Performance of Single and Grouped Helical Piles under Strong Earthquake Shaking

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Abstract

Full-scale shaking table testing was conducted to evaluate the seismic performance of single and grouped helical piles. Eight circular and one square helical piles with different properties as well as one driven circular pile were installed in dry sand enclosed in a laminar soil shear box that was situated on the shaking table. Dynamic properties of sand bed and its natural frequency, as well as the natural frequencies of single and grouped helical pile-soil systems were evaluated from the collected data during different shaking events. The effects of different pile configurations as well as successive shakings on the characteristics of the sand bed and pile-soil systems were also investigated. In addition, responses of single and grouped helical piles were computed analytically using the software DYNA6, which agreed with measured responses. These analyses accounted for degradation of soil stiffness and gap opening. Furthermore, effects of the earthquake characteristics (i.e., intensity and frequency content) on seismic performance of single and grouped helical piles were evaluated from the measured responses. The performance characteristics of helical pile groups were discussed in terms of the interaction between piles within a group and the contributions of vertical and lateral stiffness of individual piles to the rocking stiffness and the overall capacity of the pile group. The effect of pile head connection to the pile cap (1-bolt connection versus 2-bolts connection) was evaluated and compared. In addition, behaviour of a single pile and a pile within a group were compared in terms of their normalized responses. A three-dimensional nonlinear dynamic finite element model was constructed employing the software ABAQUS to simulate the shake table testing. The numerical model was validated with the experimental results, which was then used to perform an extensive parametric study. The parametric study explored the effect of the level of a single helix, diameter of a single helix, number of helices, spacing between helices as well as the effect of the pile diameter. Results revealed the superior performance of helical piles under strong ground shakings due to the significant contribution of the helices to the rocking resistance of the pile group.

Keywords: Helical piles; seismic response; shake table; soil-pile interaction; group behaviour; Finite Element Method
Summary for Lay Audience

Helical piles are considered a reliable and cost-effective alternative to conventional driven piles because of their fast installation, lower cost and lower labour risk. They are suitable for retrofitting existing deficient foundations because they require smaller installation equipment that cause minimal vibration and noise during installation. Observations from recent strong earthquakes have demonstrated excellent performance of structures supported on helical piles with negligible damage. However, some buildings in the same areas that were supported on conventional reinforced concrete piles collapsed. This motivated the research community to explore the reasons why helical piles substantially performed well during seismic events. This study aimed at understanding the behaviour of single helical piles and helical pile groups during a strong earthquake shaking. Results from full-scale testing as well as numerical modelling revealed the superior performance of helical pile groups under seismic loading.
Co-Authorship Statement

The full-scale experimental shake table testing involved in this study was performed at the Large High Performance Outdoor Shake Table (LHPOST) located at the University of California – San Diego (UCSD) under the supervision of Dr. Ahmed Elgamal. The design of piles, layout, instrumentation plan and loading schemes was provided by Dr. Hesham El Naggar and Dr. Amy Cerato from University of Oklahoma. The setup and all physical preparation of the testing as well as laboratory and field-testing of the sand bed was done under the supervision of Dr. Cerato. Raw data collected from the data acquisition system during the testing program as well as unprocessed results of field and laboratory testing of the soil were provided by Dr. Cerato to the author. The author would like to state that he did not contribute or was part of the shake table testing and/or the soil testing.

All the work involved in this study including: processing of the raw data, analyzing the field and laboratory soil tests, experimental data interpretation and numerical modelling was performed by the author alone under the direct supervision of Dr. El Naggar.

Publications resulted from this study were drafted by the author, reviewed and edited by Dr. El Naggar, Dr. Cerato and Dr. Elgamal. The following is a list of publications submitted and in preparation:


2- Fayez AF, El Naggar MH, Cerato AB, Elgamal A. “Seismic Response of Helical Pile Groups from Shake Table Experiments”. Soil Dynamics and Earthquake Engineering. **Under Final Review.**

3- Effects of Different Geometric Parameters on the Responses of Helical Pile Groups under Strong Shaking from 3D Numerical Model. **In Preparation.**
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Chapter 1

Introduction

1.1 Overview of Helical Piles

A helical pile is comprised of a steel shaft with one or more helical plates attached to it, which is commonly known as the lead shaft. The helices are characterized by their diameter, thickness and pitch. The length of the helical pile can be increased by adding extension shafts using bolted or threaded couplings. They are connected to superstructures using bolted brackets. Figure 1-1 presents a schematic of different components of helical piles.

![Figure 1-1. Schematic of different components of helical piles (Perko 2009)](image)

The helical pile is installed in the ground by applying a mechanical torque to its head. Usually, this is achieved through a hydraulic machine with an equipped torque motor. Figure 1-2 presents an example of the installation setup of a helical pile.
In recent years, helical piles have become an attractive alternative to conventional driven piles, either steel or concrete, due to their many advantages such as:

1- Easy and rapid installation.
2- Reduced cost due to efficient use of material.
3- Reduced risk to labour.
4- Small footprint, which is considered ideal for retrofitting.

And most importantly,

5- The ability to estimate their axial capacity from installation torque measurements (Perko 2009).
6- Reduce the liquefaction consequences.

Owing to their configuration, helical piles can be designed to place the load bearing helical plates below soft or liquefiable soil to ensure acceptable performance and capacity during and after an earthquake. In addition, recent advances in helical pile installation equipment promoted the use of large diameter helical piles installed to large depths to achieve large capacity for supporting high-rise buildings, bridges, and offshore installations.
1.2 Research Motivation and Knowledge Gap

Helical piles have performed well during recent large earthquakes (Cerato et al. 2017). Anecdotal evidence from recent earthquakes in Christchurch, New Zealand (2011, 2012) and Alaska, USA (2016, 2018) revealed that structures supported on helical piles withstood earthquakes with limited or no structural damage. For example, Anchorage, Alaska, experienced back-to-back earthquakes and aftershocks of 7.0 and 5.7 magnitudes that damaged public buildings and houses causing significant foundation settlement, which damaged many structures. However, structures supported on helical piles remained intact and level. Surveys conducted after the earthquake revealed that all structures supported on helical piles have not experienced any damage or settlement due to the earthquake as was published in a news article by Techno Metal Post (TMP), which is a helical pile manufacturer (Figure 1-3).

![Figure 1-3. Techno Metal Post news article about performance of structures supported on helical piles during earthquakes](https://www.technometalpost.com/en-CA/structures-supported-by-techno-metal-post-helical-piles-withstand-2016-and-2018-alaska-earthquakes/)

Helical piles are well investigated under axial loading, in which the axial capacity of helical piles are greater than equivalent conventional piles for the same pile shaft. This is mainly due to contribution of the helices to the axial capacity of the pile. Lateral capacity of single helical piles received more attention recently due to the interest in their use in wind turbines and offshore structures. Although single piles have been studied under lateral static, cyclic or seismic loading, all previous dynamic testing included loads at the pile head, which simulates the inertial pile-soil interaction only. On the other hand, dynamic and seismic testing of helical pile groups was not examined before.

While helical piles proved their excellent performance during earthquakes based on recent observations, there is still no documentation or interpretation why. This was the motivation
of this study, in which the seismic performance of single and grouped helical piles were investigated by means of: first, full-scale shake table testing, which simulates both inertial and kinematic pile-soil interaction. Second, through FEM numerical modelling.

1.3 Objectives

The main objective of this study is to investigate the seismic performance of helical pile groups subjected to strong earthquake shakings by means of experimental data obtained from full-scale shake table testing as well as FEM numerical modelling. This was achieved through the following sub-objectives:

1- Evaluate the dynamic characteristics of different configurations of single and grouped helical piles from experimental data and analytically using DYNA6 software (El Naggar et al. 2015).

2- Study the effects of successive shakings on the stiffness of different helical pile-soil systems.

3- Explore the performance of different single and grouped helical piles under different loading schemes; pulse waves, white noise signals and earthquake records.

4- Investigate the effects of the earthquake frequency content and intensity on the seismic performance of single and grouped helical piles.

5- Calibrate and validate a full 3D nonlinear dynamic FEM numerical model to simulate the shaking table test employing the software ABAQUS (SIMULIA 2013a).

6- Study the effect of some geometric parameters of helical pile groups on their seismic performance through an extensive parametric study employing the calibrated numerical model. These properties include the level of single helix, diameter of single helix, number of helices, spacing between helices and pile’s diameter.
1.4 Thesis Organization

This thesis consists of eight chapters that are briefly described as follows:

**Chapter 1** describes the problem statement and the research motivation as well as the objectives of this research work. In addition, the scope and organization of the thesis are presented.

**Chapter 2** provides a brief literature survey regarding the lateral performance of single and grouped piles. Available studies concerning single helical piles and helical pile groups are also reviewed.

**Chapter 3** provides a detailed overview of the shaking table experimental program performed at the Large High Performance Outdoor Shake Table (LHPOST) located at the University of California – San Diego (UCSD). This includes a brief description of the testing facility, soil preparation, field and laboratory testing of sand, different configurations, layouts and instrumentation of piles and/or soil for each testing day, ground motions applied throughout the testing program and finally the scheme employed for data preparation.

**Chapter 4** presents the dynamic properties of the sand bed and different pile-soil systems determined from the experimental data. This includes shear wave velocity, damping ratio and natural frequency of the sand bed as well as the natural frequencies of single and grouped helical piles. In addition, the effect of different soil and/or pile configurations as well as the effect of successive shakings are discussed. Moreover, dynamic properties are computed analytically using DYNA6 software, and compared with the experimental results.

**Chapter 5** presents the measured responses of single helical piles and helical pile groups to the strong ground motions applied during the experimental program. The effect of earthquake intensity, earthquake frequency content and pile head connection are discussed. Moreover, interaction between piles within a helical pile group is presented as well as a comparison between single piles and individual piles within a group in terms of normalized responses.
Chapter 6 discusses the different aspects involved in constructing a full three-dimensional (3D) nonlinear dynamic model using the software ABAQUS including: 3D geometry, material models, pile-soil contact formulation, mesh properties, element type, applied ground motion, boundary conditions and analysis steps. A final verification of the developed model is demonstrated in terms of dynamic results (natural frequency and damping ratio) as well as structural responses (piles bending moments and deflections). In addition, steps involved in the calibration process of the model are presented to delineate the effect of varying some parameters on the responses of different parts of the numerical model.

Chapter 7 presents the results of an extensive parametric study on the effect of some geometric parameters of helical piles on the seismic responses of helical pile groups, which includes helix diameter, helix level, number of helices, spacing between helices as well as pile diameter.

Chapter 8 provides a summary of the research work performed as well as the main conclusions drawn from previous chapters. Finally, recommendations for future work are suggested.
Chapter 2

Literature Review

2.1 Axial Capacity of Helical Piles

Performance and design of single and grouped helical piles for axial loading have been well investigated. Many of previous studies demonstrated that the axial capacity of helical piles is equal to or greater than that of an equivalent driven pile for the same pile shaft, owing to the additional load carrying capacity offered by the helices. For example:

Livneh and El Naggar (2008) presented a detailed investigation into the axial performance of single helical piles in terms of 19 full-scale load tests in different soils and numerical modeling using Finite Element Analysis (FEM). They established correlations to relate the ultimate capacity to the installation torques and demonstrated that the load transfer to the soil is predominantly through a cylindrical shear failure mechanism.

Sakr (2009) reported the results of a comprehensive pile load-test program and observations from field monitoring of single helical piles with single helix or double helixes installed in oil sand. Their results confirmed that the helical pile is a viable deep foundation option for support of heavily loaded structures.

Sakr (2011) reported the first full-scale axial compression and tension (uplift) testing program executed on large capacity, single helical piles installed in cohesionless soils. Based on their results, it was found that helical piles have developed significant resistance to axial compressive loads up to about 2920 kN and tensile loads up to 2900 kN.

Tsuha et al. (2012) examined the effect of the number of helices on the performance of helical anchors in sand, based on the results of centrifuge model tests. The results of their investigation indicated that in double and triple helix anchors, the contributions of the second and third plate to the total anchor uplift capacity decreased with the increase of sand relative density and plate diameter.
Bagheri and El Naggar (2013, 2015) performed full-scale uplift and compression load tests on single helical piles installed in structured clays and sand. Their research findings indicated that the behavior of the helical piles and anchors is significantly affected by the degree of soil disturbance induced by penetration of pile shaft and helices.

Elkasabgy and El Naggar (2013, 2015) presented the results of full-scale static and dynamic vertical load tests on a 9.0 m single, large-capacity helical pile and a driven steel pile of the same length and shaft geometry, installed in cohesive soil. Their results showed that the dynamic behaviour of helical piles is essentially the same as that of driven steel piles with the same geometric properties. However, the helical piles developed static ultimate resistances up to 1.2–1.8 times that of the driven pile.

Elsherbiny and El Naggar (2013) investigated the compressive capacity of single helical piles in sand and clay by means of field testing and numerical modeling employing the computer program ABAQUS. They proposed a bearing-capacity reduction factor and a helix efficiency factor to evaluate the compressive capacity of helical piles in cohesionless soil.

Gavin et al. (2014) presented the results of compression and tension load tests performed on a single helical pile installed in dense sand, as well as results of numerical analyses using the software ABAQUS. Their results showed that substantial bearing and uplift pressures developed on the helix, which provided the majority of axial load resistance.

Ridgley (2015) summarizes design guidelines for calculating axial capacity of single helical piles.

Fahmy and El Naggar (2016a, 2017) studied the static and cyclic axial performance of tapered, single helical piles installed in sand. They found that tapered helical piles displayed a stiffer response and yielded higher capacities compared to the straight ones. In addition, they demonstrated that tapered single helical piles exhibited enhanced cyclic compressive performance compared to straight shaft piles.
Harnish and El Naggar (2017) established correlations between installation torques and large-diameter, single helical pile capacity by means of field pile load tests on helical piles installed in cohesive soils.

Nabizadeh and Choobbasti (2017) discussed the design considerations, installation procedures, and results of full-scale field load tests of single helical piles in sand and silty clay sands, under grouted and un-grouted conditions. Their results showed that in the silty clay soil, grouted and un-grouted helical piles had a similar performance while grouted piles showed greater axial compressive strength.

Schiavon et al. (2017, 2019) studied the cyclic and post-cyclic monotonic response of single helix anchor in sand using centrifuge tests. Their results indicated that although a rapid degradation of shaft resistance was observed in all tests, the soil loosened by the anchor installation was densified due to the cyclic loading improving the monotonic post-cyclic response. In addition, they found that the order of the cyclic sequences influences the displacement response and losses in the post-cyclic capacity.

Li et al. (2018) investigated the axial performance of large-diameter, single helical piles installed in lays and sands through field-testing. They found that the zone along the shaft where the shear resistance was not developed was estimated to be 2.5–5 times the helix diameter.

Pérez et al. (2018) performed a numerical and experimental study on the influence of installation effects on behaviour of helical anchors in very dense sand. They proposed two different approaches for improving the accuracy of analytical predictions of helical anchor capacity to account for disturbance during installation.

Lanyi-Bennett and Deng (2019) studied the axial performance of 2x2 helical pile groups and single helical piles installed in glaciolacustrine clay through field compressive load tests. Their results showed that the group interaction of small-diameter helical piles was lower than that of conventional pile groups.

Li and Deng (2019) explored the axial capacity of single helical piles in cohesive and cohesionless soils using axial load tests and numerical modeling. They found that the
torque factor was smaller for the larger pile shaft diameter in the homogeneous site, whereas in the heterogeneous site, it is substantially affected by soil heterogeneity around the helix. In addition, they reported that an ineffective length of four helix diameters could properly simulate the axial load versus displacement behavior.

Alwalan and El Naggar (2020a, 2020b) studied the performance of single helical piles under impact loading through analytical and numerical models. They proposed a method to approximate the increase in the pile impedance for a helical pile with single and double helices to account for the effect of helical plates. They also established guidelines for the effective design of High Strain Dynamic Load Test (HSDT) on single helical piles as well as on driven piles.

2.2 Lateral Capacity of Single Helical Piles

Motivated by the potential applications of helical piles to support wind turbine and offshore structures, the response of helical piles to lateral loading was the focus of many recent studies. Moreover, in seismic regions, helical piles are designed to sustain both axial and lateral loading. Several studies have evaluated the performance of full-scale helical piles installed in different types of soil conditions when subjected to static and dynamic lateral loading. Generally, the performance of helical piles under lateral loading was found to be primarily governed by the soil resistance along the upper portion of pile shaft, same as the case for driven piles. Some examples of studies of lateral performance of helical piles are given below.

Puri et al. (1984) developed a theoretical model estimating the lateral load capacity of helical anchor pile-soil systems taking into account the influence of the method of installation. Their results showed that the lateral capacity was completely governed by the extension section close to the soil surface (flexible piles).

The effect of lateral cyclic loading on the pull out capability of short, rigid, model, single helical piles in clayey soil was examined by Prasad and Rao (1994). Model steel jacked and helical piles were used in the experiment. Their findings demonstrated that lateral cyclic loading had no effect on the pullout capacity of helical piles but significantly reduced
the pullout capacity of jacked piles, as the axial capacity of helical pile is mainly provided by the helix compared to the pile shaft.

Prasad and Rao (1996) conducted lateral load experiments on reasonably rigid, model, single helical piles installed in clayey soils within a laboratory setting. They demonstrated that the static lateral capacity of the helical pile was higher than a straight shaft pile, and that the lateral capacity increased as the number of helices increased. They developed a simple theoretical model to estimate the lateral capacity of single helical piles.

Sakr (2009) conducted full-scale lateral load tests on single helical piles placed in oil sands. The results demonstrated that the size of the shaft played a major role in the lateral behaviour of the test helical piles. Furthermore, an increase in the number of helices resulted in a loss in the helical piles' lateral capacity due to the disturbance during installation resulting from advancement of helical plates into the soil.

Qin and Guo (2014) evaluated the behaviour of model-scale, rigid, single helical piles installed in sand under cyclic loads. They reported that the load level had a more significant influence on the response than the number of cycles. They also showed that the maximum bending moment and applied lateral load are linearly correlated regardless of the number of load cycles applied.

Fahmy and El Naggar (2016b) investigated the cyclic behaviour of large full-scale, tapered, single helical piles installed in silty sand. They found that the helices increased the capacity for short piles as well as both tapered and straight helical piles performed similarly satisfactory under lateral loading.

Elsherbiny et al. (2019) studied the effects of pile installation on the static and dynamic lateral stiffness of helical piles installed in dense sand. Full-Scale Tests were performed on single-helix and double-helix piles, under monotonic and low-strain dynamic lateral loads. It was found that the effects of installation disturbance appeared to be negligible. However, the installation disturbance manifested by the formation of a gap between the pile shaft and soil can cause a reduction in dynamic stiffness of helical piles.
El Sharnouby and El Naggar (2018a) presented a field study on the lateral monotonic and cyclic behaviour of steel, fibre–reinforced helical pulldown micropiles. They found that grouted shaft and/or fibre-reinforced polymer (FRP) sleeve significantly improved the helical pile’s lateral performance. In addition, the piles showed a significant ductility, no observed failure up to 75 mm displacement.

Elkasabgy and El Naggar (2018, 2019) investigated the lateral performance of large-capacity full-scale, single helical piles installed in structural clayey soils. Lateral load test results showed that the dynamic behavior of helical piles was essentially similar to that of the driven pile, with insignificant effect from helices. In addition, the piles exhibited slight to moderate nonlinear behavior. They also concluded that pile installation had a significant effect on the subgrade modulus. The number of helices and the inter-helix spacing ratio had minimal influence on the mobilized load deflection at pile head and soil resistances.

These studies helped understand the lateral performance of single helical piles under different loading conditions. However, most of the full-scale cyclic and dynamic testing of the aforementioned studies, loads were applied at the pile head using mechanical actuators/shakers, which only represents the inertial loading conditions. Thus, these studies did not account for the kinematic soil-pile interaction. These alternative loading approaches have been pursued because large shake table testing opportunities are rare and expensive. In addition, Fleming et al. (2016) and Elkasabgy and El Naggar (2018) argued that scaled models and field-scale cyclic and dynamic loading applied to the pile head still allow reasonable simulations of dynamic behaviour.

2.3 Seismic Performance of Pile Groups

The pile behaviour within a group can be vastly different from that of a similar single pile. When piles are connected to a common cap and act as a group, the Pile–Soil–Pile Interaction (PSPI) may reduce their lateral resistance compared to that of individual piles, resulting in the group having less lateral resistance than the sum of the individual piles' lateral resistances. However, the contribution of the rocking resistance developed by the induced normal forces in piles increases the total lateral resistance of pile groups, and may governs the resistance in dynamic and seismic loading.
The seismic response of pile groups is governed by their dynamic characteristics, which are influenced by the Soil-Structure Interaction (SSI) (Stewart et al. 2012). The PSPI and group effects are governed by the soil and pile properties as well as the group configuration (Miura 1997, Boulanger et al. 1999, Jeremić et al. 2009). In addition, the characteristics of the supported superstructure may influence the SSI effects and even govern the response (Badry and Satyam 2017). These effects are generally significant for rigid structures supported by relatively flexible foundations, and an overall dynamic analysis of the whole Soil-Foundation-Structure system should be considered (Mylonakis and Gazetas 2000, Givens et al. 2012, Carbonari et al. 2017, Michel et al. 2018).

Furthermore, due to the nature of the earthquake loading, piles and pile groups experience kinematic effects due to the movement of the soil as well as traditional inertial effects due to the mass of the superstructure. The PSPI under earthquake loading have been examined through scaled models (i.e., 1g and centrifuge models) in controlled laboratory setting (Moss et al. 1998, Bhattacharya et al. 2012, Motamed et al. 2013, Al-Baghdadi et al. 2015, Newgrad et al. 2019). However, Kagawa et al. (2004) listed several limitations of centrifuge simulation of seismic response of Soil-Pile-Structure systems, including scaling problems related to the finite grain size, variation of centrifugal acceleration within the model and boundary effects.

Albeit expensive, large shake table facilities with large capacity actuators allow testing of almost full-scale piles and pile groups, and provide the most realistic approach for simulating of seismic loading on pile groups. There are a few well-documented seismic studies of pile groups employing full-scale shake table tests in the literature, for example:

Meymand et al. (2000) investigated the soil foundation-single pile/pile group-superstructure interaction through shaking table tests, in which the effects of kinematic and inertial interaction were demonstrated. They concluded that the derived p-y curves from the test data provided a suitable basis for analytical simulation of the seismic behaviour of single and grouped piles.

Shirato et al. (2008) examined the seismic response of a 3x3 pile group embedded in dry sand employing large-scale shake table testing where they concluded that the group effect
could be expressed solely in terms of the position of the pile within the pile group. They proposed a method to incorporate group effects during large-intensity earthquakes into any derived p-y curves.

Su et al. (2015) conducted shaking table tests for pile groups in liquefiable sites with medium and dense sand layers and compared the dynamic response characteristics of the piles and the soil. They concluded that responses of piles and superstructure were larger in liquefied dense sand than in liquefied medium-dense sand.

Wang et al. (2019) conducted a comparative study of the seismic behaviour of pile groups installed in non-liquefiable and liquefiable soils employing shake table testing and observed different failure mechanisms for piles in both sites. They concluded that the pile group effects are significant in the non-liquefiable soils while it was relatively negligible in liquefied soils.

Xu et al. (2020a, 2020b) evaluated the response and failure mechanism of a large-model pile group comprising of 2x2 piles installed in liquefiable and non-liquefiable sand and highlighted the differences in responses and failure mechanisms for the piles in different soil conditions.

Zhang et al. (2020) investigated the dynamic interaction of soil-piles-frame-structure from shaking table experiments and evaluated the variation of foundation and structure dynamic characteristics with the intensity of shaking.

None of the above-mentioned studies examined the seismic performance of grouped helical piles, neither through scaled models nor by full-scale shake table testing.

2.4 Seismic Performance of Helical Pile Groups

To address the knowledge gap mentioned before, and to develop suitable guidelines for application of helical pile groups as a viable option in seismic prone regions, a comprehensive, large-scale, shake-table testing program was undertaken to evaluate the seismic response of single helical piles and helical pile groups, which is considered the first of its kind. Some aspects of the testing program have been previously published.
ElSawy et al. (2019b, 2019a) evaluated the seismic performance of single helical piles in comparison with steel driven piles. They presented detailed descriptions of the data reduction methods for interpreting the measured responses during the tests. They concluded that the helical piles have the ability to perform as good as conventional piles under seismic loading.

Shahbazi et al. (2020a) investigated the damping characteristics of grouped helical piles in dense sands under small and large shaking events. Their results demonstrated that individual piles behave flexibly with high damping compared to group piles, which exhibit higher energy dissipation. They also concluded that a direct relation between the slenderness ratio of piles and the provided damping was observed.

Shahbazi et al. (2020b) analyzed the experimental observations to evaluate the structural natural frequency and pile-group stiffness, and used the experimental results to calibrate a numerical model that was then used to conduct a parametric study to gain a broader understanding of the seismic behavior of structures supported by helical pile groups under varying conditions.

2.5 Summary

This chapter presented a brief summary of the available studies on the static and cyclic axial performance of single and grouped helical piles. In addition, lateral performance of single helical piles under static and cyclic loading was reviewed. Furthermore, factors affecting seismic response of pile groups were presented, in which it was demonstrated that full-scale testing provides the most realistic representation of seismic loading and is an excellent means to investigate the seismic performance of pile groups. Examples of well-documented, full-scale shake table seismic testing of pile groups were covered.

This literature review collectively indicates the clear advantages of helical piles and their significant potential as a viable and efficient foundation option in seismic regions. However, there is a lack of realistic full-scale testing studies for grouped helical piles involving actual seismic loading. This provided the motivation to undertake the first full-scale shake table testing program to evaluate the seismic responses of helical piles.
Chapter 3

Full-Scale Shaking Table Experimental Program

3.1 Introduction

A full-scale shaking table experimental program was conducted to evaluate the seismic performance of single and grouped helical piles. The test program was performed employing the Large High Performance Outdoor Shake Table (LHPOST) located at The University of California – San Diego (UCSD). The LHPOST is situated in the Englekrik Structural Engineering Research Center (ESEC) at Camp Elliott, which is located 15 km away from the main UCSD campus.

The LHPOST is the largest outdoor shaking table in the world outside of Japan. It allows accurate reproduction of severe earthquake ground motions for seismic testing of full-scale structures and Soil-Foundation-Structure Interaction (SFSI). In addition to the LHPOST, the ESEC has an SFSI facility that includes a large laminar soil shear box and a refillable soil pit. This shear box is mounted on the shaking table to provide a means for testing geotechnical systems.

3.2 Description of the Testing Facility

3.2.1 Shaking Table

The shaking table is 7.6 m long x 12.2 m wide, and can handle up to 20 MN vertical payload. It is powered by two horizontal actuators with a force capacity of 6.8 MN, which can produce a maximum stroke of ± 0.75 m. The peak acceleration that can be produced is 4.2 g for the bare table and 1.2 g for 400 tons payload. A maximum peak velocity of 1.8 m/s can be achieved, which facilitates simulation of representative near-source ground shaking records. This value is amplified when the laminar shear box is mounted on the table, and the maximum velocity on the soil surface can even exceed 1.8 m/s. The frequency bandwidth of the shaking table is (0 – 33) Hz. An overview of the LHPOST facility is shown in Figure 3-1.
3.2.2 Laminar Soil Shear Box

The dimensions of the laminar soil shear box are 6.71 m x 2.9 m x 4.78 m (L x W x H). It consists of 31 steel laminar frames, each separated by a steel roller system on stainless steel lined webs, to allow for uni-directional movement. This movement provides a mechanism by which energy propagated through the soil can be absorbed, and hence, reduces the effect of wave reflection from the boundary. This simulates in-situ conditions in which energy can propagate through a uniform soil deposit with minimal energy reflection.

The laminar frames comprised nine frames of W8x35 steel section in the lower region, sixteen frames of W8x15 steel section in the mid height region and six frames of W8x10 steel section in the upper region. This variation in frames cross-section minimizes the box’s weight, so that the ratio of laminar frame to soil weight is between 8 to 15 %. Displacement of frames in the direction perpendicular to the motion is restrained by two steel towers that are designed to have a natural frequency 2.5 times that of the soil box. Figure 3-2 presents an overview of the laminar soil shear box, as well as the height of the sand bed used in the testing.

3.3 Sand Bed Preparation

The soil used in the testing program was well-graded sand. The sand bed had the same length and width of the laminar shear box, and a height of 4.57 m. The total height of sand
bed was achieved through layers of less than 30 cm thickness, where each layer was highly compacted before starting the next one. This resulted in a very dense soil with approximately 90% relative density in lower soil layers, and a less dense soil in the top layers.

![Figure 3-2. Overview of the laminar soil shear box](image)

3.3.1 Laboratory Tests

3.3.1.1 Soil Classification

Sieve analysis was performed according to ASTM C136 (2014) to determine the particle size distribution. Figure 3-3 shows the particle size distribution of a sand sample from the testing bed. Results of the test show that the average grain size, $D_{50} = 0.83 \text{ mm}$, the effective grain size, $D_{10} = 0.15 \text{ mm}$, the 60% finer grain size, $D_{60} = 1.13 \text{ mm}$, the 30% finer grain size, $D_{30} = 0.42 \text{ mm}$, the fines content, $FC = 4.54\%$ and 100% passes through sieve #4. Thus, the uniformity coefficient, $C_u = 7.53$, and coefficient of curvature, $C_c = 1.04$. Hence, the sand was classified as well-graded sand (SW) according to the Unified Soil Classification System (USCS), ASTM D2487 (2011).
3.3.1.2 Soil Material Properties

A number of tests were performed on soil samples to determine some basic material properties. All tests were conducted according to the corresponding ASTM standard. In addition, Poisson’s ratio was determined from the typical recommended values for dense sands proposed by Bowles (1988). Table 3-1 presents the results of these tests.

Table 3-1. Sand material properties

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Standard</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight, $\gamma$ (kN/m$^3$)</td>
<td>D7263 (2009)</td>
<td>19.5</td>
</tr>
<tr>
<td>Specific gravity, $G_s$</td>
<td>D854 (2014)</td>
<td>2.67</td>
</tr>
<tr>
<td>Water content, $w_c$ (%)</td>
<td>D2216 (2010)</td>
<td>6.00</td>
</tr>
<tr>
<td>Maximum void ratio, $e_{max}$</td>
<td>D7263 (2009)</td>
<td>0.74</td>
</tr>
<tr>
<td>Minimum void ratio, $e_{min}$</td>
<td>D7263 (2009)</td>
<td>0.47</td>
</tr>
<tr>
<td>Poisson’s ratio, $\nu$</td>
<td>-</td>
<td>0.40</td>
</tr>
</tbody>
</table>

3.3.1.3 Soil Shear Strength

Direct shear testing was performed according to ASTM D3080 (2011) on three soil samples in order to determine the soil shear strength parameters. The three tests were performed
under different normal stresses: 50 kPa, 100 kPa and 200 kPa. All soil samples were prepared in a way so that they match the in-situ soil conditions including relative density and water content. The shearing rate was kept constant at 0.3 mm/min for all tests. **Figure 3-4** shows the resulted shear stress for different normal stresses, as well as the determined peak and residual friction angles, while **Figure 3-5** presents the resulted horizontal and vertical displacements for different confining pressures.

Results of the direct shear testing showed that the peak and residual internal friction angles of the sand were $48^\circ$ and $47^\circ$, respectively. The dilation angle was then calculated according to **Equation (3-1)**, which yielded $1.25^\circ$.

$$\phi_p = \phi_r + 0.8 \psi$$  \hspace{1cm} (3-1)

**Figure 3-4. Direct shear test result: a) Shear stresses; b) Friction angles**

### 3.3.2 Sand Bed Testing

A Dynamic Cone Penetration test (DCP) was performed in the center of the sand bed before starting the experimental program. The number of blows per incremental depth of 51 mm was recorded. The DCP Penetration Index (DPI) was then calculated from **Equation (3-2)**.
Figure 3-5. Direct shear test results; horizontal and vertical displacements

\[
DPI \ (mm/blow) = \frac{\text{Incremental Depth}}{\text{Number of Blows}} \tag{3-2}
\]

The DPI was calculated based on the 51 mm increment and an average value of 305 mm increment. Since the soil has been constructed through layers and compacted separately, it was found that an average value for the layer depth is more convenient and reliable to use in the correlations. Moreover, building thin individually compacted soil layers causes the material closer to surface to be less confined and less compacted than deeper layers. Therefore, it is often advised to ignore the first 2 to 3 drops (seating drops) when calculating the DPI. Results of the test are presented in Figure 3-6a, which shows that the soil depth can be generally divided into two layers: the top 1 m (layer 1) and the bottom 3.5 m (layer 2). In layer 1, the DPI values are relatively high and linearly decreasing towards layer 2, where it became nearly constant through the rest of the soil depth.

The properties of the sand bed can be directly correlated to the DPI values, including: relative density, \(D_r\), friction angle, \(\phi\) and shear modulus, \(G_s\). The correlations implemented were proposed by (Mohammadi et al. 2008), in which they used a statistical
approach to develop the correlations with a high coefficient of determination ($R^2 > 0.9$). These correlations are presented in **Equation (3-3)** to **Equation (3-6)**.

**Figure 3-6. DCP test: a) DPI; b) Relative density correlation**

$D_r (%) = 189.93/(DPI)^{0.53}$ \hfill (3-3)

$\phi (\text{deg}) = 26.31 + 0.21 \times (D_r)$ \hfill (3-4)

$\phi (\text{deg}) = 52.16/(DPI)^{0.13}$ \hfill (3-5)

$G_s (\text{MPa}) = 75.74/(DPI)^{0.99}$ \hfill (3-6)

**Figure 3-6b** shows the variation of the relative density with depth obtained from **Equation (3-3)**. It can be seen that the relative density increased from 62 % near the surface to 92 % at 1 m below surface then remains nearly constant.

**Figure 3-7** presents the soil’s maximum shear modulus (low strain) and peak angle of internal friction. It is noted from **Figure 3-7a** that the variations of friction angle from both correlations, i.e. **Equation (3-4)** and **Equation (3-5)**, were in good agreement, and close to the value measured from the direct shear tests.
The shear wave velocity \( V_s \) and the Young’s Modulus \( E_s \) were calculated from \( G_s \) according to Equation (3-7) and Equation (3-8), respectively, in which \( \rho_s \) is the soil’s density and \( v \) is Poisson’s ratio.

\[
V_s \ (m/s) = \sqrt{\frac{G_s}{\rho_s}} \tag{3-7}
\]

\[
E_s \ (MPa) = 2 \ G_s \ (1 + v) \tag{3-8}
\]

Figure 3-8 presents the variation of \( V_s \) and \( E_s \) with depth, while Table 3-2 provides a summary of the determined soil properties. The values of \( V_s \) obtained from this correlation was close to the value obtained from the analysis of soil accelerometers data obtained during the shaking tests as will be discussed in next chapter.

3.4 Test Program and Piles’ Properties

The test program was conducted over five days. On each day, a different setup of piles and/or soil were subjected to series of different ground motions, which included pulse waves, white noise signals and seismic ground motions. On Testing Day 1 (TD1), the sand
bed was subjected to two pulses and a white noise motion to establish the dynamic properties of the soil. On Testing Day 2 (TD2), eight circular and one square helical piles with different properties including the length, radius and number of helices, as well as one driven circular pile were installed and subjected to different excitations without any inertial masses. This was done to examine the kinematic interaction of the soil-pile system. Table 3-3 shows the geometric properties of the various test piles.

![Figure 3-8. DCP correlations: a) Shear wave velocity; b) Young’s modulus](image)

<table>
<thead>
<tr>
<th>Property</th>
<th>Layer 1</th>
<th>Layer 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>1</td>
<td>3.57</td>
</tr>
<tr>
<td>DPI (mm/blow)</td>
<td>10.5 – 4.2</td>
<td>4.2</td>
</tr>
<tr>
<td>Dr (%)</td>
<td>55 – 90</td>
<td>90</td>
</tr>
<tr>
<td>$\phi$ (deg)</td>
<td>38 – 45</td>
<td>45</td>
</tr>
<tr>
<td>$G_s$ (MPa)</td>
<td>8 – 19</td>
<td>19</td>
</tr>
<tr>
<td>$V_s$ (m/s)</td>
<td>61 – 97</td>
<td>97</td>
</tr>
<tr>
<td>$E_s$ (MPa)</td>
<td>21 – 53</td>
<td>53</td>
</tr>
</tbody>
</table>
**Table 3-3. Geometric properties of test piles**

<table>
<thead>
<tr>
<th>Pile Group</th>
<th>Pile</th>
<th>Type*</th>
<th>Total Length / Depth (m)</th>
<th>Helix Level / Helix Diameter (m)</th>
<th>Outer Diameter / Wall Thickness (mm)</th>
<th>Yield Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PG1</td>
<td>P1</td>
<td>C-H</td>
<td>3.96 / 3.66</td>
<td>-3.51 / 0.254</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P2</td>
<td>C-H</td>
<td>3.66 / 3.35</td>
<td>-3.20 / 0.254</td>
<td>88 / 5.3</td>
<td>448.2</td>
</tr>
<tr>
<td></td>
<td>P3</td>
<td>C-H</td>
<td>3.66 / 3.35</td>
<td>-3.51 / 0.254</td>
<td>88 / 5.3</td>
<td>448.2</td>
</tr>
<tr>
<td></td>
<td>P4</td>
<td>C-HH</td>
<td>3.66 / 3.35</td>
<td>-2.59 / 0.254</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P5</td>
<td>C-D</td>
<td>3.66 / 3.35</td>
<td>-</td>
<td>88 / 5.3</td>
<td>448.2</td>
</tr>
<tr>
<td></td>
<td>P6</td>
<td>S-H</td>
<td>3.66 / 3.35</td>
<td>-3.20 / 0.254</td>
<td>76** / 5.3</td>
<td>413.7</td>
</tr>
<tr>
<td>PG2</td>
<td>P7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P8</td>
<td>C-H</td>
<td>4.22 / 3.35</td>
<td>-3.20 / 0.254</td>
<td>140 / 10.5</td>
<td>551.6</td>
</tr>
<tr>
<td></td>
<td>P9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* C (circular), S (square), H (single helical pile), HH (double helical pile), D (driven pile)

** For square pile, 76 mm is the side length

On Testing Day 3 (TD3), different masses were placed on top of each pile to simulate inertial masses, and examine the inertial interaction of pile-soil systems. On Testing Day 4 (TD4), two pile groups were formed by adding a steel skid filled with sand on top of two groups of four helical piles of the same diameter. For Pile Group 1 (PG1), the diameter of piles was 88 mm, and the steel skid weight was 62 kN. For Pile Group 2 (PG2), the diameter of piles was 140 mm, and the steel skid weight was 98 kN. For both groups, the skid was connected to each pile head using two bolts to simulate a fixed head condition. On Testing Day 5 (TD5), one bolt from each connection was removed in order to simulate a pinned head condition. Table 3-4 provides details of masses added to each pile head on TD3 as well as weights of steel skids used on TD4 and TD5.

Piles were installed at a minimum spacing of 0.91 m center-to-center, which corresponded to a spacing to diameter ratio (S/D) of 6.5. This ratio is sufficient to minimize Pile-Soil-Pile Interaction (PSPI) effects. Figure 3-9 shows the arrangement of piles in the laminar shear box, while Figure 3-10 shows the layouts of pile caps for PG1 and PG2.
Table 3-4. Added masses for single piles and pile groups

<table>
<thead>
<tr>
<th>Pile Group</th>
<th>Pile</th>
<th>Single Piles</th>
<th>Pile Groups</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Total Mass (kg)</td>
<td>Center of Mass to Ground Level (m)</td>
</tr>
<tr>
<td>PG1</td>
<td>P1</td>
<td>768</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>P2</td>
<td>749</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>P3</td>
<td>777</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>P4</td>
<td>748</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td>P5</td>
<td>370</td>
<td>0.57</td>
</tr>
<tr>
<td></td>
<td>P6</td>
<td>433</td>
<td>0.60</td>
</tr>
<tr>
<td>PG2</td>
<td>P7</td>
<td>1236</td>
<td>1.71</td>
</tr>
<tr>
<td></td>
<td>P8</td>
<td>785</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td>P9</td>
<td>701</td>
<td>1.40</td>
</tr>
<tr>
<td></td>
<td>P10</td>
<td>1244</td>
<td>1.74</td>
</tr>
</tbody>
</table>

Figure 3-9. Arrangement of piles in laminar shear box
3.5 Piles Installation and Instrumentation

Piles were installed in the sand bed using the same method of installation employed in the field. Mechanical torque was applied at the pile head to drive the pile into the soil. The torque profiles for piles of the same geometric properties were consistent and linearly increasing with depth as shown in Figure 3-11, which indicated that the soil is uniform and homogenous within the whole sand bed. These torque profiles could be used to determine the capacity of the helical piles using capacity-to-torque correlations, e.g. (Livneh and El Naggar 2008, Perko 2009, Harnish and El Naggar 2017).

In order to capture the dynamic behavior of piles and pile groups, a number of strain gauges and accelerometers were used. Each pile had six or seven pairs of strain gauges installed at different levels along the pile. In addition, an accelerometer was installed at each pile head to capture pile head movement. For pile groups, each pile cap had two accelerometers placed at the center of gravity level. The strain gauges were of type YFLA-5-5LT manufactured by Texas Measurements Inc. with 120 Ω electrical resistance.

Furthermore, 32 single axis accelerometers were placed within the sand bed to capture the movement of the soil at different elevations. The accelerometers were placed along three...
arrays: east, center and west. Two different models of accelerometers (352M54 and 355M69) were used and both were supplied by PCB Piezotronics.

Figure 3-11. Installation torque profiles: a) PG1 (P1 to P4); b) PG2 (P7 to P10)

The laminar soil shear box had 24 accelerometers as well as 15 String Potentiometers (SPs) installed on the steel frames at different levels. This arrangement allowed monitoring the soil box movement, which would be considered as the far field response during analysis of low amplitude shaking.

All instrumentations were linked to a compact Data Acquisition System (DAQ) with main chassis of model NI-cDAQ-9188 by National Instruments, capable of measuring up to 256 channels. A sampling rate of 240 Hz was used to record strain gauge signals and all accelerometer records, except for pulse tests, where a sampling rate of 3000 Hz was used for soil accelerometers.

Instrumentation of piles monitored during TD4 and TD5 are shown in Figure 3-12 and Figure 3-13 for north piles and south piles, respectively. In addition, Figure 3-14 shows a layout for sand bed and laminar shear box accelerometers, while Figure 3-15 presents the distribution of the SPs along the laminar shear box.
Figure 3-12. North piles instrumentation

Figure 3-13. South piles instrumentation
### Figure 3-14. Soil and shear box accelerometers

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Accelerometers</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.2286</td>
<td>A101E</td>
</tr>
<tr>
<td>-0.5334</td>
<td>A102E</td>
</tr>
<tr>
<td>-0.8382</td>
<td>A103E</td>
</tr>
<tr>
<td>-1.1430</td>
<td>A104E</td>
</tr>
<tr>
<td>-1.4478</td>
<td>A105E</td>
</tr>
<tr>
<td>-1.7526</td>
<td>A106E</td>
</tr>
<tr>
<td>-2.0574</td>
<td>A107E</td>
</tr>
<tr>
<td>-2.3622</td>
<td>A108E</td>
</tr>
<tr>
<td>-2.6670</td>
<td>A109E</td>
</tr>
<tr>
<td>-2.9718</td>
<td>A11E</td>
</tr>
<tr>
<td>-3.2766</td>
<td>A12E</td>
</tr>
<tr>
<td>-3.5814</td>
<td>A13E</td>
</tr>
<tr>
<td>-3.8862</td>
<td>A14E</td>
</tr>
<tr>
<td>-4.1910</td>
<td>A15E</td>
</tr>
</tbody>
</table>

### Figure 3-15. String Potentiometers (SPs) layout

[Diagram of String Potentiometers (SPs) layout]
3.6 Input Ground Motions

Different ground motions were applied to the shaking table on each testing day. These included pulse waves, white noise signals and seismic ground motions.

3.6.1 Pulse Waves

Pulse waves used in the test program had a very low amplitude ranging from 0.035 g to 0.057 g. They were applied at the base of the sand bed at the beginning of each testing day, after each test and at the end of each testing day in order to determine the dynamic properties of different systems and monitor the variation of these properties throughout the testing program. Pulse waves were used to determine the shear wave velocity, natural frequency and damping ratio of the sand bed as well as the natural frequency of piles. **Figure 3-16** presents an example of the first pulse wave applied to the sand bed on TD1.

![Figure 3-16. Pulse wave: a) Acceleration time history; b) Frequency content; c) 5% damped spectral acceleration](image)
3.6.2 White Noise Signals

The sand bed was subjected to white noise excitation once at the start of each testing day in order to compute the natural frequencies of the sand bed, as well as piles and pile groups. This also facilitated monitoring the variation in the natural frequency due to different soil and/or pile configurations during the testing program. The applied white noise signal was a random signal with nearly constant frequency amplitude for a range of frequencies. The signal used had a bandwidth of (0 – 40) Hz, a Root Mean Square (RMS) amplitude of 0.05 g or 0.07 g and a duration ranging from 60 s to 120 s. The peak acceleration was chosen in a way that it just produced deformations without causing any non-linear behavior in the soil or piles, where it ranged from 0.16 g to 0.29 g. Figure 3-17 shows the white noise signals used during the test program. The notation used is “WN-RMS (g)-duration (s)”.

![Figure 3-17. White noise signals: a) Acceleration time history; b) Frequency content; c) 5% damped spectral acceleration](image)
3.6.3 Earthquake Records

The ground motion records used were the Northridge (1994) earthquake recorded at Fire Station 108, USC station 5314, California, United States, and the Kobe (1995) earthquake recorded at Takatori Station, Takatori, Japan. In this study, they will be referred to as NOR and TAK. In addition to the two original earthquake ground motions, the time history of each earthquake record was scaled to 75% and 50% and applied to the sand bed in order to evaluate responses of piles and pile groups for a range of load intensities. Figure 3-18 and Figure 3-19 show the original ground motion record for NOR and TAK earthquakes, respectively, as well as the scaled motions. The notation used is “Earthquake-Amplitude (\%)\text{-}T0”.

![Figure 3-18. NOR earthquake records: a) Acceleration time history; b) Frequency content; c) 5% damped spectral acceleration](image-url)
Two more records were also applied to the sand bed. NOR earthquake time history was compressed with a ratio of 2.5 and applied to the sand bed, which would be referred to as T1 and T0 for the compressed and original records, respectively. This was planned to demonstrate the effect of time duration of the earthquake, however, a huge reduction (about 40 %) in the frequency content resulted from this compression as can be seen in Figure 3-20. In addition, the energy content extended on a wider range of frequencies, 0.5 Hz to 20 Hz, which made it impossible to compare responses from both earthquakes. Moreover, a 150 % amplitude of NOR earthquake was applied as the last test on TD5. This was decided to be the last test as failure of pile groups could have happened due to this very large amplitude.

Figure 3-19. TAK earthquake records: a) Acceleration time history; b) Frequency content; c) 5% damped spectral acceleration
Figure 3-20. Original and compressed NOR earthquake records: a) Acceleration time history; b) Frequency content; c) 5% damped spectral acceleration

Figure 3-21 compares the two original records for NOR and TAK earthquakes. As noted from Figure 3-21, TAK earthquake record had a very high amplitude with an energy content concentrated in the range of 0.5 Hz to 2.5 Hz, while NOR earthquake’s amplitude was nearly 80% lower with a range of frequency of 2.5 Hz to 5 Hz. In addition, the Peak Ground Acceleration (PGA) of TAK earthquake was almost twice that of NOR earthquake.

Table 3-5 presents the ground motions applied on each testing day as well as the order through which they were applied, while Table 3-6 presents a summary of the absolute PGA for different the different ground motions.
3.7 Data Preparation

To prepare the experimental data for analysis, two procedures were needed to improve the signal to noise ratio and ensure that the results are reliable and consistent. First, removing the offsets and de-trending the data to remove low frequency noise. Second, applying a filter to the data to remove high frequency noise. Removing offsets and de-trending could be achieved by subtracting the mean value of the data and/or subtracting the mean value of a fitted polynomial with a desired degree to the data. However, both could be done using a high pass Butterworth filter. On the other hand, a low pass Butterworth filter could be employed to remove any high frequency noise beyond the range of frequency under consideration. Hence, a fourth-order, band-pass, Butterworth filter was used in this study.
### Table 3-5. Testing program plan

<table>
<thead>
<tr>
<th>TD1</th>
<th>TD2</th>
<th>TD3</th>
<th>TD4</th>
<th>TD5</th>
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<td>WN-0.07-120</td>
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<td>WN-0.05-60</td>
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<td>NOR-100-T1</td>
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</tr>
<tr>
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<td>NOR-50-T0</td>
<td>NOR-50-T0</td>
<td>NOR-50-T0</td>
</tr>
<tr>
<td>Pulse</td>
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<td>NOR-75-T0</td>
<td>NOR-75-T0</td>
<td>NOR-75-T0</td>
</tr>
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<td>Pulse</td>
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</tr>
<tr>
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<td>TAK-50-T0</td>
</tr>
<tr>
<td>Pulse</td>
<td>TAK-75-T0</td>
<td>TAK-75-T0</td>
<td>TAK-75-T0</td>
<td>TAK-75-T0</td>
</tr>
<tr>
<td>Pulse</td>
<td>TAK-100-T0</td>
<td>TAK-100-T0</td>
<td>TAK-100-T0</td>
<td>NOR-150-T0</td>
</tr>
</tbody>
</table>

### Table 3-6. Absolute PGA for different ground motions

<table>
<thead>
<tr>
<th>Record</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pulse</td>
<td>0.035 – 0.057</td>
</tr>
<tr>
<td>WN-0.07</td>
<td>0.29</td>
</tr>
<tr>
<td>WN-0.05</td>
<td>0.16 – 0.20</td>
</tr>
<tr>
<td>NOR-100-T0</td>
<td>0.47 – 0.48</td>
</tr>
<tr>
<td>NOR-75-T0</td>
<td>0.35</td>
</tr>
<tr>
<td>NOR-50-T0</td>
<td>0.22 – 0.23</td>
</tr>
<tr>
<td>NOR-150-T0</td>
<td>0.74</td>
</tr>
<tr>
<td>NOR-100-T1</td>
<td>0.39 – 0.40</td>
</tr>
<tr>
<td>TAK-100-T0</td>
<td>0.68 – 0.70</td>
</tr>
<tr>
<td>TAK-75-T0</td>
<td>0.47 – 0.48</td>
</tr>
<tr>
<td>TAK-50-T0</td>
<td>0.30 – 0.32</td>
</tr>
</tbody>
</table>
It should be noted that the most important factor in filtering the data is selecting an appropriate cut-off frequency (i.e., corner frequency of the filter) as the filtered data were very sensitive to the used cutoff frequency. Herein, the cut-off frequency was chosen based on the Power Spectral Density (PSD) of the input signals. The point of 99% cumulative PSD was taken as the cutoff frequency as demonstrated in Figure 3-22 for TAK-100-T0 earthquake record.

![Figure 3-22. Corner frequency of low pass filter for TAK-100-T0](image)

Filtering was performed through MATLAB (The MathWorks Inc. 2016) using the “filtfilt” function, which is a zero phase digital filtering function. This means that data is filtered in both forward and backward directions, which ensures that there is no phase distortion in the data.

### 3.8 Summary

An experimental program was performed employing the LHPOST at UCSD, where single helical piles and helical pile groups were subjected to different ground motions. The testing program was conducted over five days. In each day, a different setup of piles and pile groups was tested. In addition, laboratory tests were performed on soil samples in order to
characterize the soil and determine its properties. Furthermore, a DCP field test was performed in the sand bed, and the results demonstrated that the soil strength and stiffness increase linearly with depth up to 1 m below surface then remain constant afterwards. Different properties of the sand bed were correlated to the DCP results, including relative density, friction angle, shear modulus, shear wave velocity and elastic modulus. In addition, the measured torques during piles’ installation revealed the uniformity and consistency of the sand bed.

Different instrumentations were installed along the piles including strain gauges and accelerometers, in order to monitor their dynamic behavior during testing. Moreover, the sand bed and the laminar shear box were also instrumented with a number of accelerometers and SPs to monitor the soil movement. Different ground motions were applied to the soil block that include pulse waves, white noise signals and earthquake ground records.

A scheme for data preparation was applied to the collected test data. This included removing low frequency noise by using a 4th order high pass Butterworth filter, as well as removing high frequency noise through a 4th order low pass Butterworth filter. The corner frequency of the low pass filter was determined based on the cumulative PSD. A zero phase digital filtering MATLAB function was used to prevent any data distortion.
Chapter 4

Experimental Results: Dynamic Properties of Pile-Soil Systems

4.1 Introduction

In this chapter, dynamic characteristics of sand bed and its natural frequencies as well as natural frequencies of single and grouped helical pile-soil systems evaluated from the collected data during different shaking events are presented. Different analysis methods are presented and compared. Moreover, the effects of different pile configurations as well as successive shakings on the natural frequencies of the sand bed and different pile-soil systems were also investigated.

4.2 Dynamic Properties of Sand Bed

4.2.1 Shear Wave Velocity

Pulse waves were applied prior to each shaking test and their measurements were used to calculate shear wave velocity of the soil mass. A peak-to-peak procedure was employed in which the propagation time between each two successive accelerometers is taken as the difference between peak arrival times. Hence, the shear wave velocity of each layer (i.e. between each two successive accelerometers) is calculated from Equation (4-1), where \( V_{si} \) is the shear wave velocity of layer \( i \), \( h \) is the elevation of the accelerometer, \( t \) is the peak arrival time at the accelerometer and subscripts \( u \) and \( l \) refer to the upper and lower accelerometers in the layer, respectively.

\[
V_{si} = \frac{h_u - h_l}{t_u - t_l}
\]  

(4-1)

Figure 4-1a displays an example of travelling waves in the east array of accelerometers for the first pulse on TD1. As can be observed form Figure 4-1a, the input pulse was amplified and the peak arrival was delayed as it traveled through the soil bed. The peak acceleration recorded at the top surface of the soil was nearly twice the input motion as demonstrated in Figure 4-1b.
Shear wave velocity profiles were calculated from the three accelerometer arrays, east, center and west, and then an average profile was computed. Figure 4-2 shows an example of shear wave profiles for the first pulse on TD1.

A weighted average for the shear wave velocity through the depth of the sand bed can be calculated using Equation (4-2), where, $H$ is the total depth of the soil, $h_i$ and $V_{s,i}$ are the height and shear wave velocity of each layer, respectively.

$$
\bar{V}_s = \frac{H}{\sum_{i=1}^{n} \frac{h_i}{V_{s,i}}}
$$

An approximate value for the $n^{th}$ natural frequency of the soil bed can then be calculated from Equation (4-3).

$$
f_n = \frac{(2n - 1) \times \bar{V}_s}{4 \times H}
$$
Figure 4-2. Shear wave velocity profile for TD1

Through the experimental program, 39 pulses were applied to the soil bed, denoted pulse (A) to pulse (MM). Figure 4-3 shows the variation of the average shear wave velocity during the test program, and Figure 4-4 shows the variation in the sand bed’s fundamental natural frequency. Both Figure 4-3 and Figure 4-4 show that there was a slight increase in the shear wave velocity and the fundamental natural frequency of the sand bed on TD4, when the two pile groups were first installed, then returned to its initial average on TD5.

The results in Figure 4-3 also show that changes in the shear wave velocity after each shake were negligible and an average value of 100 m/s could be assumed. This value of shear wave velocity resulted in a calculated natural frequency for the soil bed of approximately 5.47 Hz, which corresponds to a fundamental period of 0.183 s. These results are validated by the value of the natural frequency of the sand bed determined from the frequency analysis of the measured responses as described in later sections.
Figure 4-3. Variation of the average shear wave velocity of the sand bed throughout the testing program

Figure 4-4. Variation of the approximate sand bed’s fundamental natural frequency throughout the testing program
4.2.2 Shear Strain

The shear strain through the soil depth was determined from String Potentiometers (SPs) installed to the side of the shaking table; 15 of them were installed at different depths. These were used to record the time history of the soil block movement.

A simple first order approximation was used to determine the shear strain between each two successive SPs using Equation (4-4), in which \( \gamma_i \) is the shear strain in layer \( i \), \( u \) is the displacement of the sand, \( z \) is the depth of the SP and subscripts \( u \) and \( l \) refer to the upper and lower SPs in the layer, respectively.

\[
\gamma_i(t) = \frac{u_u(t) - u_l(t)}{z_u - z_l}
\]  

(4-4)

Figure 4-5 shows an example of the peak displacement experienced by the soil block on TD1 [pulse (A) and pulse (B)] while Figure 4-6 presents the absolute maximum shear strain through the soil depth. As can be seen from Figure 4-5, although the peak acceleration was amplified by the soil bed to almost double the input value, the peak displacement is nearly constant through the soil depth. This could be attributed to the very low frequency content of pulse waves compared to the natural frequency of the soil block. Motions in this very low frequency ratio range are mostly dominated by accelerations and experience very low amplification in displacements. On the other hand, the absolute maximum shear strain in the soil depth for both pulses did not show a clear trend throughout the soil depth as demonstrated by Figure 4-6. This could be attributed to non-uniformity in preparing the sand bed during the compaction process, especially closer to the laminar shear box borders, where the SPs used in the evaluation of the shear stress were installed. A very close trend was observed for the shear wave velocity as can be seen from Figure 4-2. However, shear strains experienced by top soil layers were consistently larger because of the amplification of ground motion from bottom to top.

A weighted average value of the maximum shear strain through the sand bed is calculated from Equation (4-5), in which \( H \) is the total depth of the soil and \( h_l \) and \( \gamma_i \) are the height and shear strain of each layer, respectively.
\[
\bar{y} = \frac{H}{\sum_{i=1}^{n} h_i / \gamma_i}
\]  

(4-5)

Figure 4-5. Peak displacements along the soil depth on TD1: a) Pulse (A); b) Pulse (B)

Figure 4-6. Maximum absolute shear strain along the soil depth on TD1: a) Pulse (A); b) Pulse (B)
This weighted average of the shear strain was used to explore the effect of different soil and/or pile configurations, as well as the effect of successive shakings. Figure 4-7 shows the variation in the average shear strain in the soil bed throughout the testing program, where a total average value for each testing day is plotted with a red thick line. Some observations could be extracted from Figure 4-7 as follows:

1- The shear strain during TD2 (soil and piles without head masses) did not show a variation or a clear trend other than fluctuating around an average value which was very close to that during TD1 (soil block only). This indicated that piles without pile head masses did not affect the shear strains through the soil depth.

2- At the beginning of TD3 (soil and piles with pile head masses), the shear strain showed a sudden increase (about 20%). This could be attributed to larger responses experienced by pile-soil systems due to the introduction of pile head masses. As all single piles had natural frequencies, closer to the TAK earthquake compared to NOR earthquake, as would be mentioned in later sections, significant responses were experienced by the piles, which resulted in high shear strains in the soil. This was evident from the decrease in the shear strain in the first half of TD3, at which NOR earthquake testing took place, and then the increase near the end of the day when TAK earthquake testing occurred.

3- The shear strain increased at the beginning of TD4 (fixed pile groups) when the two pile groups were first formed, however the ratio was less than that found on TD3. A noticeable trend was observed in which, unlike the case of single piles on TD3, the shear strain increased to maximum value through the first half of the day [pulse (U) to pulse (Y)], then decreased to reach the initial value. This was attributed to the relative closeness of the natural frequency of PG2 and the frequency content of NOR earthquake (which was applied during the first half of the testing day). This led to significant responses, which increased the shear strain experienced by the soil block. During the second half of the day, at which TAK earthquake testing took place, the shear strain started to decrease due to the very low frequency content of
TAK earthquake, which was far away from the natural frequency of both pile groups.

4- The value of the shear strain did not change from TD4 to TD5 (pinned pile groups) as well as the same trend was observed, however the maximum shear strain occurred on TD5 was lower than that for TD4. This indicated that pile head connection had a minimal effect on the shear strain through the sand bed. It is worth noting that the last value for the shear strain on TD5 did show a significant increase and the reason was that the last testing that took place on TD5 was (NOR-150-T0) which was a 150 % of NOR earthquake amplitude. Due to resonance effects of PG2 as mentioned before, the shear strain in the soil increased.

Figure 4-7. Variation of the average shear strain in the sand bed throughout the testing program

4.2.3 Small Strain Damping Ratio

Small strain damping is a key factor of soil behaviour under seismic loading. This is used along with the low strain shear modulus to establish a baseline for the dynamic properties
of the soil. These values are then changed accordingly as a factor of the shear strain during large intensity shakings.

Typically, to explore the damping of a system, it is given an initial displacement or velocity and left to undergo free vibration. Hence, the damping ratio can be determined from the decayed free vibration response of the system. In order to achieve this, pulse waves were applied to the shaking table which is defined as a signal that has a very sharp acceleration peak with a very small time-span. However, due to many concerns related to the shaking table effects on the input signal, the actual pulse wave applied to the system had different and non-uniform shape. This made it more challenging dealing with responses that were not a uniform free vibration, and that had non-uniform and scattered peaks.

Many methods are widely used to determine the damping ratio at small strain. Some of them employ the time history response to calculate the damping ratio, while others employ the frequency response. Two of the most famous methods are discussed herein, which are the logarithmic decrement method and the half-power bandwidth method.

**Logarithmic decrement method**

The logarithmic decrement is defined as the natural logarithm of the ratio of two peaks’ amplitudes in the time domain of a free vibration response divided by the number of cycles as seen in *Equation (4-6)*, where, $X$ is the peak amplitude and $n$ is the number of cycles.

\[
\delta = \frac{1}{n} \ln \frac{X_i}{X_{i+n}} \tag{4-6}
\]

The logarithmic decrement is related to the damping ratio, $\xi$, through *Equation (4-7).*

\[
\xi = \frac{1}{\sqrt{1 + \left(\frac{2\pi}{\delta}\right)^2}} \tag{4-7}
\]

For linear systems, this value should be constant through the whole time domain response regardless of how many cycles are used to determine the logarithmic decrement. However, when the system’s response is nonlinear, the damping ratio is highly dependent on the
value of the shear strain, which might typically change from a high value at the beginning of the response and gradually decreases until the end. This raised the problem of how many cycles should be taken into consideration when calculating the logarithmic decrement, in which many studies were conducted on this point. A schematic of a decayed free vibration response is shown in Figure 4-8.

**Figure 4-8. Schematic of a typical free vibration decay**

**Half-Power bandwidth method**

In this method, the Frequency Response Function (FRF) of the time domain response is calculated by converting it to the frequency domain employing a Fast Fourier Transform algorithm (FFT), and then normalizing by the frequency amplitude of the input motion. The resonant frequency \( f_r \) is then determined where the maximum amplitude occurs \( (y_r) \). The two frequency values that have an amplitude of \( 1/\sqrt{2} \) times the maximum amplitude are identified, \( (f_1) \) and \( (f_2) \), and then used to calculate the damping ratio from Equation (4-8). A schematic of these definitions is shown in Figure 4-9.

\[
\xi = \frac{f_2 - f_1}{2f_r}
\]  

(4-8)
Figure 4-9. Schematic of the half power bandwidth definitions

In this study, the logarithmic decrement method was employed in which soil accelerometer responses due to pulse waves were used to calculate the damping ratio of the sand bed. Due to the resulted non-uniform free vibration response, the logarithmic decrement could not be applied directly. Two steps were first performed:

1- A second order exponential curve was best fitted to the peaks of the response. An example of this curve fitting result as well as the equation representing the curve is shown in Figure 4-10 for the first pulse on TD1.

2- This fitted curve was then used to calculate the peak values starting from the maximum peak and separated by an average period calculated from the whole response.

The other point to determine is the number of cycles that would be considered when calculating the logarithmic decrement. Previous studies had shown that the number of cycles considered for the logarithmic decrement to give realistic values for damping ratio is related to the actual damping expected, and this number decreases as the damping ratio increases (Tweten et al. 2014).
Figure 4-10. Second order exponential curve fitting for pulse response

In order to construct a datum for the calculations, several ground response analyses were executed employing the software DEEPSOIL (Hashash et al. 2020). For each testing day, a model was constructed and the ground response was calculated due to the first pulse applied on that day. A linear frequency domain analysis was adopted in which the sand bed was modeled as one layer with unit weight of 19.5 kN/m$^3$ and an average shear wave velocity of 100 m/s that was determined previously. The input pulse wave was applied to the bottom of the model and the response was calculated at the same depth of the top level of accelerometers.

The damping ratio was back calculated in a way that the response from DEEPSOIL nearly matched those recorded by the top-level accelerometers. Figure 4-11 shows an example of the response from DEEPSOIL compared to the accelerometer readings on TD1, in which a damping ratio of 5 % resulted in a good agreement. The number of cycles used in the logarithmic decrement calculations was then determined accordingly to give the same damping ratio. The same number of cycles was used for the each testing day, which ranged from 6 cycles for the first 3 testing days and 9 cycles for TD4 and TD5.
Figure 4-11. Comparison of the sand bed’s response with DEEPSOIL for TD1

The logarithmic decrement was calculated between each two peaks through the range considered and an average value from all was found to give a reliable damping ratio. The damping ratio profile through the soil depth was calculated from the three accelerometer-arrays and an average profile was constructed. An example of the variation of damping ratio with depth is shown in Figure 4-12 for TD1.

A weighted average value of the damping ratio through the soil’s depth is calculated from Equation (4-9), in which \( H \) is the total depth of the soil and \( h_l \) and \( \xi_l \) are the height and damping ratio at each depth, respectively. This weighted average value of the damping ratio was used to explore the effect of different soil/pile configurations as well as the effect of successive shakings as seen in Figure 4-13. A single average value for each testing day is plotted with a thick red line.

\[
\bar{\xi} = \frac{H}{\sum_{l=1}^{n} \frac{h_l}{\xi_l}} \tag{4-9}
\]
Figure 4-12. Damping ratio profile for TD1

Figure 4-13. Variation of the average damping ratio of the sand bed throughout the testing program
Some observations could be made from **Figure 4-13** as follows:

1- On TD2 (soil and piles without head masses), the damping experienced a little increase compared to TD1 (soil only), which indicted that installation of piles without head masses did have a minimal effect on increasing the damping of the soil. On the other hand, the same fluctuation was observed as in the shear strain, in which the average value was very close to that of TD1.

2- When masses were added to piles on TD3, the damping ratio increased by almost 10 %. This is attributed to the increase in the response of pile-soil systems due to inertial forces. A decreasing trend was pronounced in the first half of TD3 [pulse (L) to pulse (Q)], then started to increase rapidly towards the end of the day. This is associated with resonance effects that was experienced by single piles during TAK earthquake testing, which was applied in the second half of TD3, as all single piles had a natural frequency very close to the frequency content of TAK earthquake.

3- A substantial increase in the damping ratio was observed on TD4 and TD5 at which pile group testing took place. This increase was about 32 %. Due to the huge weights of pile caps, the response of soil increased especially in the vicinity of piles and within pile groups, in which the soil accelerometers were located. The same trend as that observed in the shear strain was noticed. The damping ratio increased to the maximum value in the first half of the testing day, then decreased to nearly the initial value. This is again attributed to the very high response of PG2 during NOR earthquake testing.

4- Changing the pile head connection from fixed (TD4) to pinned (TD5) did not have a significant effect on the values of damping ratio as well as on the variation through successive shakings. However, the maximum value observed on TD5 was a little bit smaller than that on TD4, which is a result of the decrease in the response from fixed connection to pinned one.
Values of damping ratios with corresponding shear strains are compared to the well-known characteristic curves proposed by Seed and Idriss (1970) in Figure 4-14. As can be seen from Figure 4-14, two clusters of damping values were observed, in which the lower one is associated with the first three testing days, i.e., sand bed only or soil with single piles and the top one is associated with TD4 and TD5, i.e., pile groups testing. In addition, Table 4-1 presents a summary of maximum, minimum and average of shear strains experienced by the sand bed and the corresponding average damping ratio during each testing day.

<table>
<thead>
<tr>
<th>Testing Day</th>
<th>Shear Strain (%)</th>
<th>Damping Ratio (%)</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>Max</td>
<td>Min</td>
</tr>
<tr>
<td>TD1</td>
<td>0.0600</td>
<td>0.0574</td>
</tr>
<tr>
<td>TD2</td>
<td>0.0602</td>
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</tr>
<tr>
<td>TD3</td>
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</tr>
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<td>0.0721</td>
<td>0.0538</td>
</tr>
<tr>
<td>TD5</td>
<td>0.0682</td>
<td>0.0513</td>
</tr>
</tbody>
</table>

Figure 4-14. Comparison of damping ratios with Seed and Idriss (1970) damping curves
4.3 Natural Frequency

Response of pile-soil system to dynamic loads is mainly influenced by its natural frequency, in which significantly larger responses would be expected whenever the natural frequency is matching, or close to, either the frequency content of the earthquake and/or the natural frequency of the surrounding soil medium. Pile-soil systems possess different natural frequencies corresponding to the modes of vibration of both the soil mass and piles or pile groups.

Natural frequencies of a pile-soil system can be obtained by subjecting the system to a sweep of harmonic loads with varying frequencies and observing the frequencies associated with peaks of response, which correspond to the system natural frequencies. Alternatively, the natural frequency of a system can be determined from the response time history of the system subjected to a transient dynamic load. The response time history is filtered and then transferred to the frequency domain employing a Fast Fourier Transform (FFT) algorithm. The FFT can be directly used to determine the system’s natural frequencies, where peaks are pronounced at their frequency locations. Another way is to compute the Frequency Response Function (FRF), defined as the ratio between the response frequency amplitude to the input frequency amplitude. Both methods have advantages and disadvantages, and each method may be more appropriate under different loading conditions.

The FFT analysis can be used to evaluate the system’s natural frequencies accurately and clearly if the input signal has uniform and nearly constant amplitude for the range of frequencies under consideration, i.e., an ideal white noise signal. However, typical transient loading would have a wide range of frequencies with varying amplitudes. In such case, the response amplitudes at different frequencies should be compared with the input amplitudes at these frequencies to determine locations of the peaks where maximum amplifications of the input signal occur. However, it can be difficult to establish the peaks and there would be some spurious peaks, especially at higher modes of vibration. The FFT comprises fewer steps and can provide a fast estimate of the fundamental natural frequency; however, it may not be accurate. Therefore, the FFT method is not recommended if higher modes of vibration are to be considered.
The FRF method, on the other hand, yields a smooth curve with distinct peaks at frequencies that correspond to the natural frequencies of the system. It can also provide good results regardless of the input signal, i.e., it does not require a perfect white noise input signal. It is recommended to employ the FRF method to evaluate the natural frequencies of the system, as it is more accurate and especially for higher modes of vibration. Another advantage of the FRF is that it gives an indication of the amplification of the input ground motion.

In order to determine the natural frequency of different components of the system, responses due to white noise signal are investigated. To evaluate the sand bed natural frequencies, responses of soil accelerometers were used. On the other hand, to assess the natural frequencies of single piles and pile groups, responses of pile accelerometers as well as strain gauges were utilized. It should be noted that white noise signals were applied only once at the beginning of each testing day, thus it would provide one estimate for the natural frequency for each day, which can be used to compare different soil and/or piles configurations during the five testing days. However, in order to evaluate the effect of successive shakings, pulse waves were employed.

Although pulse waves were primarily applied to evaluate the shear wave velocity and damping ratios, it can be employed to evaluate the natural frequency provided that it lies within the frequency range of the pulse wave. Typically, the frequency range of a pulse wave is small compared to a white noise signal; however, in this study, all fundamental natural frequencies were in the frequency range of the pulse wave, and thus could be evaluated from the pulse wave responses. Meanwhile, higher vibration modes were evaluated from the white noise responses. Since pulse waves were applied before and after each shake test, it provided a tool to monitor the change in natural frequency due to successive shakings during each testing day.

4.3.1 Sand Bed

Separating the natural frequencies of the pile foundation and soil bed with an acceptable margin ensures the foundation response is not adversely affected by the resonance condition, which could lead to large deformations and straining actions in the piles. Hence,
it is essential to accurately determine different natural frequencies of soil bed corresponding to different modes of vibration. An example of the response of the east array of accelerometers due to white noise signal on TD1 is shown in Figure 4-15 and Figure 4-16 for the top accelerometer and the whole array, respectively.

Figure 4-15. Frequency response of top accelerometer in the east array due to white noise signal on TD1: a) FFT; b) FRF

Figure 4-16. Frequency response of the east array of accelerometers due to white noise signal on TD1: a) FFT; b) FRF
Some observations can be made from Figure 4-15 and Figure 4-16 as follows:

1- The values of natural frequencies evaluated form the FFT and FRF methods differ by 10% approximately.

2- Higher modes of vibration are more pronounced from the measurements of the lower accelerometers. This is because the soil box experienced more rotation at the bottom near its connection to the shaking table, while the top of soil bed experienced less rotation but larger horizontal movement. Thus, the fundamental (horizontal) natural frequency was evaluated from the measurements of the top accelerometer, while the higher modes were evaluated from the measurements of the lower accelerometers.

3- It is difficult to identify the second natural frequency from the FFT spectra, as there are two spurious peaks at 10 and 20 Hz besides the true second natural frequency, which is around 14 Hz. Moreover, the FFT spectrum of the top accelerometer did not even feature a peak at or close to this frequency. On the other hand, the FRF method provided a more identifiable second natural frequency compared to the FFT method, especially in top accelerometer.

Figure 4-17 to Figure 4-20 present the Fourier spectra of the east array in TD2 to TD5, respectively. These figures indicate the effects of the installed piles in TD2, adding the inertial masses in TD3 and adding the model structure in TD4 and TD5. The soil Fourier spectra displayed more peaks due to the interaction with the piles and the influence of their natural frequencies. This is particularly evident on TD4 and TD5 where it was difficult to differentiate between peaks corresponding to different natural frequencies of the surrounding sand bed and those corresponding to different pile group-soil systems. For example, Figure 4-19 shows that the effect of PG2 response on the FFT spectra was even larger than that of the soil itself, in which the maximum peak was observed at around 4 Hz, which corresponds to the natural frequency of PG2. Given the multiple peaks exhibited by the FFT, the FRF results were obtained first and then the value of the nearest peak from FFT was evaluated. The difference between the natural frequencies of the second mode of vibration evaluated from the FFT and FRF results on TD4 and TD5 was about 20%.
Figure 4-17. Frequency response of the east array of accelerometers due to white noise signal on TD2: a) FFT; b) FRF

Figure 4-18. Frequency response of the east array of accelerometers due to white noise signal on TD3: a) FFT; b) FRF
Figure 4-19. Frequency response of the east array of accelerometers due to white noise signal on TD4: a) FFT; b) FRF

Figure 4-20. Frequency response of the east array of accelerometers due to white noise signal on TD5: a) FFT; b) FRF
The measured accelerations along the depth of the soil bed were also used to evaluate the amplification of the input motion through the soil. The results of the first three testing days demonstrated that the soil bed amplified the input motion Fourier amplitudes at the fundamental frequency by almost 1100%. The same amplification was observed in TD1 (soil only testing) and TD2 and TD3 (single piles testing). However, the amplification was only 500% in TD4 and TD5 (pile groups testing). This may be attributed to the increase in the system stiffness associated with the higher stiffness and large mass of the pile groups.

The value of the natural frequency was computed from the three accelerometer-arrays, and an average value was considered. Figure 4-21 shows a comparison between the responses of the top accelerometer of the three arrays on TD1. As can be seen from Figure 4-21, an exact match in both the first and second natural frequencies was observed.

A summary of the first and second natural frequencies of the sand bed throughout the five testing days as well as Fourier amplitudes of the input and response and the amplification ratios are shown in Table 4-2 and Table 4-3, respectively from both FFT and FRF.

Figure 4-21. Frequency response of the top accelerometer in east, center, and west arrays due to white noise signal on TD1: a) FFT; b) FRF
### Table 4-2. Soil bed 1st natural frequency

<table>
<thead>
<tr>
<th>Testing Day</th>
<th>TD1</th>
<th>TD2</th>
<th>TD3</th>
<th>TD4</th>
<th>TD5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average FFT Amplitude (g.s)</td>
<td>Input Response</td>
<td>0.035</td>
<td>0.047</td>
<td>0.023</td>
<td>0.038</td>
</tr>
<tr>
<td>Amplification (%)</td>
<td>1028</td>
<td>1087</td>
<td>1098</td>
<td>444</td>
<td>497</td>
</tr>
<tr>
<td>(f_n1) (Hz)</td>
<td>FFT</td>
<td>5.07</td>
<td>5.09</td>
<td>5.19</td>
<td>6.60</td>
</tr>
<tr>
<td></td>
<td>FRF</td>
<td>5.61</td>
<td>5.67</td>
<td>5.80</td>
<td>6.74</td>
</tr>
</tbody>
</table>

### Table 4-3. Soil bed 2nd natural frequency

<table>
<thead>
<tr>
<th>Testing Day</th>
<th>TD1</th>
<th>TD2</th>
<th>TD3</th>
<th>TD4</th>
<th>TD5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fourier Amplitude (g.s)</td>
<td>Input Response</td>
<td>0.033</td>
<td>0.034</td>
<td>0.021</td>
<td>0.027</td>
</tr>
<tr>
<td>Amplification (%)</td>
<td>358</td>
<td>383</td>
<td>324</td>
<td>284</td>
<td>334</td>
</tr>
<tr>
<td>(f_n2) (Hz)</td>
<td>FFT</td>
<td>15.78</td>
<td>15.69</td>
<td>16.03</td>
<td>17.23</td>
</tr>
<tr>
<td></td>
<td>FRF</td>
<td>14.26</td>
<td>13.59</td>
<td>15.10</td>
<td>14.63</td>
</tr>
</tbody>
</table>

#### 4.3.1.1 Effect of Successive Shaking

In order to evaluate the effect of successive shaking on the natural frequency of the system, pulse waves were used. The feasibility of employing the measured responses of the pulse waves to evaluate the natural frequency is validated first by comparing the FRF results of the east array on TD1 for white noise signal and pulse wave as shown in Figure 4-22. It can be observed from Figure 4-22 that the fundamental natural frequency of the soil established from the pulse wave is slightly larger than that computed from white noise signal. The pulse wave response exhibits a peak close to the second natural frequency, but it may not be accurate because it lies outside the range of frequency of the pulse wave.

The results obtained from FRF of all pulse waves [pulse (A) to pulse (MM)] and white noise signals are plotted in Figure 4-23. The approximate values of the natural frequency obtained from shear wave velocity using Equation (4-3) are plotted as well.
Figure 4-22. FRF of the east array of accelerometers on TD1: a) White noise signal; b) Pulse wave

Figure 4-23. Variation in the soil block’s fundamental natural frequency throughout the testing program using white noise signals, pulse waves and shear wave velocity
The following observations can be made on Figure 4-23:

1- The sand bed natural frequency remained nearly constant throughout the testing program. This means that successive shaking had negligible effect on the sand bed natural frequency, i.e., no degradation in the soil stiffness.

2- The natural frequency increased slightly gradually from TD1 to TD4 for different reasons but decreased in TD5. On TD2, piles were installed, which stiffened the soil and slightly increased its natural frequency. On TD3, masses were added to piles, which reduced the responses and increased the system impedance. On TD4, large masses were added on top of the two pile groups, which further reduced the response and increased the system impedance. On TD5, the natural frequency decreased, which could be attributed to the closeness of the natural frequency of PG1 to that of the sand bed, which resulted in large nonlinear deformations in the sand and formation of gaps around piles during TD4. When white noise signal was applied on TD5, the sand caved in filling the gaps in a loose condition. This decreased the stiffness of the soil, and consequently the natural frequency. The same soil nonlinearity and gap opening occurred during TD3; however, the stiffening effect of the pile groups and the increased mass on TD4 overcame the decrease in the soil stiffness due to the nonlinearity.

3- The natural frequency values calculated from pulse waves responses were higher than the values evaluated from the measured shear wave velocity.

4.3.2 Single Piles with no Head Mass

Natural frequencies of piles were evaluated employing their responses to the white noise signals. On the other hand, pulse waves responses were used to explore the effect of successive shaking on the piles’ natural frequencies. The responses recorded from pile accelerometers and strain gauges were analyzed and compared. Since the focus of this study is on pile groups, only the results of piles (P1-P4) and (P7-P10) are presented herein.
The piles were installed in TD2 with no head mass. This means only kinematic interaction occurred between the piles and soil during the shaking, and the piles response is influenced by the vibration of the soil bed and its natural frequencies.

As an example, Figure 4-24 presents the FRF of P10 response to the white noise signal. It can be seen from Figure 4-24 that P10 pronounced response peaks corresponding to the natural frequencies of the soil vibration modes. Moreover, responses from both accelerometer and strain gauges coincide; however, peaks at higher modes were more pronounced in the accelerometer response. This is attributed to the large rotation that occurred at the location of the accelerometer at the pile head, while the strain gauge was located at the ground surface. Table 4-4 presents a summary of the fundamental frequency values evaluated from the white noise signal from both accelerometers and strain gauge readings.

![Figure 4-24. FRF of P10 due to white noise signal on TD2](image)

4.3.2.1 Effect of Successive Shaking

The responses of single piles to pulse wave signals were used to evaluate the effect of successive shaking on the fundamental frequency. Figure 4-25 displays the variation of
the fundamental natural frequency of P10 evaluated from FRF throughout TD2 [pulse (C) to pulse (K)]. It can be seen from Figure 4-25 that the natural frequency remained almost constant throughout all pulses, and that values obtained from accelerometers and strain gauges were almost identical. It is noted that the pile natural frequency matched that of the sand bed. Similar results were obtained for all piles (P1-P4) and (P7-P10).

<table>
<thead>
<tr>
<th>P1</th>
<th>5.09</th>
<th>5.67</th>
<th>5.26</th>
<th>5.68</th>
</tr>
</thead>
<tbody>
<tr>
<td>P2</td>
<td>5.12</td>
<td>5.68</td>
<td>5.07</td>
<td>5.69</td>
</tr>
<tr>
<td>P3</td>
<td>5.12</td>
<td>5.68</td>
<td>5.11</td>
<td>5.70</td>
</tr>
<tr>
<td>P4</td>
<td>5.12</td>
<td>5.68</td>
<td>5.15</td>
<td>5.69</td>
</tr>
<tr>
<td>P7</td>
<td>5.11</td>
<td>5.68</td>
<td>5.13</td>
<td>5.71</td>
</tr>
<tr>
<td>P8</td>
<td>5.14</td>
<td>5.68</td>
<td>5.11</td>
<td>5.70</td>
</tr>
<tr>
<td>P9</td>
<td>5.14</td>
<td>5.68</td>
<td>5.03</td>
<td>5.65</td>
</tr>
<tr>
<td>P10</td>
<td>5.14</td>
<td>5.68</td>
<td>5.03</td>
<td>5.65</td>
</tr>
</tbody>
</table>

Figure 4-25. Variation in the fundamental natural frequency of P10 during TD2
4.3.3 Single Piles with Head Mass

Different masses were placed on top of piles in TD3. Thus, both inertial and kinematic soil-piles interaction took place and influenced the response of the pile-soil systems. This was manifested in the soil vibration at a frequency close to the natural frequency of piles.

The pile responses to white noise signals recorded by accelerometer and strain gauges were analyzed to evaluate the natural frequencies. For example, Figure 4-26 shows the responses of P4.

![Figure 4-26. Normalized FRF of P4 on TD3](image)

The following observation can be made from Figure 4-26:

1- The pile response exhibited peaks corresponding to the first and second natural frequencies of the sand bed due to the interaction between the soil and the pile. These peaks should not be mistakenly taken as higher modes of pile vibration.

2- The responses from accelerometer and strain gauges coincided; however, the fundamental frequencies were more pronounced in the strain gauge response.
3- The pile responses were dominated by the horizontal vibration mode because the second natural frequency was outside the frequency content of the white noise signal (i.e., 0 - 40 Hz), which would be validated in subsequent sections.

The responses measured by the strain gauges were utilized to investigate the effect of different pile parameters on its natural frequency as they featured clear peaks and were less affected by the soil’s vibration. However, natural frequency values from both accelerometers and strain gauges were close.

4.3.3.1 Effect of Pile-Head Mass

The natural frequency of a system is inversely proportional to square root of its mass, i.e., as the mass attached to the pile head increases, its natural frequency decreases. To evaluate the effect of pile mass on its natural frequency, responses of piles P2 and P3 were compared. Piles P2 and P3 were identical but had different masses at their head (749 kg and 777 kg for P2 and P3, respectively). Figure 4-27 compares the FRF of top strain gauges in P2 and P3 due to white noise signal.

Figure 4-27. Effect of pile-head mass on the natural frequency of single piles
4.3.3.2 Effect of Embedment Depth

The effect of the pile embedment depth on its natural frequency was explored through comparing P1 and P2, as the embedded depth of P1 was 0.3 m greater than P2. It is expected that increasing the embedment depth would increase the stiffness and hence the natural frequency. **Figure 4-28** compares the responses of P1 and P2. Even though P1 had larger pile head mass (768 kg) than P2 (749 kg), its natural frequency was greater than that of P2.

![Figure 4-28. Effect of embedment depth on the natural frequency of single piles](image)

4.3.3.3 Effect of Number of Helices

To evaluate the effect of number of helices, responses of P2 and P4 were compared. Both piles had the same cross-section and pile-head mass, but P4 had two helices and P2 had a single helix. **Figure 4-29** presents the FRF of P2 and P4 from which it can be seen that P4 had lower natural frequency than P2. This is attributed to the additional disturbance in the soil along the pile shaft owing to the passage of two helices for the double-helix pile, and hence, reducing the stiffness of the pile-soil system. It should be noted that the second helix was well below the effective depth for lateral resistance; thus, it did not contribute to the...
lateral pile’s stiffness. However, it may contribute to the rocking stiffness of the pile group as will be discussed later.

**Figure 4-29. Effect of number of helices on the natural frequency of single piles**

4.3.3.4 Effect of Free (Stick-out) Length

The free (stick-out) length of the pile is defined as the distance from the ground surface to the center of gravity of the pile-head mass. To determine the effect of the pile’s free length on its natural frequency, responses are compared in Figure 4-30. Both P7 and P8 that had the same cross-section but different free-length (1.72 m and 1.45 m for P7 and P8, respectively. As shown in Figure 4-30, P7 had a natural frequency of 1.4 Hz while P8 had a natural frequency of 2.49 Hz. It should be noted, however, that the head mass for P7 was 1236 kg and the head mass for P8 was 785 kg. In order to isolate the effect of free length, the natural frequency of P8 was adjusted considering the same mass as P7, which yielded a natural frequency of 1.98 Hz. This large difference in natural frequency (30 %) clearly demonstrates the significant impact of the pile’s free-length on its lateral stiffness. This should be an important consideration in designing of piles subjected to dynamic lateral loads.
Figure 4-30. Effect of free length on the natural frequency of single piles

4.3.3.5 Effect of Flexural Stiffness

To evaluate the effect of pile’s flexural stiffness on its natural frequency, piles of different cross-sections were considered. Unfortunately, the two piles that had different cross-sections (88 mm and 140 mm) had different masses and free-lengths as well. Thus, it was not possible to isolate the effect of the pile’s flexural stiffness; rather a combined effect of all three parameters could be evaluated.

Figure 4-31 presents the responses of piles P2, P8 and P10. P8 and P2 had pile-head masses of 785 kg and 749 kg, and free-lengths of 1.45 m and 0.86 m, respectively. The natural frequency of P8, adjusted to consider the same mass as P2, was 2.55 Hz. This indicated that the increase in P8 stiffness due to its larger cross-section was more than the decrease due to the larger free-length.

On the other hand, head mass of P10 was 1244 kg and its free-length was 1.74 m. Its natural frequency was 1.52 Hz, which was adjusted to 1.96 Hz considering head mass equal to that of P2. In that case, the large free-length of P10 reduced its natural frequency substantially, negating the advantage of a larger pile cross-section on its natural frequency.
4.3.3.6 Summary of Natural Frequencies of Single Piles

Table 4-5 summarizes the natural frequencies for piles P1-P4 and P7-P10. Table 4-5 shows that there was some discrepancy between the values computed from FFT and FRF; however, the difference was much smaller than that was found for the case of the sand bed. On the other hand, results from strain gauges and accelerometers were in good agreement, with the values from strain gauges responses slightly higher than those from accelerometer responses. It is also noted from Table 4-5 that all piles had natural frequencies in the range of 1-3 Hz, which was closer to the predominant frequency of TAK earthquake than the NOR earthquake. Thus, it would be expected to have higher resonance effects and consequently higher pile responses during the TAK earthquake testing. This is manifested in the spectral accelerations for each earthquake as presented in Table 4-6.
### Table 4-5. Fundamental frequency of piles during TD3

<table>
<thead>
<tr>
<th>( f_{n1} ) (Hz)</th>
<th>Accelerometer</th>
<th>Strain Gauge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FFT</td>
<td>FRF</td>
</tr>
<tr>
<td>P1</td>
<td>2.48</td>
<td>2.53</td>
</tr>
<tr>
<td>P2</td>
<td>2.21</td>
<td>2.29</td>
</tr>
<tr>
<td>P3</td>
<td>1.67</td>
<td>1.76</td>
</tr>
<tr>
<td>P4</td>
<td>1.89</td>
<td>1.91</td>
</tr>
<tr>
<td>P7</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>P8</td>
<td>2.32</td>
<td>2.41</td>
</tr>
<tr>
<td>P9</td>
<td>2.58</td>
<td>3.25</td>
</tr>
<tr>
<td>P10</td>
<td>1.37</td>
<td>1.50</td>
</tr>
</tbody>
</table>

### Table 4-6. Fundamental periods and expected spectral accelerations for piles

<table>
<thead>
<tr>
<th>Pile</th>
<th>Fundamental Frequency (Hz)</th>
<th>Fundamental Period (s)</th>
<th>Expected Spectral Acceleration for NOR-100-T0 (g)</th>
<th>Expected Spectral Acceleration for TAK-100-T0 (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>2.69</td>
<td>0.372</td>
<td>0.98</td>
<td>1.52</td>
</tr>
<tr>
<td>P2</td>
<td>2.38</td>
<td>0.420</td>
<td>0.87</td>
<td>1.41</td>
</tr>
<tr>
<td>P3</td>
<td>1.84</td>
<td>0.543</td>
<td>0.57</td>
<td>1.64</td>
</tr>
<tr>
<td>P4</td>
<td>2.01</td>
<td>0.498</td>
<td>0.59</td>
<td>1.66</td>
</tr>
<tr>
<td>P7</td>
<td>1.40</td>
<td>0.714</td>
<td>0.38</td>
<td>1.76</td>
</tr>
<tr>
<td>P8</td>
<td>2.49</td>
<td>0.402</td>
<td>0.88</td>
<td>1.53</td>
</tr>
<tr>
<td>P9</td>
<td>2.99</td>
<td>0.334</td>
<td>1.00</td>
<td>1.13</td>
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<tr>
<td>P10</td>
<td>1.52</td>
<td>0.658</td>
<td>0.49</td>
<td>1.75</td>
</tr>
</tbody>
</table>
4.3.3.7 Effect of Successive Shaking

The piles responses to pulse waves were employed to evaluate the effect of successive shakings on their fundamental natural frequencies. The variations of the fundamental frequency computed from FRF during TD3 [pulse (L) to pulse (T)] for piles P1, P2, P4, P8 and P10 are shown in Figure 4-32 to Figure 4-36, respectively. The values obtained from white noise signals are also presented in the figures. The following observations could be made from these figures:

1- Natural frequency calculated from response to pulse waves was slightly larger than that computed from white noise signal.

2- Results obtained from strain gauges were somewhat larger than those obtained from accelerometers for both white noise signals and pulse waves.

3- Generally, the natural frequency decreased through successive shakings. This is an expected behaviour and can be attributed to multiple reasons. First, degradation in the pile-soil stiffness due to the significant nonlinearity experienced by the soil adjacent to the pile shaft (especially the top portion) associated with gap opening then sand cave-in to fill the gap, which resulted in loosening the soil around the piles and hence a reduction in the soil stiffness. Second, the formation of gaps around the pile shaft at the top portion increased the free-length of the pile and in turn reduced its natural frequency. Finally, as the soil became softer, the volume of soil vibrating with the pile increased, which could further decrease the natural frequency.

4- The natural frequency of some piles remained constant or even increased through successive shakings. This is attributed to closure of gaps around pile shaft and hence increased stiffness of the pile-soil system. The gap opening and closure was confirmed by visual observations during the tests (through video camera recordings).
Figure 4-32. Variation of fundamental natural frequency of P1 during TD3

Figure 4-33. Variation of fundamental natural frequency of P2 during TD3
Figure 4-34. Variation of fundamental natural frequency of P4 during TD3

Figure 4-35. Variation of fundamental natural frequency of P8 during TD3
4.3.3.8 Analytical Model

Several approaches are employed to analyze the Soil-Structure Interaction (SSI) and dynamic response of piles (El Naggar 2004). These approaches include continuum methods, Boundary Element Methods (BEM), dynamic Beam on Nonlinear Winkler Foundation (BNWF), and Finite-Element Methods (FEM).

The continuum approach is generally rigorous, and its solution can be formulated in closed form equations to reduce the computational effort. Novak (1974) and Novak and Aboul-Ella (1978) developed closed-form formulas to determine the complex impedance of single and grouped piles assuming linear visco-elastic behaviour of the pile and the soil. Kaynia (1982) and Kaynia and Kausel (1982) have presented closed form solutions based on Green’s functions incorporated in the Boundary Element Method (BEM) to determine the dynamic stiffness of pile groups.

In the BNWF models, the pile is simulated as a series of discrete linear elastic beam-column elements, and the surrounding soil is modeled by a series of non-linear detachable Winkler springs and dashpots on each side of the pile to represent the soil stiffness and damping (El
Naggar and Novak 1994, 1995, Boulanger et al. 1999, Mostafa and El Naggar 2002, El Naggar et al. 2005). Owing to its simplicity, the BNWF approach is widely employed; however, coupling effects between different layers along the piles are not accounted for. It can be particularly efficient for modeling pile and soil nonlinearity, gapping between pile and soil as well as degradation of soil strength and stiffness (Allotey and El Naggar 2008, Heidari et al. 2014).

The Finite Element Method (FEM) is a versatile approach, which is widely used for SSI problems. It can simulate the behavior of soil and structures with complex geometry subjected to varying loading conditions (Seed and Lysmer 1978, Maheshwari et al. 2004, 2005, Lou et al. 2011, Stewart et al. 2012, El Sharnouby and El Naggar 2018a, 2018b). FEM can be employed to simulate the nonlinear seismic response of piles embedded in layered soil considering different aspects of pile–soil interaction; however, it can be computationally very demanding.

The computer software DYNA6 (El Naggar et al. 2015) is used to evaluate the dynamic characteristics and response of different types of foundations subjected to harmonic, transient or random loads. It evaluates the dynamic characteristics of single piles employing the plane strain solutions proposed by Novak (1974). The pile group stiffness and damping constants are then evaluated employing the superposition approach proposed by El Naggar and Novak (1994, 1995), which utilizes the interaction factors proposed by Kaynia and Kausel (1982). This approach can simulate piles with varying cross-section (to account for the helices), the variation of soil properties along the pile shaft, the pile-head fixity condition as well as the geometrical properties and mass of the structure placed on the pile-head. DYNA6 can also approximate the gap opening and the degradation of soil stiffness within the annular zone around the pile shaft (Elkasabgy and El Naggar 2018).

The program DYNA6 was employed in this study to evaluate the dynamic characteristics of the test single piles and pile groups. Two models were constructed, one for the 88 mm-diameter pile and one for the 140 mm diameter pile. The soil profile was modeled as a homogenous layer with the measured soil properties from the experimental program, i.e., shear wave velocity, $V_s = 100$ m/s, damping ratio, $D = 0.05$, Poisson’s ratio, $\nu_s = 0.3$ and
unit weight, $\gamma_s = 19.5 \text{kN/m}^3$. To model the free-length of the pile (length from the ground surface to the bottom of the pile cap), a soil layer with the same depth was defined and assigned zero stiffness (i.e., shear wave velocity = 0.0 m/s). The piles were modeled as steel pipes with their corresponding lengths and inner and outer diameters. The helices were idealized as plates with the same thickness and modeled in the mid-height of the pitch. The pile material was modeled as linear with Young’s Modulus, $E_p = 200 \text{GPa}$, Poisson’s ratio, $\nu_p = 0.3$, damping ratio, $D_p = 0.02$ and unit weight, $\gamma_p = 77 \text{kN/m}^3$. The piles were considered as end bearing with a fixed pile head. The pile caps were modeled as cylinders with the corresponding dimensions and density as the actual mass used in the tests.

A harmonic loading scheme was employed in which a sweep of harmonic loads with frequencies ranging from 0.05 Hz to 40 Hz with 0.05 Hz step were applied to the pile-soil system and the corresponding response at the center of gravity of the pile mass was calculated. The natural frequency corresponded to the frequency at which the maximum response was calculated. Figure 4-37 and Figure 4-38 compare the natural frequencies evaluated from the experimental results on TD3 [pulse (L) to pulse (T)] with those computed from DYNA6 for piles P1 and P10, respectively.

Figure 4-37 and Figure 4-38 display the results from different analyses that were performed to capture the correct dynamic characteristics of the test piles. The initial analysis was performed considering the pile cap dimensions, pile cap mass and free-length of the pile as well as the helix properties and considering the measured soil properties along the embedded length of the pile. The results from this analysis are denoted (DYNA6 – no gap – no weak zone). As can be noted from the figures, the calculated natural frequencies were far from the experimental values. This discrepancy could be attributed to weakening of the soil surrounding the pile and/or development of a gap along the upper part of the pile. Consequently, another analysis was conducted considering a weak zone around the pile shaft (denoted DYNA6 – no gap – weak zone) to account for the disturbance in the annular soil zone adjacent to the pile due to soil disturbance during pile installation and/or gap opening and closing at the pile-soil interface during shaking. This weak zone is characterized by its thickness as a ratio of pile’s radius and reduced shear modulus as a ratio of the shear modulus of the undisturbed soil. The thickness ratio used corresponded
to the soil column above the helix which was 1.88 and 0.82 for 88 mm piles and 140 mm piles, respectively, and the shear modulus ratio was taken as 0.5. Even though the natural frequency decreased for the case of weak zone but was still much higher than the experimental results.

The observations made from monitoring the soil around the piles indicated the formation of gaps, and in some cases, these gaps closed partially due to the soil caving in, resulting in loose soil zone around the piles (Allotey and El Naggar 2008). Therefore, further analyses were conducted considering gaps between the soil and the upper portion of the pile shaft. These gaps were modeled by increasing the free length of the pile. Due to successive shakings on TD2, although piles had no mass attached to their heads, soil around the pile experienced large deformations, and gaps opened around piles and the free-length differed for each pile. DYNA6 models were established considering gaps of 15 cm and 45 cm for P1, and the results are presented in Figure 4-37. The calculated natural frequencies from these cases were in good agreement with the recorded responses at the beginning of TD3 testing and end of TD3 testing, respectively. This demonstrates that successive shakings during TD3 have promoted the formation of deeper gap around the pile.

P10 experienced more deformations during TD2 and TD3 shakings (especially during TAK earthquake, which caused resonance) and experienced larger deformations due to its larger cross-section, pile-head mass and free-length, hence deeper gaps formed. This led to significant nonlinear deformations in the soil and very large gaps were observed at the end of testing. The results from DYNA6 models of P10 considering a gap of 90 cm (at beginning of TD3) and 180 cm (at end of TD3) bounded the experimental results as shown in Figure 4-38. It is also noted that the difference between the cases with and without a weak zone is negligible because the thickness of soil column above the helix was small compared to P1 as the same helix diameter was used for both.

For all 88 mm diameter piles, the gap depth ranged from (15 cm to 45 cm), while for 140 mm piles, the gap depth ranged from (60 cm to 180 cm) depending on the pile-head mass and the free-length.
Figure 4-37. Comparison of experimental natural frequency of P1 and DYNA6

Figure 4-38. Comparison of experimental natural frequency of P10 and DYNA6
4.3.4 Pile Groups

Two pile groups were formed on TD4; the first pile group (PG1) comprised four piles of 88 mm diameter, and the second group (PG2) comprised four piles of 140 mm diameter. Each pile group supported a cubic steel skid (box) filled with sand (pile cap). Each skid was fitted with two accelerometers located at the level of its center of gravity. The natural frequencies of the pile groups were determined from the responses measured by the two accelerometers as well as the top strain gauge in each of the piles. Figure 4-39 displays the FRF for PG2 in TD4. Several observations can be made from Figure 4-39. The responses from the accelerometers and top strain gauges were in excellent agreement. The average of the two measurements can be used to define the natural frequencies. The FRF curve exhibits a large peak associated with the first natural frequency of PG2 and two smaller peaks corresponding to the natural frequencies of the soil bed, which manifest the effect of SSI. However, this effect is smaller than what was observed for single piles. The pile group behavior was dominated by the horizontal vibration mode because the rocking vibration natural frequency was outside the frequency range of the white noise signal (0 - 40 Hz).

![Figure 4-39. Normalized FRF for PG2 on TD4 (fixed connection)](image-url)
4.3.4.1 Effect of Pile-Head Connection

Two pile head conditions were simulated in the testing program. On TD4, each pile was connected to the steel kid using two bolts so as to simulate a fixed head. On TD5, one bolt was removed from each connection in order to simulate pinned connection. However, a true pin connection was not achieved and the one bolt connection still provided some restraint to pile heads.

The responses of PG2 with fixed head and pinned head connections are shown in Figure 4-40. As noted from Figure 4-40, the natural frequency of PG2 decreased as the pile head condition changed from fixed (two bolts) to pinned (one bolt). However, the difference is small between the two values because the one bolt still provided restraint to the pile head rotation. This would be further verified by comparing the natural frequency with the results from DYNA6.

Figure 4-40. Effect of pile-head connection on the natural frequency for PG2
4.3.4.2 Combined Effect of Stiffness, Mass and Free-Length

Since both pile groups had different pile cap masses (6350 kg for PG1 and 9979 kg for PG2), different pile diameters (i.e., different flexural stiffness) and different free length above the ground surface (1.17 m for PG1 and 1.73 m for PG2), the natural frequencies of the two groups cannot be compared directly. However, a combined effect of stiffness, pile cap mass as well as free length can be compared. Figure 4-41 compares the FRF of PG1 and PG2 on TD4 (fixed pile head). Figure 4-41 shows that, although PG1 comprised small diameter piles, its natural frequency was nearly double that of PG2. The natural frequency of PG2 adjusted to account for the different mass effect would be 4.91 Hz, which is lower than that of PG1. Noting that the moment of inertia of PG2 piles was 7.5 times that of PG1 piles, but the free length of PG2 was larger than that of PG1, which resulted in a natural frequency that was lower than that of PG1. The same results were observed in single piles response, which illustrates the important effect of the free length (and the development of a gap) on natural frequency of pile-soil system.

Figure 4-41. Responses of PG1 and PG2 on TD4 (fixed connection)
4.3.4.3 Summary of Natural Frequencies of Pile Groups

Table 4-7 presents the fundamental frequencies computed from accelerometers as well as strain gauges using both FFT and FRF for both pile head conditions. It is noted from Table 4-7 that the differences between the FFT and FRF results were generally less than 2%, except for PG1 in TD4 (fixed connection) where the difference was about 5%. Furthermore, the results obtained from the accelerometer and strain gauges responses were in good agreement. Table 4-7 also shows that the difference of fundamental frequency between fixed and pinned pile heads cases was about 5%, which clearly indicates that even one bolt provided sufficient restraint against rotation. This is an important observation as in many cases bolted connection of pile heads to steel skids supporting vibrating equipment is considered as pinned, which results in gross underestimation of the stiffness of the piled foundation and the natural frequency of the system. This can also have an important effect on the response of the system to seismic loading depending on the frequency content of the earthquake signal.

Moreover, Table 4-8 provides a summary of the spectral accelerations for both pile groups. It is observed from Table 4-8 that the spectral acceleration for PG2 during NOR earthquake was very large (1.5 – 1.75 g) as its natural frequency was close to the predominant frequency range of NOR earthquake; however, the natural frequency of PG1 lied outside that range and hence the spectral acceleration was lower. On the other hand, both pile groups had low spectral accelerations during TAK earthquake as their natural frequencies were far from the predominant frequency of TAK earthquake.

4.3.4.4 Effect of Successive Shaking

The responses of PG1 and PG2 to pulse wave signals are investigated to evaluate the effect of successive shaking on their natural frequencies. Figure 4-42 and Figure 4-43 show the variation of the fundamental frequency of PG1 and PG2 through TD4 [pulse (U) to pulse (CC)] and TD5 [pulse (DD) to pulse (MM)] calculated from measurements of one of the pile cap accelerometers as well as the top strain gauge in one of the piles. The values obtained from white noise signals are plotted as well.
### Table 4-7. Fundamental frequencies of fixed and pinned pile groups

<table>
<thead>
<tr>
<th>Pile Group</th>
<th>( f_{n1} ) (Hz)</th>
<th>Fixed</th>
<th>Pinned</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>FFT</td>
<td>FRF</td>
</tr>
<tr>
<td>PG1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Accelerometers</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>ACP3E</td>
<td>6.09</td>
<td>6.41</td>
</tr>
<tr>
<td></td>
<td>ACP5E</td>
<td>6.09</td>
<td>6.36</td>
</tr>
<tr>
<td></td>
<td>P1</td>
<td>6.09</td>
<td>6.38</td>
</tr>
<tr>
<td></td>
<td>P2</td>
<td>6.07</td>
<td>6.34</td>
</tr>
<tr>
<td></td>
<td>P3</td>
<td>6.05</td>
<td>6.44</td>
</tr>
<tr>
<td></td>
<td>P4</td>
<td>6.11</td>
<td>6.34</td>
</tr>
<tr>
<td></td>
<td>Strain Gauges</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P1</td>
<td>6.09</td>
<td>6.38</td>
</tr>
<tr>
<td></td>
<td>P2</td>
<td>6.07</td>
<td>6.34</td>
</tr>
<tr>
<td></td>
<td>P3</td>
<td>6.05</td>
<td>6.44</td>
</tr>
<tr>
<td></td>
<td>P4</td>
<td>6.11</td>
<td>6.34</td>
</tr>
<tr>
<td>PG2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Accelerometers</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>ACP8E</td>
<td>3.97</td>
<td>3.92</td>
</tr>
<tr>
<td></td>
<td>ACP9E</td>
<td>3.88</td>
<td>3.86</td>
</tr>
<tr>
<td></td>
<td>P7</td>
<td>3.94</td>
<td>3.89</td>
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<tr>
<td></td>
<td>P8</td>
<td>3.95</td>
<td>3.9</td>
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<td></td>
<td>P9</td>
<td>3.89</td>
<td>3.86</td>
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<tr>
<td></td>
<td>P10</td>
<td>3.9</td>
<td>3.87</td>
</tr>
<tr>
<td></td>
<td>Strain Gauges</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 4-8. Spectral accelerations for PG1 and PG2

<table>
<thead>
<tr>
<th>Pile Group</th>
<th>Pile Head Condition</th>
<th>Fundamental Frequency (Hz)</th>
<th>Fundamental Period (s)</th>
<th>Spectral Acceleration for NOR-100-T0 (g)</th>
<th>Spectral Acceleration for TAK-100-T0 (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PG1</td>
<td>Fixed</td>
<td>6.38</td>
<td>0.157</td>
<td>0.89</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>Pinned</td>
<td>5.90</td>
<td>0.169</td>
<td>1.00</td>
<td>1.05</td>
</tr>
<tr>
<td>PG2</td>
<td>Fixed</td>
<td>3.88</td>
<td>0.258</td>
<td>1.75</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td>Pinned</td>
<td>3.59</td>
<td>0.279</td>
<td>1.49</td>
<td>1.07</td>
</tr>
</tbody>
</table>
Figure 4-42. Variation in the fundamental natural frequency of PG1 through TD4 and TD5

Figure 4-43. Variation in the fundamental natural frequency of PG2 through TD4 and TD5
Several observations can be made from Figure 4-42 and Figure 4-43. The value of natural frequency obtained from the response to pulse waves was generally higher than that computed from the response to white noise signals. It is also noted that the natural frequency decreased after each successive shaking, especially during TD4 because the fixed pile group experienced larger responses, which led to stronger soil nonlinearity and gapping. Hence, the stiffness and natural frequency of the system decreased. This reduction was even more pronounced for PG2 than for PG1 for the fixed pile groups. This was because of the closeness of the natural frequency of PG2 to the predominant frequency range of NOR earthquake signal, while the natural frequency of PG1 was outside that range. This means that PG2 experienced significant resonance, which resulted in higher deflections. Figure 4-42 also shows that the natural frequency decreased the most in the first half of TD4 (NOR earthquake testing) then remained nearly constant afterwards. On the other hand, neither of the groups experienced resonance during TAK earthquake shakings because its predominant frequency range was lower than their natural frequencies. Thus, the natural frequency did not decrease during TAK shakings.

4.3.4.5 Comparison with DYNA6

The dynamic characteristics of pile groups PG1 and PG2 were evaluated employing DYNA6 software. DYNA6 accounts for pile-soil-pile interaction within a pile group using frequency dependent dynamic interaction factors employing the superposition approach (El Naggar and Novak 1995). In addition, it can account for the actual dimensions of the pile cap and free pile length. Two models were established considering the geometrical properties of PG1 and PG2 and pile cap information. Pile caps were modeled as blocks with their actual dimensions of 2.13 m x 2.13 m x 1.73 m (L x W x H) and the applied masses of 6350 kg and 9979 kg for PG1 and PG2, respectively. The piles were arranged in square configuration with a center-to-center spacing of 1.07 m in both directions.

Different conditions were considered in the DYNA6 models to simulate the observed nonlinear response of the pile groups. This included considering a weak zone around the piles and/or gap opening along the top part of the piles. The calculated natural frequencies from DYNA6 models are compared with experimental values in Figure 4-44 and Figure 4-45 for PG1 and PG2, respectively.
The calculated natural frequency of the fixed pile groups obtained from DYNA6 models had the same trends as those observed from the analysis of single piles. The model considering linear behaviour (no weak zone or gap) resulted in high natural frequencies. The model that simulated nonlinearity by defining a weak zone around pile shafts resulted in slightly smaller natural frequency, especially for PG2. The model that simulated nonlinearity by introducing a gap of 15 cm to 30 cm provided natural frequency in good agreement with the experimental results for PG1. For PG2, the depth of the gap was 45 cm at the beginning of TD4 and 75 cm at the end of the day to achieve a good match with the observed experimental results. The range of gap considered in the analysis was supported by test observations. The calculated natural frequency for pinned pile groups using DYNA6 models were lower than the experimental values even without considering a gap or weak zone around the pile shafts. Introducing a weak zone or a gap similar to the fixed head piles case, the natural frequency values were much smaller than the experimental results. This confirms that the one-bolt pile head connection cannot be considered as a pinned connection, and still provided some significant rotational constraint, which resulted in higher lateral stiffness and correspondingly natural frequency.

Figure 4-44. Comparison of natural frequency of PG1 and DYNA6 results
In addition to capturing the correct dynamic properties of the pile groups, DYNA6 was employed to evaluate the responses of PG1 and PG2 due to NOR earthquake which are compared in Figure 4-46 with the experimental results. The same considerations used in evaluating the natural frequency to simulate the nonlinearity in the soil were defined, i.e., a weak zone and a gap. However, an excellent match between DYNA6 results and experimental results was achieved at a gap depth of 75 cm for PG1 and 90 cm for PG2, which were higher than those values introduced to capture the correct natural frequency. Since, natural frequencies obtained from the experimental results were due to pulse waves with low amplitude, higher shaking amplitudes during strong earthquakes would result in larger gap depths.

Figure 4-45. Comparison of natural frequency of PG2 and DYNA6 results
Figure 4-46. Experimental and calculated responses for pile groups due to NOR earthquake using DYNA6: a) PG1; b) PG2
4.4 Conclusions

The following conclusions can be drawn from the shake table testing results as well as the analytical solutions for the single and grouped helical piles.

1- The shear wave velocity of the sand bed was not affected by successive shakings. An average value of 100 m/s was assumed and used in further analyses.

2- Shear strain in the sand increased at the beginning of TD3 and TD4 due to the introduction of pile head masses and pile caps, which resulted in larger responses. The shear strain increased accordingly when resonance was experienced, i.e., during the TAK earthquake testing on TD3, and the NOR earthquake testing on TD4 and TD5. No difference was noticed between fixed and pinned pile groups.

3- Damping ratio of the sand slightly increased from TD1 to TD2 after installing the piles, even with no pile head masses. Moreover, a 10 % more increase was pronounced on TD3 after introducing pile head masses. Furthermore, a substantial increase of about 30 % was observed on TD4 and TD5 due to the huge weight of steel skids added to pile groups. In addition, the damping ratio followed the same behaviour as the shear strain, in which it increased when resonance occurred due to significant responses.

4- The stiffness and natural frequency of the soil bed increased due to the pile installation and corresponding SSI. This was more pronounced in the case of the pile groups. However, due to large response during TD4 testing of the pile groups, soil deformations increased causing degradation of soil stiffness and gap formation along the upper portion of the piles, which reduced the natural frequency of the sand bed the following day (TD5).

5- The number of helices increased soil disturbance during pile installation, which resulted in reduced pile stiffness and natural frequency. On the other hand, as expected, increasing pile embedment depth and/or flexural stiffness increased its natural frequency.
6- Natural frequency of single piles and pile groups decreased due to successive shakings. This is attributed to the degradation in the pile-soil stiffness and opening of deeper gaps. However, in some cases the gap depth decreased due to sand caving-in after some additional shaking, which resulted in increases of stiffness and natural frequency.

7- The DYNA6 software predicted the single and grouped piles behaviour by accounting for degradation of soil stiffness and gap opening. The gap depth ranged from (15 cm to 60 cm) for 88 mm single piles, (60 cm to 180 cm) for 140 mm single piles, (15 cm to 30 cm) for PG1 and (45 cm to 75 cm) for PG2. It is evident that the gap depth increased as the pile diameter (and flexural stiffness) increased.

8- The seismic responses of single and grouped helical piles are greatly affected by the resonance condition (i.e., closeness of the natural frequency to the earthquake predominant frequency). This leads to increased spectral accelerations causing larger deformations and decrease in the stiffness of the pile-soil system.

9- The observed behaviour of pile groups with a single-bolt connection indicated that the assumption of pin connection might not hold true in most cases, which can result in serious underestimation of the pile group stiffness and hence can result in erroneous prediction of its response to seismic loading.
5.1 Introduction

This chapter presents and discusses the measured responses of helical piles and helical pile groups to the strong ground motions applied during the shake-table testing program. The effects of the main earthquake characteristics, i.e., intensity and frequency content, on the seismic performance of the single piles and pile groups are evaluated from the measured responses. In addition, the performance characteristics of helical pile groups are discussed in terms of interaction between piles within a group and the contribution of vertical and lateral stiffness of individual piles to the rocking stiffness and the overall capacity of the pile group. Furthermore, the effect of pile head connection with the pile cap (fixed or pinned) on the pile group response is evaluated and the responses of fixed pile groups (2-bolts connection) versus pinned pile groups (1-bolt connection) are compared. Finally, the behaviour of a single pile and a pile within a group is evaluated in terms of their normalized responses.

5.2 Methodology

Laterally loaded piles are typically assumed to behave as an Euler-Bernoulli’s beam, in which the response is governed by Equation (5-1), where, \( M \) is the bending moment in the pile and \( p \) is the acting soil resistance per unit length along the pile shaft.

\[
\frac{d^2M}{dz^2} - p = 0 \tag{5-1}
\]

Assuming that the pile exhibits linear bending behaviour, the bending moment is proportional to pile curvature and is given by Equation (5-2), where, \( E_p I_p \) is the pile’s flexural stiffness and \( y \) is the pile’s lateral deflection.
Knowing the bending moment profile, the deflected shape of the pile as well as the corresponding soil resistance are determined from double integration and second derivative of bending moment profile, respectively. In addition, rotations and shear forces could be determined from integrating and differentiating the bending moment profile, respectively.

In order to capture straining actions on piles during shakings, strain gauges were attached along the pile shafts to measure the strain time history. For each pile, six (for 88 mm piles) or seven (for 140 mm piles and double helix pile) pairs of strain gauges were installed at different levels. Bending moments and normal forces could be determined at the strain gauge location from the measured strain values through Equation (5-3) and Equation (5-4), respectively, in which \( E_p A_p \) is the pile’s axial stiffness, \( D_p \) is the pile’s outer diameter and \( \varepsilon_1 \) and \( \varepsilon_2 \) are strain gauge readings at opposite sides of the pile’s cross-section at a specific level. It should be noted that Equation (5-3) and Equation (5-4) are only applicable for constant \( E_p \), and before yielding is reached. This was verified from test observations that no yielding occurred.

\[
M = E_p I_p \frac{d^2y}{dz^2}
\]  
\( (5-2) \)

\[
M = \frac{E_p I_p (\varepsilon_1 - \varepsilon_2)}{D_p}
\]  
\( (5-3) \)

\[
N = \frac{E_p A_p (\varepsilon_1 + \varepsilon_2)}{2}
\]  
\( (5-4) \)

Since strain measurements are obtained at discrete locations, a numerical curve fitting procedure was employed to determine the bending moment profile along the pile shaft. ElSawy et al. (2019) assessed the applicability and accuracy of different numerical curve fitting approaches for bending behavior of helical piles, and reported that the accuracy is strongly dependent on the number of data points and their locations. In this study, two curve-fitting procedures were adopted: one was used for the integration of the bending moment, and the other was used for the differentiation process.
Integration of bending moment

A cubic piecewise polynomial spline was employed to fit the bending moment data points along the pile. In order to enforce the boundary condition of zero curvature and zero bending moment at the pile tip, three points were added at distances of 0, 0.025 and 0.05 m from pile tip and assigned a zero bending moment value. The integration and double integration of the fitted spline were evaluated to calculate the rotation and deflection along the pile, respectively. This was performed using the MATLAB function ‘csape’ with its default end condition ‘Lagrange’ in which the end slope of the fitted spline was set to match the slope of cubic polynomial that fits the last four data points at each end.

Differentiation of bending moment

A similar cubic piecewise polynomial spline was used in the differentiation procedure of the bending moment; however, some additional boundary conditions were defined. When evaluating the shear force, an additional boundary condition was added to enforce the first derivative of the function to be zero at the pile tip and a certain shear force value at the ground surface, which was back calculated from the top strain gauge (located 0.025 m and 0.102 m above ground surface for 88 mm piles and 140 mm piles, respectively). Considering that the change in bending moment is linear in the upper segment of the pile above ground up to the center of mass of the pile cap, only one known value of bending moment is required to calculate the shear force as it would be constant in that segment. The MATLAB function ‘csape’ with a ‘complete’ end condition was used in this procedure, and the first derivative of the fitted spline was matched to user-defined values at each end.

In order to evaluate the soil resistance, a boundary condition was applied to enforce the second derivative of the function to be zero at both pile tip and ground surface. The ‘csape’ function in MATLAB was employed with a ‘second’ end condition, in which the second derivative of the fitted spline is matched to user-defined values at each end.

In order to assess the suitability of the curve fitting techniques, the software LPILE (Ensoft Inc 2011) was used to analyze each pile response to an arbitrary lateral load. The bending moment values were obtained at the same locations of strain gauges attached to that pile,
and different curve fitting procedures were applied to determine the pile deflection and corresponding soil resistance. The error between the calculated response from LPILE analysis and that obtained from curve fitting was determined for each curve fitting method. Both procedures used in this study yielded minor errors (< 1 %) in all piles with different strain gauges configurations.

The p-y curves were constructed at different depths considering the pile deflection and the corresponding soil resistance. Unlike static p-y curves, the $y$ in the dynamic p-y curves are given by the relative displacement between the pile and the soil to account for soil displacement. Therefore, the free field soil displacement was subtracted from the pile deflection at each time step according to Equation (5-5).

\[ y = y_{\text{pile}} - y_{\text{soil "free field"}} \]  

(5-5)

The program DEEPSOIL was used to calculate the free field soil displacement, in which the soil was simulated with shear wave velocity, $V_s = 100$ m/s, damping ratio, $D = 5\%$ and unit weight, $\gamma_s = 19.5$ kN/m$^3$ (i.e. same values back-figured from test data). The earthquake was applied to the bottom of the model and the corresponding displacement was calculated at different depths.

5.3 Results and Discussion

5.3.1 Effect of Earthquake Characteristics

The measured responses of two single piles with different physical properties, piles P2 and P7, are compared herein. Similarly, responses of the same piles within the two pile groups, P2 in PG1 and P7 in PG2, are compared to evaluate the behaviour of single piles and pile groups with respect to different earthquake characteristics. Table 5-1 presents the physical properties of P2, P7, PG1 and PG2, in which the free length is defined as the distance from the center of mass of the pile cap to the ground surface.

The same performance trends were observed for both fixed pile groups and pinned pile groups subjected to varying ground motions; therefore, only the fixed pile group case is presented herein. It should be noted that the positive and negative notations used herein
indicate the direction of shaking whether it is in the +ve x-direction or the –ve x-direction, and not the sign of the corresponding response component.

### Table 5-1. Properties of piles P2 and P7 and pile groups PG1 and PG2

<table>
<thead>
<tr>
<th></th>
<th>Pile Diameter (mm)</th>
<th>Pile Cap Mass (kg)</th>
<th>Free Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P2</td>
<td>88</td>
<td>749</td>
<td>0.86</td>
</tr>
<tr>
<td>P7</td>
<td>140</td>
<td>1236</td>
<td>1.71</td>
</tr>
<tr>
<td>PG1</td>
<td>88</td>
<td>6350</td>
<td>1.17</td>
</tr>
<tr>
<td>PG2</td>
<td>140</td>
<td>9979</td>
<td>1.73</td>
</tr>
</tbody>
</table>

### 5.3.1.1 Earthquake Frequency Content

The earthquake frequency content can have a significant effect on the seismic response of structures in case resonance condition occurs. In order to facilitate the discussion, Table 5-2 presents the natural frequencies of P2, P7, PG1 and PG2 as well as the frequency content of both NOR and TAK earthquakes. It can be seen from Table 5-2 that natural frequencies of both single piles, P2 and P7, were within the range of frequency content of TAK earthquake (i.e. resonance) and outside the range of frequency content of NOR earthquake (i.e. no resonance). It is also observed that PG1 was not in resonance for both earthquakes, however, the natural frequency was closer to the frequency content of NOR earthquake, hence, higher responses would be expected. Moreover, PG2 was in resonance with NOR earthquake, but not in resonance with TAK earthquake.

### Table 5-2. Natural frequencies of single piles and pile groups presented in the results and frequency content of different earthquakes

<table>
<thead>
<tr>
<th>Natural Frequency or Frequency Content (Hz)</th>
<th>P2</th>
<th>P7</th>
<th>PG1</th>
<th>PG2</th>
<th>NOR</th>
<th>TAK</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.38</td>
<td>1.40</td>
<td>6.35</td>
<td>3.90</td>
<td>2.5 – 5.0</td>
<td>0.5 – 2.5</td>
<td></td>
</tr>
</tbody>
</table>
In order to explore the effects of frequency content and corresponding resonance conditions on piles behaviour, the measured pile responses from 100 % NOR earthquake (NOR-100) and 75 % TAK earthquake (TAK-75) were compared. Both earthquakes had nearly the same Peak Ground Acceleration (PGA), (0.47 g – 0.48 g), but different frequency contents as seen from Table 5-2.

The bending moment profiles for single piles and piles in a group during NOR-100 and TAK-75 earthquakes are compared in Figure 5-1 and Figure 5-2, respectively. As single piles experienced resonance during TAK earthquake, a significant increase in the response was observed, which is illustrated in Figure 5-1. Although both earthquakes had the same PGA, the maximum bending moment was more than twice during the TAK earthquake. Moreover, PG1 experienced very low bending moment in both earthquakes as no resonance was experienced. However, a slightly larger bending moment was observed during NOR earthquake due to the closeness of the natural frequency of PG1 to the frequency content of NOR earthquake compared to TAK earthquake. In addition, PG2 experienced resonance during NOR earthquake, and hence larger bending moment. This emphasizes that the earthquake frequency content and potential for resonance must be considered in seismic design.

The same observations were made for rotations, deflections, shear forces and soil resistances in which single piles experienced significant responses during TAK earthquake loading compared to NOR earthquake, PG2 responses were higher during NOR earthquakes and PG1 exhibited smaller responses during both earthquakes. Furthermore, the derived p-y curves are derived for P2 and P7 subjected to NOR-100 and TAK-75 earthquakes. Large hysteretic loops were obtained for TAK-75 earthquake with reduced slopes compared to NOR-100 earthquake, which indicated higher soil nonlinearity due to resonance condition. On the other hand, PG2 exhibited larger response loops for both earthquakes compared to PG1.
Figure 5-1. Effect of earthquake frequency content on maximum bending moment in single piles: a) P2; b) P7

Figure 5-2. Effect of earthquake frequency content on maximum bending moment in pile groups: a) PG1; b) PG2
5.3.1.2 Earthquake Intensity

The earthquake intensity is represented in terms of its PGA. Earthquakes with higher PGA would cause larger responses of piles. To evaluate the effect of earthquake intensity, responses of single piles and piles within a group due to TAK earthquake and NOR earthquake with different PGA values were compared. The planned earthquake intensities were 50 %, 75 % and 100 % of the recorded earthquake signals; however, intensities of signals applied in the testing program were slightly different. Nonetheless, they will be referred to as 50, 75 and 100 in the following discussion.

Pile head deflection and rotation

Figure 5-3 presents the variation of the maximum pile head deflection with the earthquake intensity for single piles and piles in a group, while Figure 5-4 presents the maximum pile head rotation. It can be clearly seen from Figure 5-3a and Figure 5-4a that the deflection and rotation increased as the earthquake intensity increased. In addition, the deflection of single piles was much larger than that of piles in a group. This is attributed to the increased rocking stiffness of the pile group owing to the additional contribution from the axial stiffness of individual piles. This was more evident in the rotation in which the difference was significant between single piles and pile groups due to the restraint provided by the pile cap. Responses of all single piles was higher during TAK earthquake due to resonance condition. Moreover, both PG1 and PG2 experienced higher deflections and rotations during NOR earthquake. Furthermore, Figure 5-3a shows that the maximum single pile deflection exceeded 25 mm during TAK earthquake testing, which caused gapping at the pile top and reduced the lateral and rocking stiffness of the single piles. Observations of a large gap opening were made during the test.

On the other hand, the correlation between the maximum deflection and the PGA for single piles was almost exponential, in which the rate of increase of the deflection was much higher than the increase in the PGA as can be seen from Figure 5-3b. Although deflections significantly increased when resonance occurred, the rate of increase of the deflection with increasing the PGA was almost the same whether resonance was experienced or not, i.e.,
for both piles, P2 and P7, due to both NOR and TAK earthquakes. The same behaviour was observed for the rotations of single piles as shown in Figure 5-4b; however, the rate of increase was lower than that for deflections.

![Figure 5-3. Variation of maximum pile head deflection with PGA for single piles and fixed-head pile group: a) Actual values; b) Normalized correlation](image)

On the contrary, the rate of increase of the deflection for pile groups was lower than or almost equal to the increase in the PGA owing to the contribution of the axial stiffness of piles within a group. Two observations were also made: first, although larger deflections were experienced when resonance occurred, the rate of increase of the deflection was much lower than the case with no resonance. Second, the rate of increase of the deflection was
consistently higher for PG1 compared to PG2. This could be directly related to the rocking resistance of the pile group provided by the axial stiffness of individual piles. Higher normal forces would be expected to develop in piles during resonance, as well as for large-diameter piles (PG2), and hence, higher resistance of the pile group. It is interesting to note that for small-diameter piles at no resonance, i.e., very small developed normal forces; the behaviour was close to that of single piles with high rate of increase in the deflection. This was the case for PG1 during TAK earthquake. The same observations were made for the rotation of pile heads in a group; however, the rate of increase was consistently lower than the increase in the PGA.

Figure 5-4. Variation of maximum pile head rotation with PGA for single piles and fixed-head pile groups: a) Actual values; b) Normalized correlation
In addition, Figure 5-5 and Figure 5-6 present an example of maximum deflection profiles for single piles and piles in a group, respectively, due to different earthquake intensities. It can be observed from Figure 5-5 and Figure 5-6 that the depth to which the deflection extended increased as the PGA increased for both single piles and pile groups. It is also observed that the higher the stiffness of piles (P7 and PG2), the higher depth that experienced deflection. Moreover, it can be noticed from Figure 5-5 and Figure 5-6 that the large deflections of P2 and P7 were limited to the 1.5 m and 2.1 m, respectively. This represented approximately 15 to 17 pile diameter for both a single pile and a pile within a group, which indicated a long (flexible) pile behaviour. Furthermore, it is noted that the shaft section around the helices did not experience any noticeable deflection as a result of the fixation provided by the helices.

The same observations were made for the rotations of single piles and pile groups as can be seen from Figure 5-7 and Figure 5-8, respectively. In addition, the location of the maximum rotation was consistently at top of pile for both single piles and pile groups.

In addition, Figure 5-9 presents the deflection time history at pile head due to different earthquake intensities for single pile, P7, and P2 in fixed PG1. As can be seen from Figure 5-9, the deflection amplitude consistently increased as the PGA increased during the whole time duration of the applied earthquake. However, the maximum difference was observed at peaks of the response.
Figure 5-5. Effect of earthquake intensity on maximum deflection profiles for single piles: a) P2 in NOR earthquake; b) P2 in TAK earthquake; c) P7 in NOR earthquake; d) P7 in TAK earthquake
Figure 5-6. Effect of earthquake intensity on maximum deflection profiles for pile groups: a) PG1 in NOR earthquake; b) PG1 in TAK earthquake; c) PG2 in NOR earthquake; d) PG2 in TAK earthquake
Figure 5-7. Effect of earthquake intensity on maximum rotation profiles for single piles: a) P2 in NOR earthquake; b) P2 in TAK earthquake; c) P7 in NOR earthquake; d) P7 in TAK earthquake
Figure 5-8. Effect of earthquake intensity on maximum rotation profiles for pile groups: a) PG1 in NOR earthquake; b) PG1 in TAK earthquake; c) PG2 in NOR earthquake; d) PG2 in TAK earthquake
Figure 5-9. Deflection time history at pile head due to different earthquake intensities: a) P7 (single pile) due to TAK earthquake; b) P2 (fixed-head pile group) due to NOR earthquake
**Pile bending moment and shear force**

Figure 5-10 presents the change in the maximum bending moment with the PGA, while Figure 5-11 shows the change in the maximum shear force. Generally, the maximum bending moment and the maximum shear force increased as the PGA increased as can be seen from Figure 5-10a and Figure 5-11a. Similar to the observations made for the deflection, both single piles, P2 and P7, experienced larger shear forces and bending moments during TAK earthquake, while both pile groups, PG1 and PG2, pronounced higher responses during NOR earthquake. Bending moments and shear forces experienced by single piles were significantly larger than the same piles within a group.

![Figure 5-10. Variation of maximum bending moment with PGA for single piles and fixed-head pile groups: a) Actual values; b) Normalized correlation](image)
In addition, Figure 5-10b and Figure 5-11b show that the rate of increase in the bending moment and shear force were consistently higher than the increase in the PGA for single piles, while they were much lower for pile groups. This is due to the contribution of rocking stiffness to the total stiffness of pile groups, which resulted in two mechanisms of resistance to seismic loading: bending moments due to lateral deflections and normal forces (compression and uplift) due to rocking behavior of piles within a group.

![Figure 5-11](image)

**Figure 5-11.** Variation of maximum shear force with PGA for single piles and fixed-head pile groups: a) Actual values; b) Normalized correlation
Figure 5-12 and Figure 5-13 present the maximum bending moment profiles for single piles and piles in a group, respectively, due to different earthquake intensities. Both single piles and piles in a group display the same behaviour with large bending moments observed within the top 1.5 m (17d) for P2 and 2.1 m (15d) for P7, indicating long (flexible) pile behaviour. Meanwhile, the shaft section around the helix experienced minor bending moments. It was also observed that the depth to which the bending moment profile extended increased as the PGA increased as well as for larger-diameter piles for both single piles and pile groups, the same observation made for the deflections and rotations.

Regarding the shear force, Figure 5-14 and Figure 5-15 present the maximum shear force profiles for single piles and piles in a group, respectively, due to different earthquake intensities. Results from Figure 5-14 and Figure 5-15 indicated that the maximum shear force occurred consistently at the pile head for pile groups and at the point of maximum bending moment for single piles. It is interesting to note that the shear force profile extended to almost the pile tip, especially for larger diameter piles (P7 and PG2). This indicated that the lead section and attached helices provided additional rigidity and substantial support in this case.

The results also demonstrated that the depth of maximum bending moment increased for single piles as well as for PG2 as the PGA increased, which could be seen from Figure 5-16a, while the maximum bending moment depth did not change for PG1. Maximum bending moment occurred at higher depths when resonance occurred due to larger pile deflections, which was the case for single piles in TAK earthquake and pile groups in NOR earthquake. Furthermore, the depth was higher for larger-diameter piles, P7 and PG2 compared to P2 and PG1. In addition, maximum bending moments were observed at deeper depths in pile groups compared to single piles due to the fixation provided by the pile cap, which was more evident for larger-diameter piles, P7 and PG2. On the other hand, the rate of increase in the depth with the PGA was lower for pile groups compared to single piles, as can be seen form Figure 5-16b. It should be noted that the depth of the maximum bending moment was calculated from the fitted bending moment curve for each case of loading.
Figure 5-12. Effect of earthquake intensity on maximum bending moment profiles for single piles: a) P2 in NOR earthquake; b) P2 in TAK earthquake; c) P7 in NOR earthquake; d) P7 in TAK earthquake
Figure 5-13. Effect of earthquake intensity on maximum bending moment profiles for pile groups: a) PG1 in NOR earthquake; b) PG1 in TAK earthquake; c) PG2 in NOR earthquake; d) PG2 in TAK earthquake
Figure 5-14. Effect of earthquake intensity on maximum shear force profiles for single piles: a) P2 in NOR earthquake; b) P2 in TAK earthquake; c) P7 in NOR earthquake; d) P7 in TAK earthquake
Figure 5-15. Effect of earthquake intensity on maximum shear force profiles for fixed pile groups: a) PG1 in NOR earthquake; b) PG1 in TAK earthquake; c) PG2 in NOR earthquake; d) PG2 in TAK earthquake
Figure 5-16. Variation of depth of maximum bending moment with PGA for single piles and fixed-head pile groups: a) Actual values; b) Normalized correlation

**Rocking behaviour of pile groups**

In addition to the flexural rigidity of the piles, the rocking resistance of the piles within a group is an important mechanism to resist the seismic loading. This rocking resistance is a function of the normal force developed in the piles during the seismic event. **Figure 5-17** presents the variation of the maximum normal forces with the PGA. As can be seen from **Figure 5-17a**, the increase in normal force was higher for larger-diameter pile group (PG2) as the higher flexural rigidity of the piles promoted the rocking behaviour over lateral deflection. In addition, larger normal forces were observed during resonance (NOR
earthquake) for both pile groups. On the other hand, the rate of increase in the maximum normal force was lower than the increase in the PGA for both pile groups, as seen from Figure 5-17b. However, the rate of increase when resonance was experienced was higher. This indicated that rocking behaviour increased during resonance and consequently, the normal forces increased.

Figure 5-17. Variation of maximum normal force with PGA for fixed-head pile groups: a) Actual values; b) Normalized correlation

Figure 5-18 presents the maximum normal force profiles along the pile shaft as obtained from strain measurements for different earthquake intensities. As can be seen from Figure 5-18, not only that the normal force increased by increasing the loading intensity, but also the extent of the maximum normal force increased as well. This means the piles experienced larger vertical movement due to rocking, and hence longer segments of the pile were engaged in providing resistance, which was more evident in larger-diameter piles (PG2). It is also observed that the normal forces increased when piles were moving away from the pile group center. This can be seen from Figure 5-18 in the case of negative (tension) normal force for PG1 and the positive (compression) normal force for PG2, which mainly depended on different position of piles (P2 in PG1 and P7 in PG2) within the pile group. On the other hand, when piles were moving towards the pile group center, the maximum normal force developed close to the ground surface and decreased in a linear fashion towards the pile tip.
Figure 5-18. Effect of earthquake intensity on maximum normal force profiles for fixed pile groups: a) PG1 in NOR earthquake; b) PG1 in TAK earthquake; c) PG2 in NOR earthquake; d) PG2 in TAK earthquake
Soil resistance

Figure 5-19 and Figure 5-20 show the maximum soil resistance profiles for different earthquake intensities for single piles and piles in a group, respectively. As expected, the soil resistance followed the same trends of pile deflection, i.e., increased at locations where pile deflection was significant. It is also noted that the inter-helical zone offered significant passive soil resistance, which explains the observed behavior of the piles (near fixation within the inter-helical zone). This again confirms the additional benefit of the helices in providing support to the pile under seismic loading, especially for large-diameter piles (P7 and PG2), in which the soil resistance profiles extended nearly to the pile tip.

On the other hand, Figure 5-21 presents the variation of the maximum soil resistance with PGA. As can be seen from Figure 5-21, a significant increase in the soil resistance was observed for single piles compared to pile groups, which is attributed to the relatively smaller deflections of the pile groups owing to the contribution of the rocking mechanism to resisting the seismic loading. In addition, the soil resistance was higher when resonance occurred, which was the case for single piles during TAK earthquake, and pile groups during NOR earthquake. Due to the direct relation between the pile deflection and the corresponding soil resistance, the rate of increase in the maximum soil resistance had a similar trend to that observed in the deflections. Single piles exhibited a much higher rate of increase compared to pile groups.
Figure 5-19. Effect of earthquake intensity on maximum soil resistance profiles for single piles: a) P2 in NOR earthquake; b) P2 in TAK earthquake; c) P7 in NOR earthquake; d) P7 in TAK earthquake
Figure 5-20. Effect of earthquake intensity on maximum soil resistance profiles for fixed pile groups: a) PG1 in NOR earthquake; b) PG1 in TAK earthquake; c) PG2 in NOR earthquake; d) PG2 in TAK earthquake
Figure 5-21. Variation of maximum soil resistance with PGA for fixed-head pile groups: a) Actual values; b) Normalized correlation

Dynamic p-y curves

The relation of the soil resistance to the pile deflection during earthquake events is evaluated in terms of dynamic p-y curves (hysteretic loops). Figure 5-22 displays the derived dynamic p-y curves for P2 due to different TAK earthquake intensities while Figure 5-23 presents the derived dynamic p-y curves for P7 due to different NOR earthquake intensities. As expected, both figures show that for low earthquake intensity, the hysteretic loops were almost linear. The size of hysteretic loops increased as the
earthquake intensity increased, which indicated a significant increase in soil nonlinearity and energy dissipation (damping). Moreover, the degradation in the pile-soil system’s stiffness was immensely larger in higher loading intensities. This was clearly evident from decreasing slopes of p-y curves at the same depth as the loading intensity increased.

The same observations were made for piles within group as seen from Figure 5-24 for PG1 due to different TAK earthquake intensities and Figure 5-25 for PG2 due to different NOR earthquake intensities. However, the hysteretic loops observed for pile groups were larger even for low earthquake intensities. This suggested that the pile groups provided higher damping compared to single piles, even for lower earthquake intensities.

Generally, for both single piles and pile groups, the p-y curves near the ground surface had lower slope (i.e., lower stiffness) and smaller loop area (i.e., small damping) due to low confining pressure and gaps forming around the pile shaft and soil caving in, resulting in loose sand adjacent to the pile. As the depth below ground surface increased, the size of hysteretic loops increased up to the depth of maximum soil resistance. The hysteretic loops became smaller and their slope increased indicating linear elastic behaviour. These changes are attributed to lower pile deflection and increased soil confinement. The maximum soil resistance was observed consistently at a depth of 5D for smaller-diameter piles and PG1, and 6D for larger-diameter piles and PG2.

Figure 5-22. Effect of TAK earthquake intensity on dynamic p-y curves for P2
Figure 5-23. Effect of NOR earthquake intensity on dynamic p-y curves for P7

Figure 5-24. Effect of TAK earthquake intensity on dynamic p-y curves for PG1
5.3.2 Effect of Pile Head Fixity Condition

Two pile-head fixity conditions were simulated during the experimental program. Initially, piles were connected to the pile caps using 2-bolts connection in order to simulate a fixed pile head condition. On the next test day, one bolt was removed from each connection to simulate a pinned pile head condition. However, the 2 bolts did not result in a true fixed connection (as opposed to welded connection) and one bolt did not produce a true pin connection. Both connections provided some level of rotational restraint to the pile’s head, but did not simulate a fully restrained connection (fixed pile head) nor a zero restrained connection (pinned pile head). Thus, pile groups exhibited similar behavior for both connections, which was closer to a fixed head condition rather than a pinned head condition, with the 1-bolt connection offering less restraint than the 2-bolts connection. For the sake of comparison, the 2-bolts connection is still referred to as (fixed) and the 1-bolt connection is referred to as (pinned) herein.

Figure 5-26 and Figure 5-27 compare the bending moments for fixed and pinned conditions for PG1 and PG2, respectively. The results show that the fixed pile group exhibited slightly higher maximum bending moment. Furthermore, by linearly extrapolating the bending moment profiles above the ground surface to the center of pile
cap, it may be inferred that the bending moment at the bottom of the pile cap changed sign indicating some level of restraint against rotation for both pile groups. In addition, neither of the connections displayed zero rotation at the pile cap. However, the pinned connection resulted in slightly larger rotations at the pile head as shown in Figure 5-28 and Figure 5-29 for PG1 and PG2, respectively. The deflected shape of piles for both pile head fixity conditions, and consequently, shear forces, soil resistances and dynamic p-y curves, were very similar for both head conditions. In addition, the developed normal forces in the piles were slightly larger for fixed pile groups in the case of uplift (tension forces), while for compression, both pile head conditions resulted in the same axial forces.

Figure 5-26. Effect of pile head connection on maximum bending moment for PG1: a) NOR-100 earthquake; b) TAK-100 earthquake
Figure 5-27. Effect of pile head connection on maximum bending moment for PG2: 
   a) NOR-100 earthquake; b) TAK-100 earthquake

Figure 5-28. Effect of pile head connection on maximum rotation for PG1: 
   a) NOR-100 earthquake; b) TAK-100 earthquake
Figure 5-29. Effect of pile head connection on maximum rotation for PG2:  
a) NOR-100 earthquake; b) TAK-100 earthquake

5.3.3 Pile-Soil-Pile Interaction

Piles within a group interact with each other through the soil when subjected to lateral loading. Due to the overlap of stress zones and active wedges between piles (i.e., group effect), the overall group stiffness is lower compared to the sum of stiffness of single piles within the group. The group effect is affected by the spacing between piles, i.e., the interaction between piles and the group effect are more significant as the spacing between piles decreases. On the other hand, the rocking behaviour of the group contributes to the resistance of applied seismic lateral loading.

Figure 5-30 and Figure 5-31 present the maximum bending moment profiles for two opposite piles within PG1 (P2 and P4) and PG2 (P7 and P9), respectively. It is noted from Figure 5-30 and Figure 5-31 that the bending moment distribution in the piles were sufficiently different near the soil surface indicating a significant interaction between piles. When the shaking direction reverses, the maximum bending moment in the group changes
from one pile to the other as a result of shadowing effect. As the pile moves into a softened soil zone created by adjacent piles, the load on that pile reduced, and that on the opposite pile increased. The change in the maximum bending moment is significantly larger for PG2 as a result of smaller spacing to diameter ratio, in which the change was about 50 % in PG2, while it was approximately 20 % in PG1.

On the other hand, slopes of the bending moment profiles near the soil surface indicated that the shear force in each pile differs by about 25 % in PG1 and 30 % in PG2 from the average in the group. This can be also seen from Figure 5-32 and Figure 5-33 for PG1 and PG2, respectively. This maximum shear force changes from one pile to the opposite one according to the shaking direction.

Figure 5-30. Maximum bending moment profiles in two opposite piles (P2 and P4) for fixed PG1: a) NOR-100 earthquake; b) TAK-100 earthquake
Figure 5-31. Maximum bending moment profiles in two opposite piles (P7 and P9) for fixed PG2: a) NOR-100 earthquake; b) TAK-100 earthquake

Figure 5-32. Maximum shear force profiles in two opposite piles (P2 and P4) for fixed PG1: a) NOR-100 earthquake; b) TAK-100 earthquake
The maximum axial force induced in piles within the group changes from one pile to the opposite pile during shakings as presented in Figure 5-34 and Figure 5-35 for PG1 and PG2, respectively. However, unlike the bending moment, the maximum normal force was observed in the pile when it was moving towards the softened zone, i.e., the center of the pile group. Since, the lateral resistance (flexural resistance) and rocking resistance are the two main components of the total capacity of a pile group; they both are complementary to each other. This means that, whenever the flexural capacity of the pile decreases, the axial stiffness would compensate this loss in the capacity and vice versa. However, the contribution of the axial stiffness was found to be very significant compared to the flexural stiffness of piles within a group, this would be demonstrated in the comparison of single piles and piles within a group in latter section.
Figure 5-34. Maximum normal force profiles in two opposite piles (P2 and P4) for fixed PG1: a) NOR-100 earthquake; b) TAK-100 earthquake

Figure 5-35. Maximum normal force profiles in two opposite piles (P7 and P9) for fixed PG2: a) NOR-100 earthquake; b) TAK-100 earthquake
In terms of rotations and deflections, the maximum value was observed when piles were vibrating towards the pile cap vicinity despite the fact that the load carried by the pile decreased. As a result of shadowing effects due to pile-soil-pile interaction, the soil within the pile cap vicinity would have lower stiffness and thus higher rotations and deflections were expected. Figure 5-36 and Figure 5-37 show the maximum rotation profiles in two opposite piles for PG1 and PG2, respectively. In addition, Figure 5-38 and Figure 5-39 present the maximum deflection profiles for PG1 and PG2, respectively. Accordingly, because of the direct relation between pile deflection and the mobilized soil resistance, the maximum soil resistance was observed in each pile when it was shaking towards the softened soil. This could be seen in Figure 5-40 and Figure 5-41 for PG1 and PG2, respectively.

Figure 5-36. Maximum rotation profiles in two opposite piles (P2 and P4) for fixed PG1: a) NOR-100 earthquake; b) TAK-100 earthquake
Figure 5-37. Maximum rotation profiles in two opposite piles (P7 and P9) for fixed PG2: a) NOR-100 earthquake; b) TAK-100 earthquake

Figure 5-38. Maximum deflection profiles in two opposite piles (P2 and P4) for fixed PG1: a) NOR-100 earthquake; b) TAK-100 earthquake
Figure 5-39. Maximum deflection profiles in two opposite piles (P7 and P9) for fixed PG2: a) NOR-100 earthquake; b) TAK-100 earthquake

Figure 5-40. Maximum soil resistance profiles in two opposite piles (P2 and P4) for fixed PG1: a) NOR-100 earthquake; b) TAK-100 earthquake
5.3.4 Normalized Responses: Comparison between Single Piles and Group Piles

Responses of single piles and group piles were normalized in order to facilitate comparing the performance of different soil-pile systems with different properties. The two main parameters that varied for test pile were pile cap masses and free pile length (pile stick out above ground surface to center of mass of the pile cap). These two parameters resulted in different lateral forces and pile head moment. Table 5-3 provides the normalization factors used in this study for each response component, in which $M_{gs}$ is the bending moment at the ground surface and $r_p$ is the pile’s radius. This normalization technique does not account for the effect of the natural frequency of different systems, thus it should be interpreted in conjunction with the resonance condition for a specific earthquake.
Table 5-3. Normalization factors

<table>
<thead>
<tr>
<th>Response Component</th>
<th>Normalization Constant</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending Moment</td>
<td>$M_o = M_{gs}$</td>
</tr>
<tr>
<td>Rotation</td>
<td>$S_o = M_{gs}\tau_p/E_p I_p$</td>
</tr>
<tr>
<td>Deflection</td>
<td>$\gamma_o = M_{gs}\tau_p^2/E_p I_p$</td>
</tr>
<tr>
<td>Shear Force</td>
<td>$Q_o = M_{gs}/\tau_p$</td>
</tr>
<tr>
<td>Soil Resistance</td>
<td>$p_o = M_{gs}/\tau_p^2$</td>
</tr>
<tr>
<td>Normal Force</td>
<td>$N_o = M_{gs}/\tau_p$</td>
</tr>
</tbody>
</table>

Figure 5-42 and Figure 5-43 compare the normalized bending moment profiles for P2 and P7 as a single and within fixed and pinned pile groups. For the pile within the group, the normalizing moment at the ground surface is the sum of measured bending moment and coupling moment resulting from the axial forces. The following observations could be made from Figure 5-42 and Figure 5-43:

1. Even though pile groups were subjected to significantly higher lateral load than single piles (approximately 8 times), responses of piles within the group were significantly lower than single piles. This is attributed to the group behaviour in which the axial forces developed in piles contributed to the total pile group rocking resistance.

2. The rocking component contribution to the pile group total resistance was significantly higher than the flexural resistance of individual piles. This is evidenced by the normalized bending moments at the ground surface, which indicated that the contribution of the flexural resistance for individual piles within contributed only 17% of the group capacity for PG1 and 20% for PG2.

3. The bending moment profile for single piles indicate no change in signs if extrapolated to the center of pile-head mass (i.e., theoretical location of zero bending moment). On the other hand, the bending moment profiles for fixed and pinned pile groups indicate a sign change if extrapolated to the center of the pile cap, which is attributed to the restraint at the pile head.
Figure 5-42. Normalized bending moment profiles for P2 as a single pile and within fixed and pinned PG1: a) NOR-100 earthquake; b) TAK-100 earthquake

Figure 5-43. Normalized bending moment profiles for P7 as a single pile and within fixed and pinned PG2: a) NOR-100 earthquake; b) TAK-100 earthquake
Similar observations were made for rotations experienced by single piles versus group piles, i.e., single piles experienced significantly larger rotations compared to pile groups as shown in Figure 5-44 and Figure 5-45 for P2 and P7, respectively. Deflections of single piles and group followed the same trend, i.e., piles within a group experienced less deflections than single piles as presented in Figure 5-46 and Figure 5-47 for P2 and P7, respectively. Even though single piles did not experience resonance during NOR earthquake, they exhibited larger normalized deflections compared to corresponding group piles that experienced resonance.

Figure 5-44. Normalized rotation profiles for P2 as a single pile and within fixed and pinned PG1: a) NOR-100 earthquake; b) TAK-100 earthquake
Figure 5-45. Normalized rotation profiles for P7 as a single pile and within fixed and pinned PG2: a) NOR-100 earthquake; b) TAK-100 earthquake

Figure 5-46. Normalized deflection profiles for P2 as a single pile and within fixed and pinned PG1: a) NOR-100 earthquake; b) TAK-100 earthquake
Figure 5-47. Normalized deflection profiles for P7 as a single pile and within fixed and pinned PG2: a) NOR-100 earthquake; b) TAK-100 earthquake

The normalized shear force for single piles group piles at the ground surface was nearly the same. However, single piles experienced significantly higher shear forces along the pile shaft compared to piles within the groups, especially for smaller-diameter piles (P2 and PG1). Moreover, the maximum shear force for single piles occurred at some depth below ground surface, while it occurred at the ground surface for pile groups. Meanwhile, the same normalized shear force profile was observed for both single piles and group piles with peak value along the pile shaft occurred at nearly the same location. This can be seen in Figure 5-48 and Figure 5-49 for P2 and P7, respectively.
Figure 5-48. Normalized shear force profiles for P2 as a single pile and within fixed and pinned PG1: a) NOR-100 earthquake; b) TAK-100 earthquake

Figure 5-49. Normalized shear force profiles for P7 as a single pile and within fixed and pinned PG2: a) NOR-100 earthquake; b) TAK-100 earthquake
Figure 5-50 and Figure 5-51 compare the normalized soil resistances provided by the soil in the case of single piles and group piles for P2 and P7, respectively. Due to the direct relation between deflections and soil resistances, single piles did experience larger normalized deflections, and in turn, larger soil resistances were observed. The only case that the response of group piles was actually larger than single piles was noticed in P7 and PG2 during NOR-100 earthquake, however they were very close. As mentioned before, the reason is that P7 had a natural frequency very far from the NOR-100 earthquake frequency content, while the natural frequency of PG2 was in resonance with NOR-100 earthquake. A different interpretation could be that the same normalized soil resistance was observed for both P7 and PG2; however, P7 did not experience any resonance, while PG2 did. This still indicated that pile groups were favorable to single piles especially when resonance is to be expected.

Figure 5-50. Normalized soil resistance profiles for P2 as a single pile and within fixed and pinned PG1: a) NOR-100 earthquake; b) TAK-100 earthquake
Figure 5-51. Normalized soil resistance profiles for P7 as a single pile and within fixed and pinned PG2: a) NOR-100 earthquake; b) TAK-100 earthquake

5.4 Conclusions

Different responses of single helical piles and helical pile groups were used to investigate the effect of earthquake characteristics including intensity as well as frequency content. Based on the results, the following conclusions can be made:

1- Resonance condition significantly increased responses of single piles and pile groups to almost 200 % even for the same PGA. Hence, the earthquake frequency content and potential for resonance must be considered in seismic design.

2- As expected, the responses of single piles and pile groups, including rotations and deflections increased as the earthquake intensity increased. However, rotations and deflections of piles within a group increased linearly with a lower rate than the earthquake intensity, while rotations and deflections of single piles increased exponentially with earthquake intensity due to gap formations and rapid loss of soil stiffness around the top portion of piles.

3- Straining actions of single piles and pile groups, including bending moments and shear forces increased linearly with earthquake intensity. However, the rate of
increase of straining actions for single piles was much higher due to the degradation in soil-pile strength.

4- Depth of point of maximum bending moment increased as earthquake intensity increased for both single piles and pile groups when resonance occurred. For single piles, the depth of point of maximum shear force increased as the earthquake intensity increased, for pile groups, maximum shear force was constantly at the pile head.

5- Maximum normal force developed in piles within a pile group increased in a linear fashion as earthquake intensity increased. However, the rate of increase in the normal force greatly increased when resonance occurred as rocking resistance represented the main contribution to the total group resistance.

6- Depth of point of maximum normal force increased as the earthquake intensity increased due to negative skin friction only when piles were moving away from the pile group center. When the shaking was in the other direction, maximum normal force occurred at the ground surface and decreased with depth towards the pile tip.

7- Dynamic soil resistance for single piles increased in a nonlinear fashion as the earthquake intensity increased, while it increased linearly for pile groups. In addition, the resonance condition greatly increased the response and promoted more nonlinear behaviour in single piles.

8- Larger loops were observed in the derived dynamic p-y curves for larger earthquake intensities indicating higher soil nonlinearity as well as higher damping. Lower intensities displayed a linear behaviour. In addition, degradation experienced in the soil stiffness increased by increasing the loading intensity. The same observation was made for p-y curves of pile groups; however, larger loops were pronounced even for low intensities. This indicated that pile groups could provide higher damping compared to single piles.

9- The 2-bolt connection and 1-bolt connection provided partial fixation, and resembled neither a fixed connection nor a pin connection. The behaviour of pile groups with either connection type was closer to fixed condition rather than pinned condition, and their responses to seismic loading were close.
10- Maximum responses within a pile group changed from one pile to the opposite one when shaking reversed due to Pile-Soil-Pile Interaction and shadowing effects. Loads carried by individual piles differ by about 25 % for PG1 and 30 % for PG2, while bending moments varied by about 20 % for PG1 and 50 % for PG2. It was found that the main factor was the Spacing to Diameter (S/D) ratio.

11- Maximum bending moment as well as shear force occurred in piles within a pile group when they were moving away from the softened soil zone, and in the vicinity of the pile group. On the contrary, maximum rotations, deflections, and soil resistances were observed when piles were moving towards the pile cap center despite the lower load carried individual piles.

12- Rocking stiffness of the piles within a group due to normal forces and the flexural stiffness of the individual piles were complementary to each other. The rocking stiffness due to axial loads increased as the flexural capacity of individual piles decreased, and vice versa. Thus, unlike bending moment, the piles maximum normal force occurred when the piles were moving towards the softened soil zone.

13- Single piles normalized responses were significantly higher than that for piles within a group. This is due to the significant contribution of the rocking stiffness to the overall capacity of the pile group compared to the flexural stiffness of the individual piles (almost 80 %). This highlights the advantage of helical pile groups in resisting seismic loading.
Chapter 6

Numerical Modelling: Model Construction and Validation

6.1 Introduction

The Finite Element Method (FEM) was employed to simulate the full-scale shaking table experiment for pile groups. The software ABAQUS (SIMULIA 2013a) was utilized to construct a full three-dimensional (3D) nonlinear dynamic model with the same geometric dimensions as the experimental setup. This chapter presents the different aspects of the developed model including 3D geometry, material models, pile-soil contact formulation, mesh properties, element type, applied ground motion, boundary conditions and analysis steps. Moreover, steps involved in the calibration of the model are presented to delineate the effect of varying some parameters on the responses of different parts of the numerical model, and finally, to achieve an acceptable match with the experimental results. The validation of the developed model is demonstrated in terms of dynamic characteristics (natural frequency and damping) as well as structural responses (piles’ bending moments and deflections).

6.2 FEM Model Development

6.2.1 3D Geometry Construction

Different parts of the experimental setup were modelled with the same physical dimensions. This included piles, pile cap and soil block. The soil block enclosed in the laminar shear box used in the shaking table test was modeled with the dimensions of the sand bed (i.e., 6.71 m long x 2.90 m wide x 4.57 m high). Piles were modeled as steel pipes with outer diameter of 140 mm and thickness of 10.5 mm, length of 4.22 m and depth of 3.35 m. Helices were idealized as a circular plate with the same diameter (0.254 m) and thickness (12.7 mm) and placed at the middle of their pitch. This simulation is widely adopted and has minimal effect on the lateral behaviour of helical piles (Kurian and Shah 2009). The pile cap was modelled with the same physical dimensions, i.e., 2.13 m x 2.13 m x 1.73 m (L x W x H). Figure 6-1 shows the shapes and dimensions of different parts of
the model, while Figure 6-2 presents an overview of the assembled pile group as well as the pile group within the soil block. Although two pile groups (PG1 and PG2) were installed in the sand bed during the experimental program, it was assumed that no interaction would occur between them as the spacing between PG1 and PG2 was 2.43 m, which corresponded to 17.4 d (diameter of larger piles) and 9.6 D (diameter of helices). Hence, only PG2 was modelled in the center of the soil block as can be seen in Figure 6-2b.

Figure 6-1. Geometric properties of different model parts: a) Soil block; b) Pile cap; c) Steel piles
6.2.2 Material Models

6.2.2.1 Piles

Piles were modelled as a linear-elastic-perfectly-plastic material, in which the pile initially behaves as an elastic material until the stress reaches the yielding stress of steel. After yielding, the pile starts to deform plastically. Although a post yielding stress-strain path could be defined in ABAQUS, yielding of steel piles was not expected, so only the limiting yield stress was defined.

6.2.2.2 Pile Cap

As the response of the pile cap is not of interest in this study, it was modelled as a rigid body and piles were fixed at the pile cap connection. A rigid body can be modelled directly in ABAQUS, however due to convergence issues, it was idealized as an elastic material that was assigned a very high Young’s modulus, three orders of magnitude higher than adjacent materials in order to avoid singularities during the numerical solution. The total...
mass of the pile cap applied in the experimental setup was defined. A summary of material properties of piles and pile cap is given in Table 6-1.

| Table 6-1. Material properties of piles and pile cap |
|-----------------|-----------------|-----------------|-----------------|
| Part            | Density (kg/m²³) | Young’s Modulus (GPa) | Poisson’s Ratio | Yielding Stress (MPa) |
| Steel Piles     | 7850            | 200              | 0.3            | 552               |
| Pile Cap        | 1267            | 200000           | 0.3            | –                 |

6.2.2.3 Sand Bed

Many comprehensive models have been developed to describe the complex cyclic behaviour of sand (Hashiguchi 1989, Collins and Kelly 2002, Allotey and El Naggar 2008). Although some models are implemented in FEM software (Manzari et al. 2019), there is still difficulties in direct numerical implementation either because some of these software are not open-source, or it is not directly implemented and user-developed subroutines are required to introduce the material model into the software. On the other hand, identifying the model parameters by means of standard material testing is still a challenge (Zhan et al. 2012). Meanwhile, an elastic-perfectly-plastic model with Mohr-Coulomb failure criterion, usually referred to as Mohr-Coulomb (MC) model, is widely used in FEM analyses due to its simplicity and sufficient accuracy (Chen and Saleeb 1983). However, as discussed by Desai and Zaman (2013), conventional soil plasticity models, such as the MC model, provide good prediction of ultimate or failure strength of the material, but do not provide realistic predictions for the entire stress-strain response.

The soil around the piles and in the vicinity of pile groups was expected to experience significant nonlinearity, which leads to formation of gaps around the top portion of piles during strong shakings. Thus, the overall piles’ response was mainly governed by the soil plastic behaviour. In addition, the main interest of this study was to predict the performance of pile groups under strong shakings and to explore the contribution of helices to the structural responses of piles, rather than studying the stress-strain relation of sand. Hence, the MC yielding criterion was deemed suitable and was employed in this study.
Previous studies have shown that the MC criterion was sufficient to model the dynamic Soil-Pile-Structure-Interaction (SPSI) as well as Soil-Pile-Interaction (SPI) problems. Al-Isawi et al. (2019) simulated a shaking table test of single piles with structural head masses in the software ABAQUS. Albusoda et al. (2020) studied the performance of single pile foundations subjected to earthquake excitations using ABAQUS. Fahmy and El Naggar (2016) studied the cyclic lateral performance of helical tapered piles in silty sand, and simulated the experimental tests numerically using ABAQUS. Peiris (2014) explored the SPI of piles embedded in deep layered marine sediments under seismic loadings using the software ABAQUS.

The MC material model was also employed for different applications of dynamic Soil-Structure-Interaction (SSI) problems. Rayhani and El Naggar (2008) modelled the seismic response of rigid foundation using the software FLAC3D (Itasca Consulting Group 2005). Amrei et al. (2020) used MC material model to explore the effects of near-field seismic components on circular metro tunnels. Zadeh (2020) studied the effects of SSI on the seismic response of Reinforced Concrete (RC) bridges employing the software ABAQUS. Qaftan (2019) modelled the response of Soil-Foundation-Structure-Interaction (SFSI) of tall buildings (frames and walls structural systems) under seismic loadings in ABAQUS software. These studies collectively indicated the sufficiency of the MC material model to simulate the behaviour of soil body in dynamic applications of general SSI and especially SPSI. This is further confirmed through the results of this study.

The MC criterion implemented in ABAQUS assumes that yield occurs when the shear stress at any point in a material reaches a value that depends linearly on the normal stress in the same plane as shown in Figure 6-3. In general, the MC model is defined by Equation (6-1), where $\tau$ is the shear stress, $\sigma$ is the normal stress (negative for compression), $c$ is the material’s cohesion and $\phi$ is its angle of internal friction.

$$\tau = c - \sigma \tan \phi \quad (6-1)$$

This can be written in terms of the principle stresses as given by Equation (6-2), where $\sigma_1$ and $\sigma_3$ are the maximum and minimum principle stresses, respectively.
\[ \frac{1}{2} (\sigma_1 - \sigma_3) + \frac{1}{2} (\sigma_1 + \sigma_3) \sin \phi - c \cos \phi = 0 \]  
\text{(6-2)}

In a 3D general state of stress, the model is more conveniently written in terms of three stress invariants given by:

\[ F = R_{mc} q - p \tan \phi - c = 0 \]  
\text{(6-3)}

where,

\[ R_{mc}(\Theta, \phi) = \frac{1}{\sqrt{3} \cos \phi} \sin \left( \Theta + \frac{\pi}{3} \right) + \frac{1}{3} \cos \left( \Theta + \frac{\pi}{3} \right) \tan \phi \]  
\text{(6-4)}

\( \Theta \) is the deviatoric polar angle defined as:

\[ \cos(3\Theta) = \left( \frac{r}{q} \right)^3 \]  
\text{(6-5)}

\( p \) is the equivalent pressure stress and is defined as:

\[ p = -\frac{1}{3} \text{trace}(\sigma) \]  
\text{(6-6)}

\( q \) is the Mises equivalent stress and is defined as:

\[ q = \sqrt{\frac{3}{2}} \sqrt{(S : S)} \]  
\text{(6-7)}

\( r \) is the third invariant of deviatoric stress and is defined as:

\[ r = \left( \frac{9}{2} S \cdot S : S \right)^{\frac{1}{3}} \]  
\text{(6-8)}

and finally, \( S \) is the deviatoric stress and is given by:

\[ S = \sigma + pI \]  
\text{(6-9)}
The resulting yield surface in the deviatoric plane is an irregular hexagonal shaped which is bounded by the well-known Tresca yielding surface (corresponding to $\phi = 0^\circ$) and Rankine tension cutoff yielding surface (corresponding to $\phi = 90^\circ$) as shown in Figure 6-4. Isotropic cohesion hardening is assumed for the hardening behaviour of the Mohr-Coulomb yield surface, in which the hardening curve is defined as the cohesion yielding stress in terms of the absolute plastic strain (SIMULIA 2013b).

**Figure 6-3.** Mohr-Coulomb yielding model (SIMULIA 2013b)

**Figure 6-4.** Mohr–Coulomb yielding surface (SIMULIA 2013b)
The flow potential adopted in ABAQUS for the MC model is the function proposed by (Menétrey and Willam 1995), which has a hyperbolic shape in the meridional stress plane and a smooth elliptic shape in the deviatoric stress plane. This non-associated flow potential, which is continuous and smooth, ensures that the flow direction is always uniquely defined (SIMULIA 2013b).

The main four parameters that define the Mohr-Coulomb plasticity model in ABAQUS are the friction angle, $\phi$, the cohesion, $c$, the dilation angle, $\psi$, and the absolute plastic strain at yielding, $\varepsilon^{pl}$. Although the soil used in the testing was pure sand, which implies a zero cohesion, a very small value was defined in ABAQUS to avoid any numerical errors, which corresponded to a zero absolute plastic strain. The internal friction angle and dilation angle were defined as obtained from field and lab tests performed during the experimental program. On the other hand, the elastic soil properties, Young’s modulus and Poisson’s ratio, as well as the density and Rayleigh damping coefficients were defined for the soil material.

The soil profile was divided into six layers as shown in Figure 6-5 and the assigned initial material properties are shown in Table 6-2. All soil layers had the same density of 1988 kg/m$^3$, Poisson’s ratio of 0.4 and dilation angle of 1.25°.
### Table 6-2. Soil material properties used in numerical analysis

<table>
<thead>
<tr>
<th>Layer</th>
<th>Young’s Modulus (MPa)</th>
<th>Friction Angle (°)</th>
<th>$K_s^*$</th>
<th>Rayleigh Damping Coefficients $^\text{**}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\alpha$</td>
</tr>
<tr>
<td>Layer 1</td>
<td>23</td>
<td>39</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 2</td>
<td>29</td>
<td>40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 3</td>
<td>35</td>
<td>42</td>
<td>0.3</td>
<td>2.08465</td>
</tr>
<tr>
<td>Layer 4</td>
<td>41</td>
<td>43</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 5</td>
<td>49</td>
<td>44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 6</td>
<td>52</td>
<td>45</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* At rest lateral earth pressure coefficient
** Values corresponding to 5% viscous damping

### 6.2.2.4 Material Damping

Material viscous damping was defined in ABAQUS employing the Rayleigh damping which assumes that the damping is proportional to a linear combination of mass and stiffness of the system, i.e.,

$$[C] = \alpha [M] + \beta [K]$$ (6-10)

where, $[C]$ is the damping matrix, $[M]$ and $[K]$ are the mass and stiffness matrices, respectively, $\alpha$ and $\beta$ are Rayleigh damping coefficients and can be calculated from Equation (6-11), in which $\xi$ is the target damping ratio, $\omega_l$ and $\omega_f$ represents the desired frequency range.

$$\begin{bmatrix} \alpha \\ \beta \end{bmatrix} = \frac{2\xi}{\omega_l + \omega_f} \begin{bmatrix} \omega_l \omega_f \\ 1 \end{bmatrix}$$ (6-11)

The frequency range for Rayleigh damping was selected according to the method implemented in the software DEEPSOIL. In this method, the bounding frequencies are chosen as the first and third natural frequencies of the soil block, which can be calculated from Equation (6-12), where $n$ is the mode number, $V_s$ is the shear wave velocity and $H$ is the height of the soil block.
\[ f_n = \frac{(2n - 1) V_s}{4H} \]  \hspace{1cm} (6-12)

Due to its mathematical convenience, Rayleigh damping is widely used to model internal damping in FEM analyses. However, one of the less attractive features of Rayleigh damping is that the achieved damping ratio varies with the frequency and is not constant over the range of frequencies of interest and the system is over damped outside this frequency range. The stiffness proportional term governs the damping at higher frequencies and increases linearly with the frequency. On the other hand, the mass proportional term governs the damping at lower frequencies and is inversely proportional to the response frequency. **Figure 6-6** illustrates the variation of the two components as well as the overall resulting damping with frequency.

**Figure 6-6. Rayleigh damping variation with frequency**

6.2.3 Pile-Soil Interface

Several contact formulations can be used to simulate the pile-soil interface condition in ABAQUS. Each formulation involves contact surface discretization, tracking approach and
assignment of “master” and “slave” roles to contact surfaces. Generally, ABAQUS applies conditional constraints at various locations on interacting surfaces to simulate contact conditions. The locations and conditions of these constraints depend on the contact surface discretization used (SIMULIA 2013b).

The surface-to-surface discretization technique, which accounts for the shape of both the slave and the master surfaces in the region of contact, was used in this study. It considers an average region nearby slave nodes rather than only individual slave nodes when enforcing contact conditions, which represents a true contact between both surfaces (SIMULIA 2013b). The master surface should be chosen as the larger surface, the surface of the stiffer body and the surface with a coarser mesh (SIMULIA 2013b). Hence, the master surface was assigned to the pile and the slave surface was assigned to the soil. The finite-sliding tracking approach that allows for arbitrary relative separation, sliding and rotation of the contact surfaces was employed to allow for gaps formation around piles.

Two interaction properties were defined for the contact surfaces: tangential and normal. Tangential behaviour is defined by the isotropic Coulomb friction model. As long as the equivalent shear stress at contact is less than or equal to the critical shear stress, no slippage will occur. The critical shear stress, \( \tau_{\text{crit}} \), is a function of the normal stress, \( p \), and the friction coefficient, \( \mu \), which depends on the nature of the materials in contact, and is given by Equation (6-13).

\[
\tau_{\text{crit}} = \mu p
\]  \hspace{1cm} (6-13)

The friction coefficient was taken as 0.46 in the first step of the calibration of the numerical model based on the study performed by Aksoy et al. (2017). However, this value did not yield good match with the experimental results. Hence, a final value of 0.7 was used as recommended in the API RP-2A design guidelines (API 1991) for very dense sand and steel pipe piles interface.

The normal behaviour was defined as “hard” contact that allows for separation if the contact pressure is zero. In addition, it does not limit the magnitude of contact pressure
transmitted between surfaces when the clearance is zero, which prevents both surfaces from penetrating each other.

6.2.4 Mesh Properties

The mesh size and geometric properties have significant effect on the FEM accuracy and computational effort. The finer mesh offers more accurate solution but increases computational demands, especially for dynamic analysis. Hence, an optimum mesh size should be selected to yield minimal errors at reasonable computational efficiency. In addition, the element aspect ratio and maximum and minimum edge lengths should be selected to eliminate numerical divergence and allow proper wave propagation for dynamic loading.

6.2.4.1 Piles

A sensitivity analysis was performed in order to choose an appropriate mesh size that captures the correct lateral response of piles with a reasonable number of elements. A single pile was modelled and subjected to a static lateral force at the pile head, while fixing it at the helix level. The maximum deflection, $\delta$, at the pile head is compared to the analytical solution of the deflection of a cantilever beam under a point load, $P$, at the free end, i.e.,

$$\delta = \frac{PL^3}{3EI},$$

where $L$ is the unsupported length of the pile and $EI$ is its flexural rigidity. Applying a load of 10 kN to the pile head, the calculated deflection was 12.40 cm.

The auto-mesh generated in ABAQUS had a size of 0.14 m, and its properties are presented in the first row of Table 6-3. To examine the effect of element size in each dimension of the pile, i.e., thickness elements ($N_t$), perimeter elements ($N_p$) and length elements ($N_L$), each was doubled and the corresponding deflection was calculated. Table 6-3 shows that varying $N_t$ and $N_L$ had minimal effect on the calculated deflection. On the other hand, doubling $N_p$ reduced the error from 6.9 % to 1.9 %. Accordingly, the final mesh was selected maintaining a maximum aspect ratio of 10, which resulted in 1696 elements for each pile as shown in Figure 6-7.
### Table 6-3. Piles’ sensitivity analysis results and final mesh properties

<table>
<thead>
<tr>
<th>Mesh</th>
<th>Number of Elements</th>
<th>Total Number of Elements</th>
<th>Average Aspect Ratio / Maximum Aspect Ratio</th>
<th>δ (cm)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Default</td>
<td>1 16 29</td>
<td>592</td>
<td>11.55 / 13.65</td>
<td>13.26</td>
<td>6.94</td>
</tr>
<tr>
<td>2N_t</td>
<td>2 16 29</td>
<td>1120</td>
<td>23.41 / 27.31</td>
<td>13.23</td>
<td>6.66</td>
</tr>
<tr>
<td>2N_L</td>
<td>1 16 58</td>
<td>1072</td>
<td>6.38 / 6.83</td>
<td>13.27</td>
<td>7.01</td>
</tr>
<tr>
<td>2N_p</td>
<td>1 32 29</td>
<td>1216</td>
<td>11.03 / 13.65</td>
<td>12.64</td>
<td>1.91</td>
</tr>
<tr>
<td>Final</td>
<td>1 32 40</td>
<td>1696</td>
<td>7.94 / 9.77</td>
<td>12.64</td>
<td>1.94</td>
</tr>
</tbody>
</table>

**Figure 6-7. Pile’s mesh**

6.2.4.2 Soil Block

For wave propagation problems, the minimum number of elements per wavelength should be limited to (8 - 10) to prevent any dispersion or unreal attenuation in the wave amplitude due to not accounting for shorter wavelengths ([Smith 1975](#)). Thus, the maximum element size is a function of the soil shear wave velocity, $V_s$, and the maximum circular frequency of the applied dynamic load, $\omega_{max}$, as given by **Equation (6-14)**.
Maximum element Size = \frac{V_s}{(8 - 10)\omega_{max}} \quad (6-14)

For \( V_s = 100 \, m/s \) and \( \omega_{max} = 9 \times 2\pi \) (for NOR earthquake motion with maximum frequency of 9 Hz), the maximum element length was 0.163 m.

In addition, to ensure convergence in the contact algorithms in ABAQUS, the slave surface mesh (soil) has to be smaller than or equal to the master surface mesh (pile), which required a very fine mesh around the piles, and the element size increased gradually towards the boundaries while maintaining a maximum aspect ratio of 10. As a result, the number of elements in the soil mesh was 105984. The final soil mesh is shown in Figure 6-8.

![Figure 6-8. Soil block’s mesh](image)

6.2.4.3 Pile Cap

As the pile cap’s response is not of interest in this study, it was modeled as a rigid body. The connection between the pile cap and the piles was assumed to be fixed. In order to attain the mesh compatibility between the piles and the pile cap, the size of elements in the pile cap should be the same as the pile at the connection. This led to very small element sizes at the pile cap’s bottom surface, which was increased gradually away from the pile.
resulting in 19072 elements. An overview of the mesh of the assembled pile group is presented in **Figure 6-9**. The total number of elements in the final model arrangement was 131840. A summary of the mesh sizes and properties are shown in **Table 6-4**.

![Figure 6-9. Assembled pile group’s mesh](image)

**Table 6-4. Final mesh sizes and properties for different parts**

<table>
<thead>
<tr>
<th>Part</th>
<th>Number of Elements</th>
<th>Aspect Ratio</th>
<th>Min Edge Length (m)</th>
<th>Max Edge Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piles</td>
<td>1696</td>
<td>7.94 / 9.77</td>
<td>0.0102 / 0.0064</td>
<td>0.083 / 0.103</td>
</tr>
<tr>
<td>Pile Cap</td>
<td>19072</td>
<td>7.83 / 10.0</td>
<td>0.0410 / 0.0082</td>
<td>0.144 / 0.144</td>
</tr>
<tr>
<td>Soil Block</td>
<td>105984</td>
<td>5.42 / 9.49</td>
<td>0.0525 / 0.0064</td>
<td>0.121 / 0.157</td>
</tr>
</tbody>
</table>
6.2.5 Element Type

Linear hexahedral elements (8 nodes – C3D8), which have three translational degrees of freedom at each node, were selected to provide acceptable accuracy at reasonable computational effort. Gaussian quadrature method is used to evaluate the material response at each integration point in each element. Reduced integration (1 integration point in the center) and hourglass control were used to model piles, pile cap and the soil block.

6.2.6 Ground Motion

Two earthquake records were applied during the experimental program (Northridge (1994) recorded at Fire Station 108, USC station 5314, California, and the Kobe (1995) recorded at Takatori Station, Takatori, Japan). They are referred to herein as NOR and TAK earthquakes, respectively.

In the validation process of the numerical model, NOR earthquake was used as the input ground motion. The original earthquake record had a sampling frequency of 256 Hz with a time duration of 45 seconds. This implied a total number of 11520 data points, which resulted in unnecessarily long computational time. To reduce the size of the earthquake record, both sampling frequency and time duration were reduced as described below.

6.2.6.1 Sampling Frequency

The original time history (256 Hz) was resampled with lower sampling frequencies without affecting the frequency content. Figure 6-10 and Figure 6-11 show the time history and Fourier amplitude, respectively, for NOR earthquake record with different sampling frequencies ranging from 256 Hz to 32 Hz. It was found that using a sampling frequency of 32 Hz, which implied a maximum frequency content of 16 Hz, resulted in a reasonable number of data points without affecting the frequency content of the original earthquake record. Table 6-5 summarizes the number of data points corresponding to different sampling frequencies.
Figure 6-10. NOR earthquake record with different sampling frequencies

Figure 6-11. Frequency content of NOR earthquake with different sampling rates
Table 6-5. Number of data points for different sampling frequencies

<table>
<thead>
<tr>
<th>Sampling Frequency (Hz)</th>
<th>Number of Data Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>256</td>
<td>11520</td>
</tr>
<tr>
<td>128</td>
<td>5760</td>
</tr>
<tr>
<td>64</td>
<td>2880</td>
</tr>
<tr>
<td>32</td>
<td>1440</td>
</tr>
</tbody>
</table>

6.2.6.2 Time Duration

The duration of the earthquake record is typically selected as a function of a predefined cutoff acceleration, in which values lower than the cutoff acceleration are not considered. The acceleration record was initially truncated between 10 and 30 seconds, which corresponded to a 0.02 g cutoff acceleration and resulted in 643 data points. The analysis time for this ground motion was 160 hrs (6.7 days) on a 32-core processor. Consequently, different cut-off frequencies were considered to optimize the computational time as shown in Table 6-6. Figure 6-12 and Figure 6-13 compare time histories and frequency contents for different cutoff accelerations. Figure 6-13 shows that a 0.065 g had a minimal effect on the frequency content of the earthquake record, which was considered a reasonable choice. The final record employed in further analyses had a cutoff acceleration of 0.065 g with 32 Hz sampling frequency, resulting in 8.94 s record with 287 data points.

Table 6-6. Duration, number of data points, and running time for different cutoff accelerations

<table>
<thead>
<tr>
<th>Cutoff Acceleration (g)</th>
<th>Number of Data Points</th>
<th>Time Duration (s)</th>
<th>Number of Cores</th>
<th>Running Time (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.02</td>
<td>643</td>
<td>20</td>
<td>32</td>
<td>160</td>
</tr>
<tr>
<td>0.05</td>
<td>406</td>
<td>12.66</td>
<td>64</td>
<td>71</td>
</tr>
<tr>
<td>0.065</td>
<td>287</td>
<td>8.94</td>
<td>64</td>
<td>58</td>
</tr>
</tbody>
</table>
Figure 6-12. NOR earthquake record with different cutoff accelerations

Figure 6-13. Frequency content of NOR earthquake with different cutoff accelerations
6.2.7 Boundary Conditions

In static analysis, a fixed boundary can be applied at some distance from the region of interest. Meanwhile, dynamic analysis of pile-soil interaction problems considers surrounding soil strata as infinite in the horizontal direction to prevent wave reflection at the boundary into the model. Therefore, non-reflecting boundaries should be defined to account for the infinite extent of the soil.

Viscous (absorbing) boundaries, which involve dashpots attached to the sides of the model, can be used to absorb the impeding waves and prevent their reflection into the model (Lysmer and Kuhlemeyer 1969). Alternatively, infinite boundaries that have a virtual infinite thickness with fixed far nodes can be used. Both methods can be employed when the dynamic load is applied within the soil block (e.g., vibrating machine problems). However, when the dynamic load is applied through the boundary (e.g., earthquake excitation), fixed boundaries cannot be employed as the soil block itself is displaced. Moreover, absorbing boundaries and infinite boundaries would attenuate the travelling wave as it travels up the soil block, such phenomenon is called leakage (Nielsen 2014). Two alternate solutions were proposed for these types of dynamic problems (Zienkiewicz et al. 1989): tied boundaries and free-field boundaries.

In the tied boundaries methodology, the degrees of freedom in the direction of shaking in the lateral boundaries of the model are assumed to be tied. This can exactly simulate the shaking of the laminar shear box, in which any excitation that exits the right boundary will re-enter from the left boundary. However, the boundaries should be placed at sufficient distance, so it does not interfere with pile-soil interaction behaviour, and only the seismic wave is propagated through the boundary. In the free-field boundaries methodology, two independent soil columns, as well as viscous dashpots, are added next to the model boundaries, which are excited with the same input motion. The travelling wave through these soil columns represent the free-field response. The dashpots absorb the radiating energy while a coupling procedure between the free-field soil columns and the main model ensures an undisturbed free-field motion at model boundaries (Nielsen 2006, 2014).
In this study, the tied boundaries procedure was adopted. In order to implement the tied boundaries into ABAQUS, an equation constraint (SIMULIA 2013b) was applied between each two opposite nodes at the lateral sides of the soil block in the direction of shaking. This equation constraint has the form given by Equation (6-15), in which \( u_i \) is the displacement of nodes in x-direction (direction of shaking) and \( i \) and \( j \) are the node labels of two opposite nodes.

\[
 u_i^j - u_i^j = 0
\]  

Equation (6-15)

The equation constraint is applied such that the first term in the equation \( u_i^j \) is deactivated and replaced by the equivalent value from the equation constraint, in this case \( u_i^j \). A Python (Python Software Foundation 1995) script was developed to define these constraints. The script reads the geometry of the lateral sides in the shaking direction, arranges the nodes into pairs based on their levels and locations, applies the equation constraint between each pair and finally merges these updates to the input file form ABAQUS. Running this script added about 2 to 3 hours for the analysis time due to the large number of nodes.

In order to validate the tied boundaries application, a simple model that consisted of the soil block only was excited under an earthquake ground motion, and the responses were compared with the linear elastic, time domain solution obtained from DEEPSOIL. The soil movement was allowed only in the direction of shaking to approximately simulate a 1D wave propagation same as considered in DEEPSOIL. The soil block was modelled with the same dimensions and properties of the sand bed in the experimental setup. Table 6-7 summarizes the soil properties used in the validation process. The soil block was then excited by the Coyote earthquake record implemented in the ground motions library of DEEPSOIL.
Table 6-7. Soil properties used for validation of tied boundaries

<table>
<thead>
<tr>
<th>Soil Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (m)</td>
<td>4.57</td>
</tr>
<tr>
<td>Shear Wave Velocity (m/s)</td>
<td>100</td>
</tr>
<tr>
<td>Unit Weight (kN/m$^3$)</td>
<td>19.5</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.40</td>
</tr>
<tr>
<td>Damping Ratio (%)</td>
<td>5.00</td>
</tr>
</tbody>
</table>

Figure 6-14 compares the responses obtained from DEEPSOIL and ABAQUS using the tied boundaries in terms of the acceleration, velocity and displacement at the ground surface. Figure 6-14 shows an exact match, which validated the suitability of tied boundaries methodology. In addition, the calculated responses at the ground surface were exactly the same at the center and at the boundaries of the model, which indicated that the boundaries did not affect the wave propagation or reflect the outward waves. Figure 6-15 presents the Frequency Response Function (FRF), defined as the ratio between the surface responses to the input excitation. Exact match was observed for the first mode of vibration (horizontal), and little discrepancies in higher modes (rocking). This is due to the strict 1D solution of DEEPSOIL that does not account for the 3D geometry, which is accounted for in ABAQUS.

6.2.8 Analysis Steps

The analysis of the full numerical model was divided into four steps: initial step, geostatic step, pile insertion step and dynamic shaking step.

6.2.8.1 Initial Step

In this step, boundary conditions of the model as well as a predefined geostatic stress field were defined. Vertical boundaries of the model were fixed in both horizontal directions ($U_x = 0$ & $U_y = 0$), while the base of the model was fixed in the three translational directions ($U_x = 0$, $U_y = 0$ & $U_z = 0$). The predefined geostatic field was simply a linear stress distribution starting from zero at ground surface and had a value of ($-\gamma h$) at the base, in which $\gamma$ is the sand’s unit weight and $h$ is the sand’s height.
Figure 6-14. Comparison between responses obtained from DEEPSOIL and ABAQUS with tied boundaries: a) Acceleration; b) Velocity; c) Displacement
6.2.8.2 Geostatic Step

In the geostatic step, only the soil block was activated and the weight of the soil block was applied. The applied load was equilibrated with the predefined geostatic stress, which should ideally cancel and results in zero deformations. However, due to effects of boundary conditions and complexity of the mesh, the stress field required for equilibrium is usually different from the predefined field and the deformations are not exactly null. In this study, the deformations were very small, in the magnitude of $10^{-7}$ m. Hence, the obtained stress field during this step is then used as the initial stress field in subsequent steps. The same boundary conditions propagated from initial step were used.

6.2.8.3 Pile Insertion

In this step, the soil volume occupying piles’ locations was removed and the pile group was activated (wished in place). Contact interaction between piles and soil was activated as well. Own weight of the pile group was applied to initialize the contact stresses before applying the dynamic shaking. The same boundary conditions were used.
6.2.8.4 Dynamic Shaking

Dynamic integration schemes provided in ABAQUS can be either Explicit or Implicit. In the Explicit scheme, the values of dynamic quantities at a time step are based entirely on available values at the previous time step, which means the solution is determined without iterating by explicitly advancing the kinematic state from the end of previous increment. In addition, the global mass and stiffness matrices need not to be formed and inverted, which means the calculations in each increment are computationally efficient. However, the central difference operator used in ABAQUS/Explicit scheme for integrating the equations of motion is only conditionally stable depending on the size of time increment. The stability limit is equal to the time for an elastic wave to cross the smallest element dimension, which requires very small time increment for fine meshes. In this study, the stable time increment was in the order of magnitude of $10^{-7}$ s. In addition, ABAQUS/Explicit does not enforce equilibrium at each time increment as the system of nonlinear equations is only solved once at the beginning of the solution.

Meanwhile, ABAQUS/Implicit solver calculates the dynamic quantities based on values at previous step, as well as the values of the current step, which implies an unconditionally stable technique regardless of the step size allowing a better computational efficiency. In this study, the minimum time increment determined by ABAQUS/Implicit in the analysis was in the order of magnitude of $10^{-4}$ s, which is substantially larger than that needed for the ABAQUS/Explicit scheme. However, the stiffness matrix must be inverted and the set of nonlinear equilibrium equations must be solved in each step. A direct-integration dynamic procedure that uses the implicit Hilber-Hughes-Taylor operator to integrate the equations of motion is employed for this purpose. Hence, an ABAQUS/Implicit scheme was employed in this study.

ABAQUS provides an automatic incrementation method that increases or reduces the time step depending on the convergence rate. A maximum time step was defined the same as the time step of the earthquake load (0.03125 s) to ensure that the ground motion is captured accurately. The horizontal fixation in the x-direction (direction of shaking) was removed in this step and replaced with tied boundaries for vertical sides ($U_x = \text{tied} \& U_y = 0$) and the earthquake acceleration load for the bottom surface ($A_x = \text{EQ}, U_y = 0 \& U_z = 0$).
6.3 Effects of Different Parameters on Numerical Responses

The calibration process of the numerical model involved several steps until acceptable match with the experimental results was achieved. Different parameters were varied through the calibration process including soil stiffness and soil damping ratio, pile head connection modelling technique, and the friction coefficient of the pile-soil interface. The effect of varying each parameter is discussed in this section.

6.3.1 Effect of Damping Ratio of the Sand

A soil’s damping ratio of 5 % was initially chosen, based on the low-strain damping ratio calculated from the experimental results. However, to achieve a good match between the experimental responses and the FEM results, the damping ratio was increased to 10 %. This was expected because the damping ratio should increase as a function of higher shear strains during strong shaking. This value was in agreement with what was reported by Shahbazi et al. (2020a) for the case of pile groups under strong shakings.

Figure 6-16 compares the acceleration time history and frequency content at the soil surface for different damping ratios, while Figure 6-17 compares the pile cap responses. As can be seen from Figure 6-16 and Figure 6-17, the acceleration at the ground surface decreased significantly for a 10 % damping ratio, while the pile cap response has not changed, indicating a minimal effect of the soil’s damping ratio on the pile group response at strong shaking.

Figure 6-18 and Figure 6-19 present the maximum bending moments and maximum deflections experienced by two opposite piles in the pile group (P7 and P9) for different damping ratios, respectively. Both figures show that increasing the soil’s damping ratio had no effect on the piles structural responses.
Figure 6-16. Effect of damping ratio of the sand on the dynamic response of the soil block: a) Time history; b) Frequency content
Figure 6-17. Effect of damping ratio of the sand on the dynamic response of the pile group: a) Time history; b) Frequency content
Figure 6-18. Effect of damping ratio of the sand on the maximum bending moment in piles: a) P7; b) P9

Figure 6-19. Effect of damping ratio of the sand on the maximum deflection of piles: a) P7; b) P9
6.3.2 Effect of Pile Head Connection Modelling Technique

Two different techniques were considered for modelling the fixed pile head connection with the pile cap. First, the whole pile group was modelled as one part. A large number of elements was required to model the pile cap to be compatible with the piles’ meshes. Second, piles and pile cap were modelled as separate parts and a tie constraint was imposed at connection of each pile head with the pile cap. This reduced the number of pile cap elements from 19072 to 800. However, the nonlinearity introduced by the tie constraint increased the computational time significantly, and both models yielded the same running time. Moreover, both models resulted in the same dynamic responses for both the soil and the pile group, as can be seen in Figure 6-20. Therefore, the whole pile group was modelled as one part in further analyses.

6.3.3 Effect of Young’s Modulus of the Sand

The soil Young’s modulus profile was increased slightly with depth to reflect the expected increase in soil stiffness as a function of confining pressure. The values used were within the range of Young’s modulus for medium to dense sand (Bowles 1988). A summary of the initial and correlated values of Young’s modulus are shown in Table 6-8.

<table>
<thead>
<tr>
<th>Layer</th>
<th>E initial (MPa)</th>
<th>E correlated (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>23</td>
<td>35</td>
</tr>
<tr>
<td>Layer 2</td>
<td>29</td>
<td>40</td>
</tr>
<tr>
<td>Layer 3</td>
<td>35</td>
<td>45</td>
</tr>
<tr>
<td>Layer 4</td>
<td>41</td>
<td>50</td>
</tr>
<tr>
<td>Layer 5</td>
<td>49</td>
<td>58</td>
</tr>
<tr>
<td>Layer 6</td>
<td>52</td>
<td>63</td>
</tr>
</tbody>
</table>

This resulted in a minor change in the soil block’s natural frequency, however, the PGA has not changed as can be seen in Figure 6-21a. The pile cap’s response changed slightly as can be seen in Figure 6-21b, however, this change was very small and has not achieved the required match with the experimental. Hence, further adjustments were needed in the calibration of the numerical model.
Figure 6-20. Effect of the tie connection on the dynamic response: a) Soil block; b) Pile cap
Figure 6-21. Effect of Young’s modulus of the sand on the dynamic response:
   a) Soil block; b) Pile cap
6.3.4 Effect of Pile-Soil Friction Coefficient

The final step of the calibration process focused on the effect of the friction coefficient of the pile-soil interface. Initially, the friction at sand-pile interface was assigned a value of 0.46 (Aksoy et al. 2017). To improve the match with measured responses, this value was then increased to 0.7 as recommended in the API RP-2A design guidelines (API 1991) for very dense sand and steel pipe piles interface.

The soil’s PGA decreased slightly as the friction coefficient increased as shown in Figure 6-22a. On the other hand, the pile group’s response was slightly affected by the friction coefficient as presented in Figure 6-22b. However, a significant reduction in the maximum bending moment as well as the maximum deflection in piles was observed for the case of a 0.7 friction coefficient compared to the case of a 0.46 as shown in Figure 6-23 and Figure 6-24, respectively. This was mainly observed when piles were moving towards the pile group vicinity, while a negligible change was observed when piles were moving in the opposite direction. This was the case for the (+ve) deflection of pile P7 and (-ve) deflection of pile P9. Because the soil outside the pile group was stronger and stiffer than the soil within the pile group as a result of high nonlinearity and pile-soil-pile interaction, increasing the friction coefficient had a higher effect on the outer sides of the piles. This would provide higher friction resistance when piles were moving towards the pile cap vicinity.

6.4 Final Validation with Experimental Results

Based on the calibration of the numerical model, the final material properties employed to simulate the soil behaviour are shown in Table 6-9. All soil layers had the same density of 1988 kg/m³, Poisson’s ratio of 0.4 and dilation angle of 1.25 °. In addition, the friction coefficient at the pile-soil interface was assigned a value of 0.7.
Figure 6-22. Effect of pile-soil friction coefficient on the dynamic response: a) Soil block; b) Pile cap
Figure 6-23. Effect of pile-soil friction coefficient on maximum bending moment in piles: a) P7; b) P9

Figure 6-24. Effect of pile-soil friction coefficient on the maximum deflection of piles: a) P7; b) P9
Table 6-9. Final sand properties used in numerical analysis

<table>
<thead>
<tr>
<th>Layer</th>
<th>Young’s Modulus (MPa)</th>
<th>Friction Angle (°)</th>
<th>$K_s^*$</th>
<th>Rayleigh Damping Coefficients $^{**}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>35</td>
<td>39</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 2</td>
<td>40</td>
<td>40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 3</td>
<td>45</td>
<td>42</td>
<td>0.3</td>
<td>5.2677</td>
</tr>
<tr>
<td>Layer 4</td>
<td>50</td>
<td>43</td>
<td>0.3</td>
<td>5.2677</td>
</tr>
<tr>
<td>Layer 5</td>
<td>58</td>
<td>44</td>
<td>0.3</td>
<td>5.2677</td>
</tr>
<tr>
<td>Layer 6</td>
<td>63</td>
<td>45</td>
<td>0.3</td>
<td>5.2677</td>
</tr>
</tbody>
</table>

* At rest lateral earth pressure coefficient
** Values corresponding to 10% viscous damping

The calibrated numerical model was first employed to evaluate the natural frequency and damping ratio of the soil block and the pile group. The calculated accelerations were compared with those obtained from the accelerometers installed within the soil block as well as on the pile cap. Figure 6-25 compares the acceleration time history and its frequency content obtained from the FEM model and the record of the top accelerometer within the sand bed in the experimental setup. As we can see form Figure 6-25, a very good agreement was obtained in terms of the PGA and the frequency content of the soil block. This indicated that the calibrated values of the soil Young’s Modulus and damping ratio represented the soil stiffness and damping correctly.

On the other hand, Figure 6-26 compares the acceleration computed at the mid-height of the pile cap from the FEM model with the readings of the accelerometers from the experimental testing. As shown in Figure 6-26, a good match between the experimental results and the FEM model was achieved, which indicated that the criterion used to model the pile-soil interface was able to reproduce the real behaviour in terms of nonlinear deformations and gap formations around the piles. This was the main parameter affecting the natural frequency of the pile group as discussed in earlier chapters.
Figure 6-25. Comparison of acceleration in the soil block from ABAQUS and experimental data: a) Time history; b) Frequency content
Figure 6-26. Comparison of acceleration at the pile cap from ABAQUS and experimental data: a) Time history; b) Frequency content
Figure 6-27 and Figure 6-28 compare the maximum bending moments and maximum deflections in piles from ABAQUS and the experimental results, respectively, from which a close match could be observed.

Figure 6-27. Comparison of maximum bending moment in piles from ABAQUS and experimental results: a) P7; b) P9

Figure 6-28. Comparison of maximum deflection of piles from ABAQUS and experimental results: a) P7; b) P9
In addition, the time histories of the bending moment for two opposite piles in the pile group (P7 and P9) obtained from the FEM model are compared with the experimental results in Figure 6-29. The bending moment is compared at the depth where the maximum bending moment occurred. As seen from Figure 6-29, a very good agreement was achieved in terms of the actual values and the natural frequency of the system.

Figure 6-29. Comparison of bending moment time history from ABAQUS and experimental results: a) P7; b) P9
6.5 Summary

The software ABAQUS was utilized to construct a full three-dimensional (3D) nonlinear dynamic model with the same geometric dimensions as the experimental setup. Different aspects of the developed model including 3D geometry, material models, pile-soil contact formulation, mesh properties, element type, applied ground motion, boundary conditions and analysis steps were presented. Moreover, different parameters varied through the calibration process of the numerical model were discussed and their effects were presented. This included soil stiffness and soil damping ratio, pile head connection modelling technique, and the friction coefficient of the pile-soil interface. The developed FEM model was validated by the experimental results in terms of dynamic and structural responses.

The excellent agreement observed between the numerical results from ABAQUS and those measured during the experimental program indicated the ability of the developed FEM model to capture the behaviour of pile groups under seismic loading, and particularly, to simulate experimental shaking table tests. The validated model was then employed to perform an extensive parametric study to examine the effect of some geometric parameters of helical pile groups on their responses under earthquake loading, which will be discussed in the next chapter.
Chapter 7

Numerical Modelling: Parametric Study

7.1 Introduction

The verified FEM model, incorporating the calibrated soil material model, was employed to perform an extensive parametric study covering a practical range of helical piles geometric configurations to study the effect on the dynamic response of the groups under strong earthquake shakings. Mainly, the effect of the level of a single helix, diameter of a single helix, number of helices as well as the spacing between helices was explored. Moreover, the effect of the pile diameter was also studied.

Two sets of analyses were performed. In the first set, the pile cap was raised above the ground surface considering the same value used in the experimental testing, i.e., 0.86 m. In the second set, the pile cap was modelled at the ground surface and the applied weight on the pile cap was doubled. This was done to simulate a realistic pile foundation with the pile cap resting on the ground and the supported weight is close to the pile’s design capacity.

All numerical models involved a 4-pile group installed at the center of a soil block. However, dimensions of the soil block, the piles, the helices as well as the pile caps varied in some cases to ensure minimizing the boundary effects on the calculated response. It should be noted that the (+ve) and (–ve) signs in all results indicate the direction of shaking whether it was in the (+ve) X-axis or the (–ve) X-axis. In addition, the results are displayed for one pile within the pile group, which is located at the west pile row. This means that the (+ve) shaking direction represents the pile moving towards the pile group center (soft soil) and the (–ve) shaking direction represents the pile moving away from the pile group center (stiff soil).

7.2 First Analysis Set

Nine helical pile groups’ numerical models, as well as one driven pile group model were constructed and analyzed in this set. In all models, the soil block dimensions were 8.64 m x 4.83 m x 5.84 (L x W x H). The pile cap was modelled as a cube with side length of 4.06
m and total mass of 20,000 kg. The piles had identical shaft with diameter, \( d = 0.18 \) m (7 in), wall thickness, \( t = 8.05 \) mm (0.317 in), length, \( l = 5.49 \) m, with an embedded depth of 4.63 m and a stick-out length of 0.86 m. All helices had a thickness of 12.7 mm (0.5 in); however, the number of helices and helix diameter (D) were varied. The piles were spaced at 2.03 m and were fixed to the pile cap. Table 7-1 presents the notations used for different numerical models and a summary of the helical piles configurations.

Table 7-1. Notations and pile configurations of models in first analysis set

<table>
<thead>
<tr>
<th>Model Notation</th>
<th>Diameter of Helices m (in)</th>
<th>Number of Helices</th>
<th>Depth of Helices m (D)</th>
<th>Spacing Between Helices (D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P7-1H14-5D</td>
<td>0.36 (14)</td>
<td>1</td>
<td>2.03</td>
<td>-</td>
</tr>
<tr>
<td>P7-1H16-5D</td>
<td>0.41 (16)</td>
<td>1</td>
<td>2.03 (5D)</td>
<td>-</td>
</tr>
<tr>
<td>P7-1H16-7D</td>
<td></td>
<td>1</td>
<td>2.85 (7D)</td>
<td>-</td>
</tr>
<tr>
<td>P7-1H16-10D</td>
<td></td>
<td></td>
<td>4.06 (10D)</td>
<td>-</td>
</tr>
<tr>
<td>P7-2H16-3D</td>
<td></td>
<td>2</td>
<td>2.03 (5D) / 3.25</td>
<td>3D</td>
</tr>
<tr>
<td>P7-3H16-1.5D</td>
<td>0.41 (16)</td>
<td></td>
<td>2.03 (5D) / 2.64 / 3.25</td>
<td>1.5D</td>
</tr>
<tr>
<td>P7-3H16-2D</td>
<td></td>
<td>3</td>
<td>2.03 (5D) / 2.85 / 3.66</td>
<td>2D</td>
</tr>
<tr>
<td>P7-3H16-3D</td>
<td></td>
<td></td>
<td>2.03 (5D) / 3.25 / 4.47</td>
<td>3D</td>
</tr>
<tr>
<td>P7-1H20-5D</td>
<td>0.51 (20)</td>
<td>1</td>
<td>2.03</td>
<td>-</td>
</tr>
<tr>
<td>P7-0H*</td>
<td>-</td>
<td>0</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

* Driven pile
7.2.1 Effect of Number of Helices

In order to explore the effect of the number of helices, results of models P7-0H, P7-1H16-5D, P7-2H16-3D and P7-3H16-3D are compared. The piles in these models were identical except for the number of helices, which was zero (0H) “driven piles”, one (1H), two (2H) or three (3H) for P7-0H, P7-1H16-5D, P7-2H16-3D and P7-3H16-3D, respectively. The first helix was placed at a depth of 5D from ground surface, while the spacing was 3D as mostly used in practice. A schematic of piles considered is shown in Figure 7-1.
7.2.1.1 Piles Responses

For piles in a group, two mechanisms were observed contributing to the total pile movement during shaking: flexural deflection of the individual piles and rigid rotation of the pile group. Rotation of piles generated higher passive soil resistance at deeper soil (which had higher strength and stiffness owing to higher confining pressure), while flexural deflection was mainly resisted by the upper soil layer (which was softer and weaker). Hence, pile rotation mechanism would increase soil resistance and reduce the pile flexural deflection. However, the pile’s total lateral displacement depended on the contribution of both mechanisms, which varied during shaking as the soil strength decreased and the direction of shaking changed.

Figure 7-2 presents the maximum total lateral movement of one pile within the pile group for different number of helices. As can be seen from Figure 7-2, as the number of helices increased, the total lateral displacement increased. This is because multiple helices caused fixation of the pile’s lower part, which reduced the pile rotation and promoted the flexural behaviour. On the other hand, the driven pile and the single-helix pile displayed tendency to rotate due to less fixation at the bottom part. To further elucidate this effect, the maximum values of both response components (i.e., flexural deflection and rotation) are shown in Figure 7-3.

The 1H pile displayed more rotation about or close to the helix location, compared to the 2H and 3H piles, which implied higher soil resistance along the whole depth of the pile. The rotation of the driven pile was much larger at shallower depths. As a result, the flexural deflection of the driven pile and 1H pile was the lowest, and it increased as the number of helices increased as can be seen in Figure 7-3a. The total movement of the pile, which is the summation of contributions from both mechanisms, was found to be governed by the rotation in the driven pile and 1H pile with a ratio of 1.5, almost equal for 2H pile, and governed by flexural deflection in 3H pile with a ratio of 1.2. Moreover, Figure 7-3 shows that the displacement of all piles increased when the shaking was towards the pile group center (+ve) due to the soil softening within the pile group because of the significant Pile-Soil-Pile-Interaction (PSPI), compared to soil outside the pile group.
Figure 7-2. Effect of number of helices on the maximum total lateral displacement in first analysis set

Figure 7-3. Individual components of lateral displacements for different number of helices in first analysis set: a) Flexural deflections; b) Rotations
This increase in the lateral displacement could also be attributed to the change in the natural frequency of the system. Figure 7-4 presents the normalized Frequency Response Function (FRF) of the pile cap’s response for both horizontal and rocking amplitudes. This normalization was done with respect to the free field acceleration in order to eliminate any effects of the soil block’s vibration modes. As can be seen from Figure 7-4, as the number of helices increased, both natural frequencies, horizontal and rocking, increased. This shifted the natural frequency of the system closer to a peak frequency amplitude in the input ground motion, which increased the resonance effects, and resulted in larger movement of piles. Moreover, vibration of the 1H pile was governed by the rocking mode, while an almost equal amplitude of horizontal and rocking components was observed for piles 2H and 3H. This confirmed the illustration mentioned before regarding the rotation and flexural deflection of piles.

Figure 7-5 presents the maximum developed bending moments and normal forces in piles with different number of helices. Figure 7-5 reveals that the bending moment developed in piles increased as the number of helices increased. This was attributed to the increase in the flexural deflection, which directly relates to the bending moment. Also, the maximum bending moment consistently occurred at the pile head with a change in the sign due to the fixation of pile heads at the pile cap. In addition, as the number of helices increased, the bending moment profile extended downward and the bending moment sign changed at lower depths due to the deeper fixation point. On the other hand, the normal force increased as the number of helices increased, which indicated higher axial resistance provided by the helices. This also indicates that even small rotations of the pile would develop normal forces due to the soil resisting the movement of helices. However, the same end bearing force was observed at the pile tip.

Comparing helical piles to driven piles, even the 1H pile exhibited much larger normal forces than the 0H pile, which was almost 1.5 times. This ratio significantly increased if compared to the 3H pile, which was nearly 3 times. This indicated the significant contribution of helices, or even a single helix, to the rocking resistance component of the pile group.
Figure 7-4. Normalized Frequency Response Function (FRF) of the pile group for different number of helices in first analysis set: a) Horizontal amplitudes; b) Rocking amplitudes
7.2.1.2 Pile Cap Responses

Figure 7-6 presents the horizontal acceleration, velocity, and displacement histories at the center of pile cap, while Figure 7-7 presents the vertical acceleration, velocity, and settlement time histories.

The results show a slight increase in both horizontal and vertical acceleration and velocity as the number of helices increased (2H and 3H piles), which implied larger amplification to the input motion. This corresponded to larger lateral forces and bending moments acting on the pile group. However, the lateral displacement of the pile cap increased slightly (compared to acceleration) due to the increased rocking resistance of the piles with more helices. It was observed that the settlement of the pile cap decreased as the number of helices increased. However, the difference between 2H piles and 3H piles was lower compared to the 1H and 2H pile, which indicated a less efficiency of adding a third helix as two helices already provided significant normal resistance. A significant settlement was observed for driven piles compared to helical piles.
Figure 7-6. Horizontal responses of pile cap for different number of helices in first analysis set: a) Accelerations; b) Velocities; c) Displacements
Figure 7-7. Vertical responses of pile cap for different number of helices in first analysis set: a) Accelerations; b) Velocities; c) Settlements
In addition, rotation of the pile cap decreased as the number of helices increased owing to the additional rocking resistance (and larger normal forces) provided by 2H and 3H piles. On the other hand, driven piles experienced higher rotations compared to helical piles. This is further demonstrated by the differential settlement of the pile cap presented in **Figure 7-8**.

**Figure 7-8. Differential settlements of pile cap for different number of helices in first analysis set**

### 7.2.2 Effect of Helix Level

To evaluate the effect of the elevation of the helix relative to the ground surface, results of models P7-1H16-5D, P7-1H16-7D and P7-1H16-10D are compared. These models were identical except for the elevation of the helix, which varied, i.e., 5D (P7-1H16-5D), 7D (P7-1H16-7D) and 10D (P7-1H16-10D). **Figure 7-9** presents a schematic of the three pile configurations.
Figure 7-9. Schematic of piles used to determine the effect of helix level in first analysis set: a) P7-1H16-5D (5D); b) P7-1H16-7D (7D); c) P7-1H16-10D (10D)

7.2.2.1 Piles Responses

Figure 7-10 presents the total lateral displacement of piles with different helix levels. It is observed from Figure 7-10 that the displacement increased as the helix depth increased. This was attributed to the fixation provided by the helix in resisting the lateral movement. As the helix is placed closer to the surface, the unsupported length of the pile decreases and the pile lateral stiffness increases, and the lateral displacement decreases.

However, the difference was small between the considered cases because the minimum helix depth considered was 5D (as required in practice) that corresponded to 11.4d. Given
that the pile lateral response is affected mainly by the deformation within the top 10d-15d, the beneficial effect of the helix was small. It is also noted from Figure 7-10 that the displacement was noticeably larger for all piles when they displaced towards the pile group center (+ve). All piles experienced the same displacement (the helix had no effect) as they moved into the stronger soil outside the pile group (-ve) because the effective length of the pile decreases as the soil stiffness increases, and all helix depths were below that effective length.

The flexural and rocking components of the piles lateral movement are presented in Figure 7-11. It is noted from Figure 7-11 that the flexural deflection decreased, and the rotation component increased as the helix depth decreased (i.e., near the surface).

Figure 7-10. Effect of helix level on the maximum total lateral displacement in first analysis set
Figure 7-11. Individual components of lateral displacements for different helix levels in first analysis set: a) Flexural deflections; b) Rotations

Figure 7-12 presents the maximum bending moments and normal forces for the piles with different helix levels. As can be seen from Figure 7-12, all piles had the same bending moment values; however, as the helix depth increased, the bending moment profile extended downward, and the sign of bending moment changed further down due to the fixation provided by the helix. Moreover, the same normal force values were observed for all the piles as well as the same contribution of the helices. The same observation is made for the normal force profile, which extended downward as the helix depth increased.
Figure 7-12. Internal forces in piles for different helix levels in first analysis set:
 a) Bending moments; b) Normal forces

7.2.2.2 Pile Cap Responses

Figure 7-13 presents the horizontal acceleration, velocity and displacement time histories calculated at the center of pile cap for piles with different helix depth, while Figure 7-14 presents the time histories of vertical acceleration and velocity, and settlement. Initially, as expected, larger settlement was observed as the helix depth increased. However, as the shaking progressed, the settlement of 5D and 7D started to increase over that of 10D. This is attributed to degradation of soil’s strength beneath the helix due to plastic deformation, resulting in larger movement at the helix level. The degradation of soil strength beneath the helix was more pronounced for shallower depth given the higher normal force at the helix elevation and lower confining pressure. On the other hand, the differences between accelerations, velocities, and displacements among the three helix levels were minimal. However, a slight increase in the rotation of the pile cap was observed for shallower helix depths as can be seen from Figure 7-15.
Figure 7-13. Horizontal responses of pile cap for different helix levels in first analysis set: a) Accelerations; b) Velocities; c) Displacements
Figure 7-14. Vertical responses of pile cap for different helix levels in first analysis set: a) Accelerations, b) Velocities, and c) Settlements
7.2.3 Effect of Helix Diameter

To explore the effect of the helix diameter, results of models P7-1H14-5D, P7-1H16-5D and P7-1H20-5D were compared. These models were identical except for the helix diameter, i.e., 14 in (H14), 16 in (H16) and 20 in (H20) for P7-1H14-5D, P7-1H16-5D and P7-1H20-5D, respectively. All helices were placed at a depth of 5D from ground surface. A schematic of the piles geometry is shown in Figure 7-16.
Figure 7-16. Schematic of piles used to determine the effect of helix diameter:
a) P7-1H14-5D (H14); b) P7-1H16-5D (H16); c) P7-1H20-5D (H20)

7.2.3.1 Piles Responses

**Figure 7-17** presents the total lateral displacement of piles with different helix diameters and **Figure 7-18** presents the flexural and rocking components of the displacement. As can be seen from these figures, the effect of the helix diameter was minimal and did not change as the shaking progressed. This is perhaps because the smallest diameter considered (H14) was already large enough to provide the required fixation and no further benefit could be achieved as the helix diameter increased. However, as the H20 helix provided more rigidity to the pile, a slight increase in the flexural deflection was observed, while the rotation experienced by H14 and H16 was higher.
Figure 7-17. Effect of helix diameter on the maximum total lateral displacement

Figure 7-18. Individual components of lateral displacements for different helix diameters: a) Flexural deflections; b) Rotations
Figure 7-19 presents the developed bending moments and normal forces in piles with different helix diameter. As can be seen from Figure 7-19, a slight increase in the bending moment was observed by increasing the helix diameter, which was attributed to larger flexural deflections. On the other hand, larger helices provided higher normal resistance due to a higher contribution of the helix. However, the lower part of the piles (beneath the helix) provided the same normal resistance for all helix diameters.

![Figure 7-19. Internal forces in piles for different helix diameters: a) Bending moments and b) Normal forces](image)

7.2.3.2 Pile Cap Responses

Figure 7-20 presents the time histories of the horizontal acceleration, velocity and displacement calculated at the center of cap for piles with different helix diameter and Figure 7-21 presents the time histories of vertical acceleration, velocity, and settlement. Almost the same responses were experienced by the pile cap for all cases except for the settlement, which decreased for H20, while it was almost the same for H14 and H16.
Figure 7-20. Horizontal responses of pile cap for different helix diameters:
  a) Accelerations, b) Velocities, and c) Displacements
Figure 7-21. Vertical responses of pile cap for different helix diameters:
a) Accelerations, b) Velocities, and c) Settlements
On the other hand, a very slight increase in the rotation of the pile cap was observed for H14 and H16 as can be seen from Figure 7-22, which could be attributed to the higher rotation experienced by the piles during shaking compared to H20.

Figure 7-22. Differential settlements of pile cap for different helix diameters

7.2.4 Effect of Spacing between Helices

In order to evaluate the effect of the spacing between helices, results of models P7-3H16-1.5D, P7-3H16-2D and P7-3H16-3D were compared. These models were identical except for the spacing between helices which was 1.5D (P7-3H16-1.5D), 2D (P7-3H16-2D) and 3D (P7-3H16-3D). The first helix was at a depth of 5D from the ground surface. Figure 7-23 presents a schematic of piles analyzed.
Figure 7-23. Schematic of piles used to determine the effect of spacing between helices: a) P7-3H16-1.5D (1.5D); b) P7-3H16-2D (2D); c) P7-3H16-3D (3D)

7.2.4.1 Piles Responses

The total lateral displacements for piles with different helix spacing are compared in Figure 7-24. It is noted from Figure 7-24 that piles with spacing of 1.5D exhibited slightly larger displacement. This is because the significant interaction between the helices with smaller spacing. However, 3D piles experienced higher flexural deflection component compared to 1.5D and 2D as can be seen from Figure 7-25. This is because closer helices fixed the inter-helix segment of the pile and hence the flexural behaviour became dominant.
Figure 7-24. Effect of spacing between helices on the maximum total lateral displacement

Figure 7-25. Individual components of lateral displacements for different spacing between helices: a) Flexural deflections; b) Rotations
On the other hand, Figure 7-26 shows that the bending moment increased slightly as the spacing between helices increased owing to the higher flexural deflection. In addition, the normal force increased as the inter-helical spacing increased indicating less interaction between the helices and providing more vertical resistance. However, the differences were small because the first helix was already at a depth of 11.4d, and the lower helices had minimal effects on the lateral resistance.

![Figure 7-26. Internal forces in piles for different spacing between helices; a) Bending moments; b) Normal forces](image)

7.2.4.2 Pile Cap Responses

Figure 7-27 presents the horizontal acceleration, velocity and displacement experienced at the center of pile cap for piles with different number of helices and Figure 7-28 presents the time histories of vertical acceleration and velocity and settlement. It could be observed that no significant change was observed in the pile cap response for different spacing between helices. However, the settlement decreased as the spacing between helices increased, which is attributed to higher resistance provide by the helices due to less interaction in the inter-helical zone.
Figure 7-27. Horizontal responses of pile cap for different spacing between helices:
  a) Accelerations; b) Velocities; c) Displacements
Figure 7-28. Vertical responses of pile cap for different spacing between helices:
   a) Accelerations; b) Velocities; c) Settlements
Moreover, the pile cap rotation decreased as the spacing between helices increased due to higher rotation experienced by 1.5D and 2D piles, which could be seen from Figure 7-29.

![Figure 7-29. Differential settlements of pile cap for different spacing between helices](image)

### 7.3 Second Analysis Set

A total number of 6 models were constructed in this set. In all models, the soil block dimensions were 7.82 m x 4.01 m x 5.44 m (L x W x H). In order to avoid bearing contact between the pile cap and soil, the pile cap was raised 0.15 m (6 in) above the ground surface. The pile cap was modelled as a cube block with a total mass of 40,000 kg and side length of 3.25 m. All piles had length of 4.37 m, with an embedded depth of 4.22 m, and were fixed to the pile cap. The spacing between the piles was 1.63 m (64 in), which corresponded to 4D to minimize pile-soil-pile interaction (PSPI). The helix had diameter of 0.41 m (16 in) and thickness of 12.7 mm (0.5 in). A summary of variable geometric properties as well as notations of the models is presented in Table 7-2.
Table 7-2. Notations and pile configurations of models in second analysis set

<table>
<thead>
<tr>
<th>Model Notation</th>
<th>Pile Diameter m (in)</th>
<th>Pile wall Thickness mm (in)</th>
<th>Number of Helices</th>
<th>Depth of Helices m (D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P5-5D-1</td>
<td>0.14 (5.5)</td>
<td>7.72 (0.304)</td>
<td>1</td>
<td>2.03 (5D)</td>
</tr>
<tr>
<td>P5-5D-2</td>
<td></td>
<td></td>
<td>2</td>
<td>2.03/3.25 (5D/8D)</td>
</tr>
<tr>
<td>P7-5D-1</td>
<td></td>
<td></td>
<td></td>
<td>2.03 (5D)</td>
</tr>
<tr>
<td>P7-7D-1</td>
<td>0.18 (7)</td>
<td>8.05 (0.317)</td>
<td>1</td>
<td>2.85 (7D)</td>
</tr>
<tr>
<td>P7-10D-1</td>
<td></td>
<td></td>
<td></td>
<td>4.06 (10D)</td>
</tr>
<tr>
<td>P8-5D-1</td>
<td>0.22 (8 5/8)</td>
<td>8.18 (0.322)</td>
<td>1</td>
<td>2.03 (5D)</td>
</tr>
</tbody>
</table>

7.3.1 Effect of Pile Diameter

To explore the effect of the pile diameter, results of models P5-5D-1, P7-5D-1 and P8-5D-1 were compared. These models were identical except for the pile diameter, i.e., 5.5 in (P5-5D-1), 7 in (P7-5D-1) and 8 5/8 in (P8-5D-1). Herein, they are referred to as P5, P7 and P8 for P5-5D-1, P7-5D-1 and P8-5D-1, respectively. These pile diameters were chosen as they are commonly used in practice. Figure 7-30 presents a schematic of the piles analyzed, and Table 7-3 presents a summary of the piles flexural rigidity (EI) as well as the axial rigidity (EA).
Figure 7-30. Schematic of piles used to evaluate the effect of pile diameter in second analysis set: a) P5-5D-1 (P5); b) P7-5D-1 (P7); c) P8-5D-1 (P8)

Table 7-3. Flexural and normal rigidity of different piles

<table>
<thead>
<tr>
<th>Model</th>
<th>Flexural Rigidity, EI (kN.m²)</th>
<th>Normal Rigidity, EA (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P5-5D-1</td>
<td>1399</td>
<td>640</td>
</tr>
<tr>
<td>P7-5D-1</td>
<td>3100</td>
<td>859</td>
</tr>
<tr>
<td>P8-5D-1</td>
<td>6034</td>
<td>1084</td>
</tr>
</tbody>
</table>
7.3.1.1 Piles Responses

Two main mechanisms were observed for the movement of piles during shaking: flexural deflection and rigid body rotation. **Figure 7-31** presents the maximum total lateral displacement of one pile within each pile group. The piles mainly rotated when the shaking was towards the pile group center (+ve shaking direction for this pile’s location). Due to the higher weight of the pile cap in this analysis set, the lateral inertial loads were higher, and the soil enclosed within the piles experienced strong nonlinearity due to PSPI, which resulted in softer soil compared to the soil outside the pile group. In addition, deep gaps were observed between the piles and adjacent soil, especially in the vicinity of the pile group. This promoted the rotation of the pile group as a rigid body (as opposed to flexural deflection), which can be seen from the total displacement in the (–ve) shaking direction from **Figure 7-31**.

In general, the total lateral displacement of piles decreased as the pile diameter increased. However, the two components of the displacement (flexural deflection and rotation) contributed differently as shown in **Figure 7-32**. These components are discussed below:

1- As the pile diameter increased, its flexural rigidity (EI) increased, and the flexural deflection decreased. On the other hand, as the flexural rigidity increased rigid body movement became more dominant and the associated displacement increased.

2- The maximum rotation occurred when piles moved towards the pile group center, while the maximum flexural deflection occurred when piles moved outside. This indicated that the piles rotated when they moved towards soft soil (enclosed within the piles), while the bending deflection dominated the response when piles moved towards stiff soil.
Figure 7-31. Effect of pile diameter on the maximum total lateral displacement

Figure 7-32. Individual components of lateral displacements for different pile diameters: a) Flexural deflections; b) Rotations
Figure 7-33a presents the normal forces in piles due to static load, which indicate two expected trends. The helix contribution to axial resistance was larger for smaller pile diameter, which represented higher helix to pile diameter ratio, (D/d) = 2.91, 2.29 and 1.86 for P5-5D-1, P7-5D-1 and P8-5D-1, respectively. It is also noted that the load transfer was higher as the pile diameter increased due to increased shaft friction. However, nearly the same end bearing resistance was observed for all piles, which implied that although the contribution of the helix was higher for small-diameter piles, it was compensated through larger surface area and axial rigidity of large-diameter piles.

Figure 7-33b displays the maximum normal force developed in piles during shaking (without the static load contribution discussed above). Figure 7-33b shows that the maximum normal force increased as the pile diameter increased. This is because the increase in pile diameter promoted the rocking behaviour, and correspondingly the rotation, causing an increase in the pile’s axial force. In addition, the maximum normal force occurred when piles were under compression, i.e., when the pile was moving away from the pile group center (–ve shaking direction for this pile’s location). However, when piles were moving towards the pile group center, the tensile forces developed were small and the effect of pile diameter was small.

The helix contribution was larger under compression than under tension, especially for small-diameter piles. This was attributed to the reduced contact pressure between the soil and the helix’s upper surface compared to the contact pressure at the helix’s bottom surface at the beginning of the shaking. This required large upward vertical movement in order for the helix to contribute to the tension resistance. Meanwhile, the helix contribution to compression resistance was nearly the same for all piles. This was attributed to yielding of the soil beneath the helix, which was more pronounced for smaller pile diameter. This was compensated by the additional resistance on the upper helix’s surface due to helix rotation.
Figure 7-33. Effect of pile diameter on developed normal forces: a) Static own weight; b) Maximum dynamic during shaking

Figure 7-34 displays the maximum bending moment and rotation experienced by different piles during shakings. Figure 7-34 shows that the pile rotation increased as its diameter decreased (i.e., EI decreased). However, as the pile diameter decreased, its bending moment decreased. It is also noted that the maximum bending moment consistently occurred at the pile head due to the fixed head connection, while the maximum rotation occurred at a depth of approximately 0.4 m. The maximum bending moment decreased when the pile moved towards the group center because the soil enclosed within the piles was softer than the soil outside the piles and hence offered lower resistance. In addition, rigid body rotation was more prominent when piles moved into soft soil. Finally, it is noted that the maximum bending moment occurred at almost the same location (1.0 m) for piles with different diameters when the piles moved away from the pile group center. However, the depth of the maximum bending moment increased as the pile’s diameter increased when the pile moved into the soft soil enclosed within the piles. This was mainly because rigid body rotation was dominant for large-diameter piles, and lower soil layers contributed to resisting the pile rotation.
Figure 7-34. Maximum responses in piles for different pile diameters: a) Bending moments and b) Slopes

7.3.1.2 Pile Cap Responses

Figure 7-35 presents the horizontal acceleration, velocity and displacement time histories calculated at the center of pile cap for piles with different diameter and Figure 7-36 presents the vertical acceleration, velocity, and settlement time histories. The settlement decreased noticeably as the pile diameter increased due to the significant increase in the pile’s axial rigidity. However, both horizontal and vertical acceleration and velocity increased as the pile diameter increased. This is because the natural frequencies of the pile group system (horizontal and vertical) increased and became closer to the predominant frequency of the ground shaking (resonance condition). This resulted in larger lateral forces and bending moments acting on the pile group for larger diameter piles, and consequently a slight increase in the lateral displacement of P7 and P8 compared to P5.
Figure 7-35. Horizontal responses of pile cap for different pile diameters:
a) Accelerations; b) Velocities; c) Displacements
Figure 7-36. Vertical responses of pile cap for different pile diameters:
  a) Accelerations; b) Velocities; c) Settlements
In addition, the rotation of the pile cap increased as the pile diameter increased as shown in Figure 7-37. This is because piles with higher EI tends to rotate than deflect. In addition, the rocking natural frequency of the pile group system was closer to the predominant frequency of the shaking and hence rocking behaviour was significantly evidenced by the pronounced higher peak rotation of P7 compared to P8.

Figure 7-37. Differential settlements of pile cap for different pile diameters

7.3.2 Effect of Number of Helices

Results of models P7-5D-1 and P7-5D-2 were compared to evaluate the effect of the number of helices. These two models were identical except for the number of helices, which was one helix (1H) and two helices (2H) for P7-5D-1 and P7-5D-2, respectively. Figure 7-38 shows a schematic of the piles analyzed.
7.3.2.1 Piles Responses

The same observations were made with respect to the piles responses for different number of helices as the case in the first analysis set. Increasing the number of helices increased the pile group total lateral displacement, due to higher flexural deflection as well as an increase in the pile’s stiffness due to introducing more helices, which increased the natural frequency of the system, and shifted it closer to the predominant frequency of the ground motion. Thus, resonance effects were more experienced.

Moreover, bending moments developed in piles increased as the number of helices increased due to higher flexural deflection. In addition, the bending moment profile extended downward and the bending moment sign changed at lower depths due to the
deeper fixation point. Normal forces increased as the number of helices increased, which indicated higher axial resistance provided by the helices.

7.3.2.2 Pile Cap Responses

**Figure 7-39** presents the time histories of the horizontal acceleration, velocity and displacement calculated at the center of pile cap and **Figure 7-40** presents the time histories of the vertical acceleration, velocity, and settlement. The settlement decreased significantly for 2H pile because of the additional axial resistance provided by the second helix. On the other hand, 2H pile exhibited a slight increase in horizontal and vertical accelerations and velocities due to resonance condition, which resulted in larger amplification of ground motion, and consequently larger lateral inertial forces and bending moments. However, the lateral displacement of the pile cap increased slightly owing to the significant rocking resistance provided by the 2H pile.
Figure 7-39. Horizontal responses of pile cap for different number of helices in second analysis set; a) Acceleration, b) Velocity, c) Displacement
Figure 7-40. Vertical responses of pile cap for different number of helices in second analysis set; a) Accelerations, b) Velocities, c) Settlements
Moreover, the rotation of the pile cap was higher for 1H piles as can be observed from the differential settlement presented in Figure 7-41. This could be attributed to the fixation provided by two helices compared to one helix, which acted as a rotation restraint.

![Figure 7-41. Differential settlements of pile cap for different number of helices in second analysis set](image)

7.3.3 Effect of Helix Level

To evaluate the effect of the helix level, results of models P7-5D-1 (5D), P7-7D-1 (7D) and P7-10D-1 (10D) were compared. Helices were placed at depths of 5D, 7D and 10D for P7-5D-1, P7-7D-1 and P7-10D-1, respectively. Figure 7-42 presents a schematic of the piles analyzed.
7.3.3.1 Piles Responses

Same observations were made regarding the helix level as those from the first analysis set. Lateral displacement increased as the helix depth increased, which was attributed to the fixation provided by the helix in resisting the lateral movement. The difference between different helix levels was small as the minimum depth considered was 5D (as required in practice) that corresponded to 11.4d. Given that the pile lateral response is affected mainly by the deformation within the top 10d-15d, the beneficial effect of the helix was small. In
addition, increasing the elevation of the helix increased the rotation of the pile, while reducing the flexural deflection. All helix levels yielded the same bending moments as well as normal forces indicating a minimal effect of the helices on the lateral resistance of piles.

7.3.3.2 Pile Cap Responses

Figure 7-43 presents the time histories of horizontal acceleration, velocity and displacement calculated at the center of pile cap for piles with different number of helices and Figure 7-44 presents the time histories of vertical acceleration, velocity, and settlement. The settlement decreased as the helix elevation increased, due to the shorter unsupported length of the pile above the helix (point of fixation). Interestingly, as the shaking progressed, the settlement of P5 and P7 started to increase more than that of P10. This could be attributed to two reasons: first, the axial resistance is governed by the contribution of the helix compared to the pile shaft friction beneath it, i.e., lower segment of the 10D case provided higher normal resistance. Second, as the shaking progressed, degradation of the soil’s strength beneath the helix was pronounced, which reduced the helix contribution to axial resistance while the shaft resistance was not affected as much. As a result, the final settlement of 10D case was lower than 5D and 7D. The same behaviour was observed in the first analysis set for different helix levels.

On the other hand, the horizontal and vertical accelerations and velocities increased as the helix elevation increased. This is primarily related to the closeness of the pile group-soil system natural frequencies (horizontal and vertical) to the pre-dominant frequency of the ground motion. As the helix elevation increased, the natural frequencies increased approaching resonance condition. Thus, larger inertial lateral forces and bending moments occurred for piles 5D and 7D and their lateral displacement became slightly larger than 10D.

On the other hand, rotation of the pile cap was higher for deeper helix levels as can be seen from Figure 7-45. This is attributed to the increase in the lateral unsupported length of the pile by increasing the depth of the helix, which resulted in higher rotations.
Figure 7-43. Horizontal responses of pile cap for different helix levels in second analysis set; a) Accelerations, b) Velocities, c) Displacements
Figure 7-44. Vertical responses of pile cap for different helix levels in second analysis set: a) Accelerations; b) Velocities; c) Settlements
Summary and Conclusions

An extensive parametric study was performed employing the fully calibrated nonlinear FEM model. Two sets of analysis were conducted considering the location and mass of the pile cap: one similar to the experimental test setup and one to replicate practical loading conditions more realistically. Different geometric parameters of helical piles were varied, and their effects on the performance of the groups were explored. Based on the results, some conclusions can be drawn as follows:

Effect of number of helices

1- Increasing the number of helices increased the pile group’s total lateral displacement, due to higher flexural deflection as well as an increase in the pile’s stiffness due to introducing more helices shifted the natural frequency of the pile-soil system closer to the predominant frequency of the ground motion. Thus, more resonance effects were experienced.
2- Bending moments developed in piles increased as the number of helices increased due to higher flexural deflection. In addition, the bending moment profile extended downward and the bending moment sign changed at lower depths due to the deeper fixation point.

3- Normal force increased as the number of helices increased, which indicated higher axial resistance provided by the helices. This difference was significant between driven piles and helical piles, even for single helix.

4- Settlement of the pile cap decreased as the number of helices increased. However, the difference between two and three helices was very low, which indicated a less efficiency of adding a third helix as two helices already provided significant normal resistance. On the contrary, a significant increase in the settlement was observed for the case of driven piles.

5- A slight increase in both horizontal and vertical acceleration and velocity was observed as the number of helices increased, which implied larger amplification to the input motion.

6- Rotation of the pile cap decreased as the number of helices increased owing to the additional rocking resistance (and larger normal forces) provided by two or three helices.

**Effect of helix level**

1- Lateral displacement increased as the helix depth increased, which was attributed to the fixation provided by the helix in resisting the lateral movement.

2- The difference between different helix levels was small as the minimum depth considered was 5D (as required in practice) that corresponded to 11.4d. Given that the pile lateral response is affected mainly by the deformation within the top 10d-15d, the beneficial effect of the helix was small.

3- Increasing the elevation of the helix increased the rotation of the pile, while reducing the flexural deflection.
4- All helix levels yielded the same bending moment values; however, as the helix depth increased, the bending moment profile extended downward, and the sign of bending moment changed further down due to the fixation provided by the helix.

5- The same normal force values were observed for all the helix levels as well as the same contribution of the helices.

6- Differences between accelerations, velocities and displacements of the pile cap among the three helix-levels were minimal. However, a slight increase in the rotation of the pile cap was observed for shallower helix depths.

7- Larger settlement was observed initially as the helix depth increased. However, as the shaking progressed, the settlement of shallower-helix piles started to increase, as a result of degradation of soil’s strength beneath the helix.

8- A slight increase in the rotation of the pile cap was observed for shallower helix depths as a result of higher rotation of piles.

**Effect of helix diameter**

1- The effect of the helix diameter was minimal as the smallest diameter considered (14 in) was already large enough to provide the required fixation.

2- Increasing the diameter of helix slightly increased the flexural deflection and decreased the rotation of pile.

3- A slight increase in the bending moment was observed by increasing the helix diameter, which was attributed to larger flexural deflections.

4- Larger helices provided higher normal resistance due to the higher contribution of the helix. However, the lower part of the piles (beneath the helix) provided the same normal resistance for all helix diameters.
5- Almost the same responses were experienced by the pile cap for all cases except for the settlement, which decreased for (20 in) helix, while it was almost the same for (14 in) and (16 in) helices.

6- A slight increase in the rotation of the pile cap was observed for (14 in) and (16 in), which could be attributed to the higher rotation experienced by the piles.

Effect of spacing between helices

1- Reducing the spacing between helices increased the lateral displacement of the pile group owing to the significant interaction in the inter-helix-zone.

2- Piles with larger helices spacing exhibited higher flexural deflections due to the higher resistance provided, and hence the flexural behaviour became dominant. However, rotation was significantly smaller for larger helix-spacing piles.

3- Bending moments increased slightly as the spacing between the helices increased owing to the higher flexural deflection.

4- Normal forces increased as the helix spacing increased indicating less interaction between the helices. However, the differences were small because the first helix was already at a depth of 11.4d, and the lower helices had minimal effects on the lateral resistance.

5- Settlement of the pile cap decreased as the spacing between the helices increased, which is attributed to higher resistance provide by the helices due to less interaction in the inter-helical zone.

6- Accelerations and velocities as well as lateral displacement of the pile cap were almost the same for all helix spacing.

7- Pile cap rotation decreased as the spacing between helices increased due to less rotation experienced by piles.
Effect of pile diameter

1- As the pile diameter increased, the flexural deflection significantly decreased while the rigid rotation of piles increased. The total lateral displacement was smaller for larger piles.

2- Larger bending moments and normal forces were observed by increasing the pile diameter owing to significantly higher flexural and normal rigidity.

3- Settlement of the pile cap decreased significantly by increasing the pile diameter. However, a slight increase in the accelerations and velocities were observed for larger piles.
Summary, Conclusions and Recommendations

8.1 Summary

Full-scale shaking table testing was conducted to evaluate the seismic performance of single and grouped helical piles. Eight circular and one square helical piles with different properties including length, radius and number of helices, as well as one driven circular pile were installed in dry sand enclosed in a laminar soil shear box that was situated on the shaking table. Dynamic properties of sand bed and its natural frequencies as well as natural frequencies of single and grouped helical pile-soil systems were evaluated from the collected data during different shaking events. The effects of different pile configurations as well as successive shakings on the sand bed and pile-soil systems characteristics were also investigated. In addition, responses of single and grouped helical piles were computed analytically using the software DYNA6 in which a good agreement was achieved by correctly accounting for degradation of soil stiffness and gap opening.

Furthermore, the effects of the earthquake characteristics (i.e., intensity and frequency content) on seismic performance of the single and grouped helical piles were evaluated from the measured responses. Moreover, the performance characteristics of helical pile groups were discussed in terms of the interaction between piles within a group and the contributions of vertical and lateral stiffness of individual piles to the rocking stiffness and the overall capacity of the pile group. Furthermore, the effect of pile head connection to the pile cap (fixed or pinned) on the pile group response was evaluated and the responses of fixed pile groups (2-bolts connection) versus pinned pile groups (1-bolt connection) were compared. Finally, the behaviour of a single pile and a pile within a group was compared in terms of their normalized responses.

A full 3D nonlinear dynamic FEM numerical model was constructed employing the software ABAQUS to simulate the shake table testing. The numerical model was verified with the experimental results, which was then used to perform an extensive parametric
study. The parametric study explored the effect of the level of a single helix, diameter of a single helix, number of helices, spacing between helices as well as the effect of the pile diameter on the response of helical pile groups due to strong earthquake shaking.

8.2 Conclusions

Based on the results of this study, the main conclusions can be drawn as follows:

Conclusions from experimental results

1- The shear wave velocity of the sand bed was not affected by successive shakings. An average value of 100 m/s was assumed and used in further analyses.

2- Shear strain in the sand increased at the beginning of TD3 and TD4 due to the introduction of pile head masses and pile caps, which resulted in larger responses. The shear strain increased accordingly when resonance was experienced, i.e., during the TAK earthquake testing on TD3, and the NOR earthquake testing on TD4 and TD5. No difference was noticed between fixed and pinned pile groups.

3- Damping ratio of the sand slightly increased from TD1 to TD2 after installing the piles, even with no pile head masses. Moreover, a 10 % more increase was pronounced on TD3 after introducing pile head masses. Furthermore, a substantial increase of about 30 % was observed on TD4 and TD5 due to the huge weight of steel skids added to pile groups. In addition, the damping ratio followed the same behaviour as the shear strain, in which it increased when resonance occurred due to significant responses.

4- The stiffness and natural frequency of the soil bed increased due to the pile installation and corresponding SSI. This was more pronounced in the case of the pile groups. However, due to large response during TD4 testing of the pile groups, soil deformations increased causing degradation of soil stiffness and gap formation along the upper portion of the piles, which reduced the natural frequency of the sand bed the following day (TD5).
5- The number of helices increased soil disturbance during pile installation, which resulted in reduced pile stiffness and natural frequency. On the other hand, as expected, increasing pile embedment depth and/or flexural stiffness increased its natural frequency.

6- Natural frequency of single piles and pile groups decreased due to successive shakings. This is attributed to the degradation in the pile-soil stiffness and opening of deeper gaps. However, in some cases the gap depth decreased due to sand caving-in after some additional shaking, which resulted in increases of stiffness and natural frequency.

7- The DYNA6 software predicted the single and grouped piles behaviour by accounting for degradation of soil stiffness and gap opening. The gap depth ranged from (15 cm to 60 cm) for 88 mm single piles, (60 cm to 180 cm) for 140 mm single piles, (15 cm to 30 cm) for PG1 and (45 cm to 75 cm) for PG2. It is evident that the gap depth increased as the pile diameter (and flexural stiffness) increased.

8- The seismic responses of single and grouped helical piles are greatly affected by the resonance condition (i.e., closeness of the natural frequency to the earthquake predominant frequency). This leads to increased spectral accelerations causing larger deformations and decrease in the stiffness of the pile-soil system.

9- The observed behaviour of pile groups with a single-bolt connection indicated that the assumption of pin connection might not hold true in most cases, which can result in serious underestimation of the pile group stiffness and hence can result in erroneous prediction of its response to seismic loading.

10- Resonance condition significantly increased responses of single piles and pile groups to almost 200% even for the same PGA. Hence, the earthquake frequency content and potential for resonance must be considered in seismic design.

11- As expected, the responses of single piles and pile groups, including rotations and deflections increased as the earthquake intensity increased. However, rotations and deflections of piles within a group increased linearly with a lower rate than the
earthquake intensity, while rotations and deflections of single piles increased exponentially with earthquake intensity due to gap formations and rapid loss of soil stiffness around the top portion of piles.

12- Straining actions of single piles and pile groups, including bending moments and shear forces increased linearly with earthquake intensity. However, the rate of increase of straining actions for single piles was much higher due to the degradation in soil-pile strength.

13- Depth of point of maximum bending moment increased as earthquake intensity increased for both single piles and pile groups when resonance occurred. For single piles, the depth of point of maximum shear force increased as the earthquake intensity increased, for pile groups, maximum shear force was constantly at the pile head.

14- Maximum normal force developed in piles within a pile group increased in a linear fashion as earthquake intensity increased. However, the rate of increase in the normal force greatly increased when resonance occurred as rocking resistance represented the main contribution to the total group resistance.

15- Depth of point of maximum normal force increased as the earthquake intensity increased due to negative skin friction only when piles were moving away from the pile group center. When the shaking was in the other direction, maximum normal force occurred at the ground surface and decreased with depth towards the pile tip.

16- Dynamic soil resistance for single piles increased in a nonlinear fashion as the earthquake intensity increased, while it increased linearly for pile groups. In addition, the resonance condition greatly increased the response and promoted more nonlinear behaviour in single piles.

17- Larger loops were observed in the derived dynamic p-y curves for larger earthquake intensities indicating higher soil nonlinearity as well as higher damping. Lower intensities displayed a linear behaviour. In addition, degradation experienced in the soil stiffness increased by increasing the loading intensity. The same observation was made for p-y curves of pile groups; however, larger loops were pronounced even for low intensities. This indicated that pile groups could provide higher damping compared to single piles.
18- The 2-bolt connection and 1-bolt connection provided partial fixation, and resembled neither a fixed connection nor a pin connection. The behaviour of pile groups with either connection type was closer to fixed condition rather than pinned condition, and their responses to seismic loading were close.

19- Maximum responses within a pile group changed from one pile to the opposite one when shaking reversed due to Pile-Soil-Pile Interaction and shadowing effects. Loads carried by individual piles differs by about 25 % for PG1 and 30 % for PG2, while bending moments varied by about 20 % for PG1 and 50 % for PG2. It was found that the main factor was the Spacing to Diameter (S/D) ratio.

20- Maximum bending moment as well as shear force occurred in piles within a pile group when they were moving away from the softened soil zone, and in the vicinity of the pile group. On the contrary, maximum rotations, deflections, and soil resistances were observed when piles were moving towards the pile cap center despite the lower load carried individual piles.

21- Rocking stiffness of the piles within a group due to normal forces and the flexural stiffness of the individual piles were complementary to each other. The rocking stiffness due to axial loads increased as the flexural capacity of individual piles decreased, and vice versa. Thus, unlike bending moment, the piles maximum normal force occurred when the piles were moving towards the softened soil zone.

22- Single piles normalized responses were significantly higher than that for piles within a group. This is due to the significant contribution of the rocking stiffness to the overall capacity of the pile group compared to the flexural stiffness of the individual piles (almost 80 %). This highlights the advantage of helical pile groups in resisting seismic loading.
Conclusions from numerical modelling

Effect of number of helices

1- Increasing the number of helices increased the pile group’s total lateral displacement, due to higher flexural deflection as well as an increase in the pile’s stiffness due to introducing more helices shifted the natural frequency of the pile-soil system closer to the predominant frequency of the ground motion. Thus, more resonance effects were experienced.

2- Bending moments developed in piles increased as the number of helices increased due to higher flexural deflection. In addition, the bending moment profile extended downward and the bending moment sign changed at lower depths due to the deeper fixation point.

3- Normal force increased as the number of helices increased, which indicated higher axial resistance provided by the helices. This difference was significant between driven piles and helical piles, even for single helix.

4- Settlement of the pile cap decreased as the number of helices increased. However, the difference between two and three helices was very low, which indicated a less efficiency of adding a third helix as two helices already provided significant normal resistance. On the contrary, a significant increase in the settlement was observed for the case of driven piles.

5- A slight increase in both horizontal and vertical acceleration and velocity was observed as the number of helices increased, which implied larger amplification to the input motion.

6- Rotation of the pile cap decreased as the number of helices increased owing to the additional rocking resistance (and larger normal forces) provided by two or three helices.
Effect of helix level

1- Lateral displacement increased as the helix depth increased, which was attributed to the fixation provided by the helix in resisting the lateral movement.

2- The difference between different helix levels was small as the minimum depth considered was 5D (as required in practice) that corresponded to 11.4d. Given that the pile lateral response is affected mainly by the deformation within the top 10d-15d, the beneficial effect of the helix was small.

3- Increasing the elevation of the helix increased the rotation of the pile, while reducing the flexural deflection.

4- All helix levels yielded the same bending moment values; however, as the helix depth increased, the bending moment profile extended downward, and the sign of bending moment changed further down due to the fixation provided by the helix.

5- The same normal force values were observed for all the helix levels as well as the same contribution of the helices.

6- Differences between accelerations, velocities and displacements of the pile cap among the three helix-levels were minimal. However, a slight increase in the rotation of the pile cap was observed for shallower helix depths.

7- Larger settlement was observed initially as the helix depth increased. However, as the shaking progressed, the settlement of shallower-helix piles started to increase, as a result of degradation of soil’ strength beneath the helix.

8- A slight increase in the rotation of the pile cap was observed for shallower helix depths as a result of higher rotation of piles.

Effect of helix diameter

1- The effect of the helix diameter was minimal as the smallest diameter considered (14 in) was already large enough to provide the required fixation.
2- Increasing the diameter of helix slightly increased the flexural deflection and decreased the rotation of pile.

3- A slight increase in the bending moment was observed by increasing the helix diameter, which was attributed to larger flexural deflections.

4- Larger helices provided higher normal resistance due to the higher contribution of the helix. However, the lower part of the piles (beneath the helix) provided the same normal resistance for all helix diameters.

5- Almost the same responses were experienced by the pile cap for all cases except for the settlement, which decreased for (20 in) helix, while it was almost the same for (14 in) and (16 in) helices.

6- A slight increase in the rotation of the pile cap was observed for (14 in) and (16 in), which could be attributed to the higher rotation experienced by the piles.

**Effect of spacing between helices**

1- Reducing the spacing between helices increased the lateral displacement of the pile group owing to the significant interaction in the inter-helix-zone.

2- Piles with larger helices spacing exhibited higher flexural deflections due to the higher resistance provided, and hence the flexural behaviour became dominant. However, rotation was significantly smaller for larger helix-spacing piles.

3- Bending moments increased slightly as the spacing between the helices increased owing to the higher flexural deflection.

4- Normal forces increased as the helix spacing increased indicating less interaction between the helices. However, the differences were small because the first helix was already at a depth of 11.4d, and the lower helices had minimal effects on the lateral resistance.
5- Settlement of the pile cap decreased as the spacing between the helices increased, which is attributed to higher resistance provided by the helices due to less interaction in the inter-helical zone.

6- Accelerations and velocities as well as lateral displacement of the pile cap were almost the same for all helix spacing.

7- Pile cap rotation decreased as the spacing between helices increased due to less rotation experienced by piles.

Effect of pile diameter

1- As the pile diameter increased, the flexural deflection significantly decreased while the rigid rotation of piles increased. The total lateral displacement was smaller for larger piles.

2- Larger bending moments and normal forces were observed by increasing the pile diameter owing to significantly higher flexural and normal rigidity.

3- Settlement of the pile cap decreased significantly by increasing the pile diameter. However, a slight increase in the accelerations and velocities were observed for larger piles.

8.3 Recommendations for Future Work

This study have demonstrated the superior ability of helical pile groups to withstand earthquake loadings through the significant contribution of helices to the rocking resistance of the pile group. However, there are still gaps that need to be addressed in future work such as:

1- Explore the seismic behaviour of single and grouped helical piles in saturated sand by means of full-scale testing and numerical modelling. This can demonstrate the ability of helical piles to withstand liquefaction during seismic events.
2- Study the seismic performance of helical piles in different types of soils such as clays and silts, as well as in layered soil profiles.

3- Perform full-scale shake table tests and/or numerical parametric study by employing a variety of earthquake records with wider range of frequencies.

4- Investigate the influence of shallower helices through full-scale experimental testing, as they are expected to yield better lateral performance.

5- Study the performance of large-diameter helical piles under seismic loading, and compare the behaviour to small-diameter helical piles.

6- Explore the effect of different piles and/or helix configurations as well as different cross-sections.

7- Construct FEM models employing an advanced material model for the soil. This could be used to study the load transfer mechanisms and load sharing between the pile shaft and the helices during seismic events.
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