soil box (i.e. screen was located below the hopper). First, an initial thickness of 0.20 m was deposited following the foundation placement on top of the soil model. In the next stage, an additional 0.40 m thick deposit of crust layer was placed, making the total thickness of the crust layer 0.60 m. The achieved relative density of the crust layer was about 50-55%. The above-mentioned procedure was applied for the model ground preparation in both baseline and helical pile tests, although in the helical pile test four helical piles were connected to the shallow foundation using side brackets. Further details about helical piles and connecting brackets are provided in the next section.

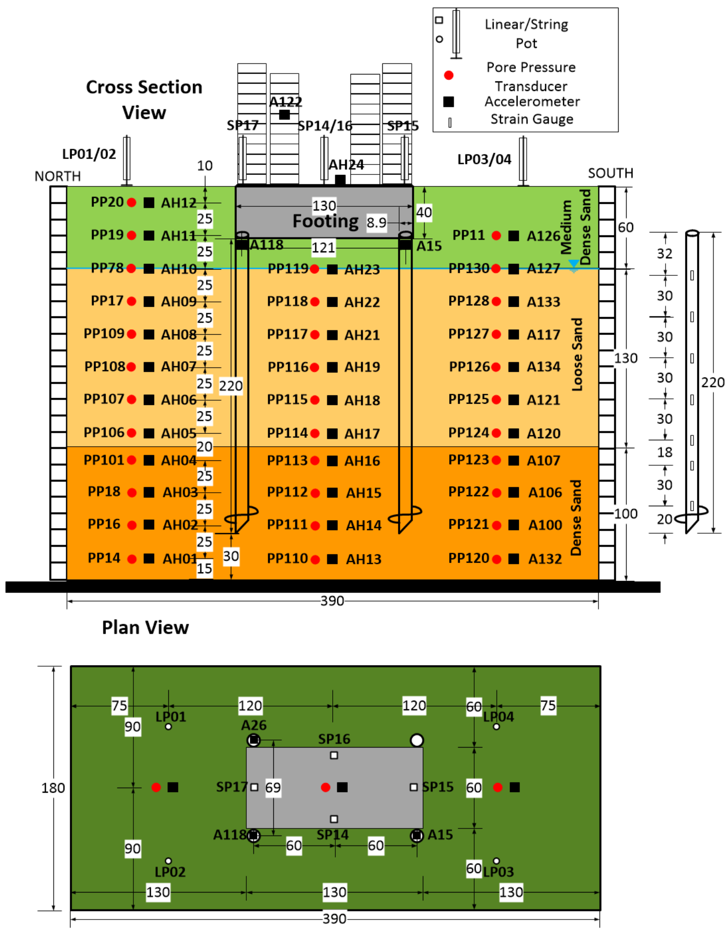


Fig. 3. Model configuration and instrumentation plan for helical pile test (All dimensions are in centimeters).

Helical Piles

In the helical pile test, a total of four single-helix helical piles were used to underpin a shallow foundation sitting on the top of the ground model. Specifications of the helical piles are provided in Table 1. In order to connect helical piles to the shallow foundation, four 4021.1 side brackets were used. Two 1.6 cm anchor bolts were used to connect each side bracket to the concrete foundation. Further details of the side brackets and mechanical properties of the helical piles are provided in the ICC-ES evaluation report (*ESR-1854 Ram Jack,*2017). The individual bearing capacity method was used to calculate the bearing capacity of helical piles. The N_q value was obtained from the Hansen and Vesic method (Perko, 2009). The allowable individual bearing capacity of each helical pile was about 16.7 kN. Assuming a pile group efficiency factor of 1, the pile group capacity was about 66.8 kN, which was higher than the total applied load on top of the shallow foundation. The foundation contact pressure through both tests was 41.6 kPa, which corresponded to a 32.5 kN total load, resulting in a factor of safety of about 2.0.

Table 1. Helical pile specification

Property	Value	Property	Value
Helix Pitch (cm)	7.5	Outside Diameter (cm)	8.9
Helix Level below Ground (cm)	240	Wall Thickness (cm)	0.50
Penetration into Dense Layer (cm)	76.2	Shaft Length (cm)	220
Longitudinal Pile to Pile Distance (cm)	121	Helix Diameter (cm)	20
Transverse Pile to Pile Distance (cm)	69		

Instrumentation

Extensive instrumentation was deployed in these series of tests to measure settlements, strains, pore-water pressure, and accelerations at different depths. A total of 134 and 149 sensors were used in the baseline and helical pile tests, respectively, to capture the seismic performance of the soil-foundation-structure system along with the dynamic behavior of helical piles. The instrumentation plan was similar in both series of tests (i.e. baseline and helical pile tests) except that the strain gauges were only used in the helical pile test. Figure 3 illustrates the instrumentation layout for both tests. Seven pairs of strain gauges were also attached to each helical pile to capture the bending strains along the helical piles, which are shown in Fig. 3 as well.

Recorded Motion Characteristics

Two shaking sequences with varying peak accelerations were applied in both the baseline and helical pile tests. Due to the page limitation, only the results of the first shake are included in this paper. The acceleration time history of the first shake as well as the spectral acceleration response for the base, foundation, and near foundation motions are presented in Fig. 4 for both tests. The peak acceleration in both tests was about 0.53g without any filtering, which is representative of the event recorded during the February 22, 2011 earthquake in New Zealand. The uniform sinusoidal motion was applied in 15 seconds with a fundamental frequency of 2 Hz. Figure 4 further illustrates that both foundation and near foundation motions at the ground surface were significantly damped due to the development of extensive liquefaction in the middle of the loose layer.

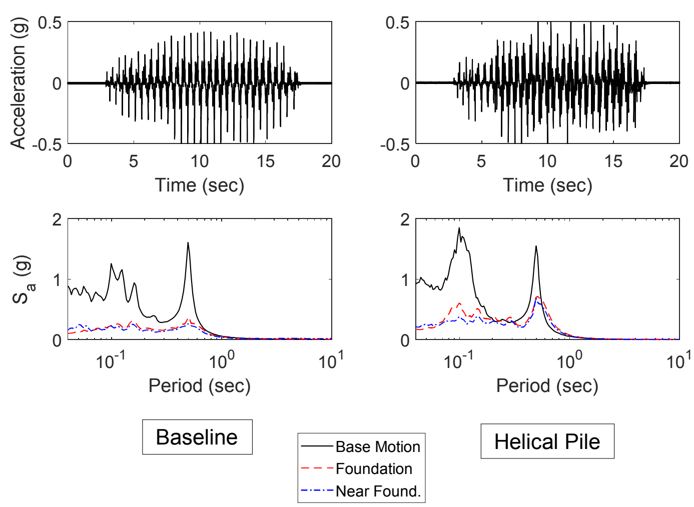


Fig. 4. Recorded motion characteristics in baseline and helical pile tests.

RESULTS AND DISCUSSION

The efficiency of using helical piles to mitigate liquefaction-induced foundation settlement was evaluated through a comparison between two test series. The first (i.e. baseline) test was conducted to provide a benchmark standpoint in terms of liquefaction-induced foundation settlement with significant emphasis on surficial liquefiable deposits. The main objective of the second (i.e. helical pile) test was to evaluate the use of helical piles as a cost-effective countermeasure to reduce liquefaction-induced foundation settlement. Figure 5 presents the model before and after the helical pile test, exhibiting insignificant foundation settlements. It was observed that the amount of settlement and tilt of the foundation was significantly reduced during the helical pile test (Fig. 5b). Further details regarding excess pore water pressure generation, differential settlement and tilt of the foundation, as well as overall settlement of the foundation during both test series, are discussed in the following sections.

(a) (b)

Fig. 5. Model shallow foundation (a) before and (b) after completion of helical pile test.

The helical piles were carefully examined for any potential damage after tests were completed, and Fig. 6 demonstrates the extracted helical piles after the completion of all the shaking sequences. As can be seen, there was no observed deformation of the single helix helical piles, which indicates the satisfactory performance of the helical piles when subjected to strong ground motions in unstable liquefiable soils.

Fig. 6. Extracted helical piles after test completion.

EPWP Time Histories

The generation of excess pore water pressure (EPWP) during liquefaction is an indication of strength reduction, which can cause foundation settlement due to the various liquefaction-induced settlement mechanisms. The EPWP generation during both tests was captured through three different arrays of pore water pressure sensors, where the results of the array below the foundation are presented in this paper. Figure 7 illustrates the EPWP below the foundation array for both testing series at different depths. The maximum EPWP was generated at depths from 0.60 m to 1.9 m within the liquefiable layer during both tests. The generated EPWP in the helical pile test was generally lower than its corresponding values during the baseline test. Another observation based on Fig. 7 is the EPWP generation rate was often higher in the baseline test compared to the helical pile test. The highest EPWP was observed at the mid-depth of the liquefiable layer for both tests. All of the above-mentioned observations are mainly attributed to the presence of helical piles, which caused relative densification around the foundation array. In addition, applied pressure by the foundation and continuous contact throughout the baseline test could also be responsible for this observation. More detailed discussion about the EPWP generation and dissipation during the baseline test can be found in Jahed Orang et al. (2020).

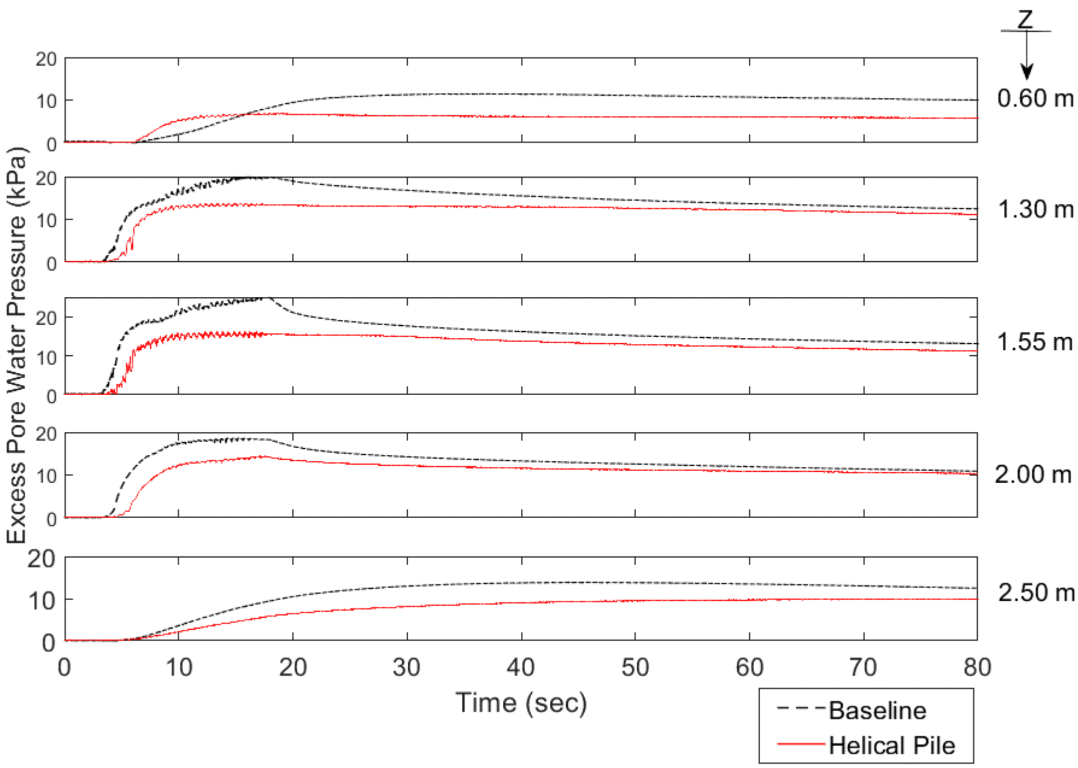


Fig. 7. Excess pore water pressure generation along depth during baseline and helical pile tests for below foundation array of sensors.

Differential Settlement and Tilt of the Foundation

One of the consequences of liquefaction is the induced foundation settlement. The main purpose of this shake table experimental research was to provide a cost-effective countermeasure to mitigate liquefaction-induced foundation settlement. Tilting and differential settlements are other aspects of liquefaction-induced damage in shallow foundations, which can ultimately result in the demolition of buildings after earthquakes. The observed behavior of a shallow foundation with and without any mitigation measure (i.e. the helical pile and baseline test) is provided in Fig.8. The in-plane and out-of-plane differential settlements during the baseline test were about 4.9 and 2.0 cm, respectively, which were significantly reduced to almost no differential settlement in the case of the helical pile test. The same discussion is also valid for the tilt of the shallow foundation. As can be observed in Fig. 8, the amount of transient differential settlement and tilt of the foundation were negligible during the helical pile test compared to the baseline test. The maximum transient differential settlement and transient tilt of the foundation during the helical pile test were less than 1.0 cm and 0.5 degrees, respectively. In summary, the amount of improvement due to the use of helical piles was verified through the observation of the differential settlement and tilt of the foundation.

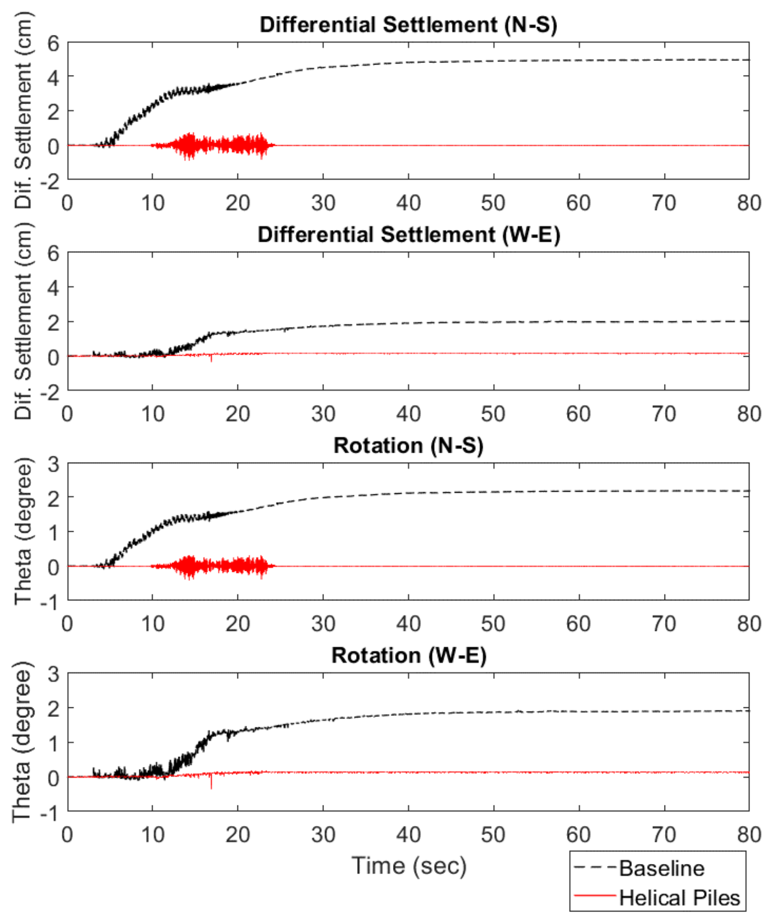


Fig. 8. In-plane and out-of-plane rotation and differential settlement for baseline and helical pile tests.

Liquefaction-Induced Foundation Settlement

Previously, the contributing mechanisms of the liquefaction-induced foundation settlement were discussed in the background section. Figure 9 demonstrates the average foundation settlement in both the baseline and helical pile tests based on four string potentiometer measurements located on the foundation. The position of string potentiometers is presented in Fig. 3. The average foundation settlements during the baseline and helical pile tests were 28 cm and 1.2 cm, indicating a significant reduction which in turn is evidence of improved performance of the shallow foundation. Figure 9 further shows that the vertical settlement of the foundation during the helical pile test ceased after the shaking stopped, indicated with a red line in Fig. 9. This observation illustrates that the foundation settlement was not affected due to the volumetric-induced and ejecta-induced mechanisms during the helical pile test as the piles transferred the foundation loads to the competent dense layer. One of the shear-induced mechanisms known as partial bearing capacity failure was also compromised due to the added bearing capacity of helical piles during the helical pile test. The contributing mechanisms of the foundation settlement during the helical pile test were transient high hydraulic gradients and SSI ratcheting during the shaking. These contributing liquefaction-induced foundation settlement mechanisms during the helical pile test had minimal effect compared to the foundation settlement in the baseline test. In contrast to the helical pile test observation, all of the liquefaction-induced settlement mechanisms contributed to the foundation settlement during the baseline test. Detailed discussions regarding the foundation settlement during the baseline experiment are provided in Jahed Orang et al. (2020). Another important observation is the reduced rate of settlement accumulation during shaking in the helical pile test compared to the baseline experiment.

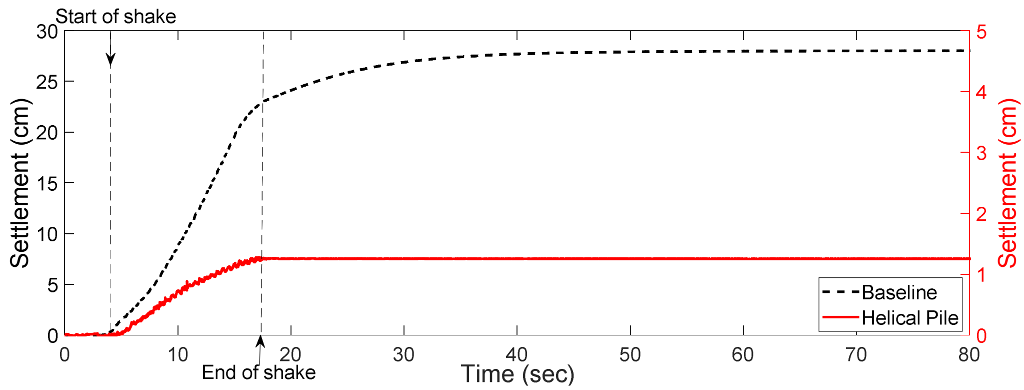


Fig. 9. Foundation settlement during baseline and helical pile tests.

CONCLUSIONS

Two series of large-scale shake table tests, namely baseline and helical pile tests, were conducted to evaluate the efficiency of using helical piles to mitigate liquefaction-induced foundation settlement. A shallow foundation was located on top of a three-layered ground model in a two-experiment program where a prototype scale representative of the shallow liquefiable deposit was reproduced similar to the observed case studies in past earthquakes. In the helical pile test series, four single helix helical piles were used to underpin an existing shallow foundation and improve its performance during strong shaking sequences. In this paper, only the results of the first shaking sequence for tests were presented. The important findings of this study are summarized hereafter:

The built-up rate and the amount of generated pore water pressure during shakings were lower for the helical pile test compared to the baseline experiment. This can be attributed to the densification of soil around helical piles, which resulted in lower pore water pressure generation. In addition, the foundation contact pressure in the baseline test contributed to this observation, whereas in the helical pile test, the foundation load was transmitted to the dense layer below the liquefiable soil.

There was no residual tilt and differential settlement of the shallow foundation supported on helical piles during and after the shaking. The total settlement of the foundation was reduced by approximately 96% in the helical pile test, which highlights the achieved performance improvement and the efficiency of using helical piles in mitigating liquefaction-induced foundation settlements. The cost-effectiveness of utilizing helical piles, especially when dealing with low headroom areas or in the case of underpinning an existing residential building, are also other important features of helical piles.

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