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Nonlinear seismic behavior of pile groups in cement-improved soft clay

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ABSTRACT

Centrifuge tests were performed to investigate the effects of ground improvement on the seismic behavior of pile groups in soft clay. The soil profile consisted of four lightly overconsolidated clay layers overlying a dense layer of sand. The pile groups had a symmetrical layout consisting of 2×2 piles spaced at 3.0 pile diameters and were driven into both unimproved soft clay and soft clay improved by a simulated Cement Deep Soil Mixing (CDSM) method. The centrifuge model was subjected to seven different earthquake events with peak accelerations ranging from 0.03 to 0.66*g*. The foundation level motions of the improved pile groups were different than the surface free-field motion. The foundation level motion for the unimproved pile group was, however, identical to that in the free-field. Higher peak accelerations were observed in the pile cap of the group with smaller CDSM block (GIS) compared to the unimproved pile group (GU) and the group with the largest CDSM block (GIL). Higher pile cap to the soil surface spectral ratios were also obtained for the GIS group in both short and long periods. Cement-Deep-Soil-Mixing was effective in reducing the peak displacements of the GIL pile cap. The peak displacements of the GIS pile cap remained about the same as the GU pile cap. As the size of the ground improvement increased, the fundamental period of the pile groups reduced. The estimated fundamental periods of the GIS and GU pile groups were, however, close to each other. Acceleration and displacement response spectra of the foundation level motions in comparison to the fundamental periods of the pile groups provided insight into the observed acceleration and displacement responses. The adhesion between soft clay and CDSM blocks helped to reduce the soft clay settlement in the vicinity of CDSM blocks compared to the free-field and the vicinity of unimproved pile group. More residual excess pore water pressure was, however, generated in the vicinity of CDSM blocks compared to the free-field and the corresponding location in the unimproved pile group, likely due to vibrations of the CDSM blocks and the piles.

1. Introduction

During past earthquakes, cases of poor performance of pile foundations in weak soils (e.g., soft clays and liquefiable sands) as a result of low lateral resistance have been observed [1,2]. Increasing the lateral resistance, to decrease the lateral deflections, is therefore one of the main objectives in the design of pile foundations. It is relatively easy to restrict the lateral deflections of pile foundations in competent soils. In case of weak soils (e.g. soft clay and liquefiable sands), however, large lateral deflections are mitigated by using an increased number of more ductile, larger diameter piles that are expensive to construct. An innovative, more cost-efficient solution to this problem is to improve the soil surrounding the pile foundation [3,4]. Improving the soft clay surrounding pile foundations is of particular interest to geotechnical engineers considering the fact that these soils are quite prevalent in many parts of the world and piles in soft clays often exhibit low lateral resistance. Studies on the behavior of pile groups in improved and unimproved soft clays are, however, very limited. Due to the dearth of experimental data and lack of thorough understanding of their behavior, engineers design pile groups in improved soft clay in a conservative manner to mitigate the uncertainties. Well-documented case histories and physical model tests on the seismic behavior of pile foundations in soft clay are also rare [5,6]. There are only a few studies documenting the seismic performance of pile foundations in improved ground [2,3,7,8].

The main components to consider in analyzing the seismic behavior of pile groups are: 1) kinematic interaction between soil and piles; 2) inertial forces imposed by the superstructure; 3) pile-soil-pile interaction; and 4) nonlinear coupled soil (solid skeleton and pore water) response as a result of strong ground motions. These components constitute a complex pile group behavior known as Soil-Pile-Structure Interaction (SPSI). SPSI can be studied under the following main categories: analytical and semi-analytical, numerical, and experimental methods. A detailed literature review of this subject is beyond the scope

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of this paper and interested readers are referred to Finn [9], Gazetas and Mylonakis [10], Novak [11], and Pender [12]. A brief review of literature is provided below.

Analytical or semi-analytical solutions are based on mechanics and give an insight into the physical mechanisms involved in SPSI. However, because of the considerable difficulties that arise in the description of the physics of the seismic SPSI, simplifying assumptions are made to obtain closed-form expressions [13–15]. Different numerical methods such as Finite Difference Method (FDM) [16], Finite Element Method (FEM) [17–27], Boundary Element Method (BEM) [28,29], and a combination of FEM and BEM [30,31] have also been used to study SPSI. These models generally consider both the soil volume and the structures in the same model based on continuum mechanics principles and analyze them in a single step; which is the so called direct method [32]. However, the three-dimensional nature of the problem and nonlinear soil behavior makes these techniques computationally intensive. If linear behavior is assumed, a simpler substructure procedure can also be adopted [32]. In this method, pile foundation impedances are determined by performing soil-pile interaction analyses considering group interaction effects [33]. The superstructure model is then analyzed using these impedances and input motions obtained from free-field. It is generally assumed that these motions are not affected by the pile foundation itself, which may not always be a reasonable assumption as shown later in this paper and reported in Muraleetharan, Wei and Mish [24]. The Beam on Nonlinear Winkler Foundation (BNWF) model [22,34–37] is a useful tool capable of analyzing the soil–pile interaction with less computational effort than rigorous continuum models; provided the impedance functions can be accurately determined and it is assumed that the free-field motion is not affected by the pile foundation. In order to obtain reliable results from numerical models, they should be calibrated and validated by means of experimental results [38,39].

Experimental study of SPSI can be done by conducting field or model tests. Field tests have the advantage of closely modeling the in-situ conditions. A few dynamic loading tests have been performed on pile foundations with loading applied at the pile head [6,40–43]. There are also studies on the behavior of pile foundations under blast induced liquefaction [8,44]. The main drawback of these tests is that dynamic loading at the top produces disturbances only in the soil adjacent to the pile. Therefore, the pore water pressures will only be generated closer to the piles and dissipate faster than the case for seismic base (bedrock) shaking. A similar discussion is also applicable to the stiffness and strength degradation of the soil. Furthermore, there will be much less kinematic interactions between soil and piles in pile head load tests. Shaking table tests have been widely used to model SPSI [45–47]. Large scale shaking table tests have the advantage of modeling SPSI with dimensions equal or comparable to the prototype scale [48–50]. However, it is difficult to account for high gravitational stresses associated with deep soil profiles in shaking table tests and constructing large models can be time consuming. Most shaking table tests have studied dynamic SPSI in sand, especially during liquefaction or under lateral spreading [48,49,51]. Due to difficulties associated with preparing large volumes of clay layers, there are only a few dynamic shaking table tests for pile foundations in clay [52]. Geotechnical centrifuge tests have been used to investigate the complex seismic SPSI. Compared to field experiments, soil profiles can be well defined in centrifuge tests and the volume of soil involved is much smaller than those required for shake table tests. In centrifuge tests, the stress condition at any point of the model and therefore the overall model behavior (e.g. acceleration, displacement, and failure mechanisms) is similar to that in the full-scale prototype. The main drawback of centrifuge tests is the fact that because different scaling laws apply to different phenomena (e.g. dynamics and consolidation), similitude may not be provided simultaneously between all parameters. Furthermore, boundary effects are usually present due to the model containers used in the tests [53,54]. Like shaking table tests, most studies in geotechnical centrifuges have dealt with the behavior of pile foundations in sand, especially during liquefaction or lateral speading [55–59]. Preparing and consolidating clay and the long time often required to reach the desired degree of consolidation are some of the difficulties in centrifuge modeling of pile foundations in clay. Geotechnical centrifuge models studying SPSI in soft clay are rare and there are only limited results and observations [5,60,61].

The results of a series of dynamic centrifuge tests performed on pile groups in improved and unimproved soft clay are presented in this paper. A simulated Cement-Deep-Soil-Mixing (CDSM) method was used to improve the soft clay. The centrifuge model consisted of three pile groups and three single piles. One pile group and a single pile were installed in unimproved soil. All the other pile groups and single piles were installed in improved ground with different improvement dimensions. Tests on single piles were performed for comparison and verification purposes only. Details about the experimental set-up including instrumentation, container boundary effects, and sequential earthquake motions applied to the base of the model are presented in this paper. The transient acceleration and displacement responses of the structural models, their settlement, and their effect on the dissipation of excess pore water pressure (EPWP) are presented and discussed in detail. The series of centrifuge tests performed in this study, to the authors' knowledge, represent the first attempt to characterize nonlinear seismic SPSI effects of pile groups in improved and unimproved soft clay.

2. Centrifuge tests

Model tests were carried out at a centrifuge acceleration of 30*g* in a flexible shear beam container [53] using the 9-m radius centrifuge at the Network for Earthquake Engineering Simulation (NEES) facility at the University of California, Davis (NEES@UC Davis). The container had internal dimensions of 1722 mm (length) × 686 mm (width) × 700 mm (height). Conversion of the model scale parameters and recorded test data from the model to prototype scale was done using the scaling laws as described by Schofield [62]. From this point onwards, all the results are presented in prototype scale unless otherwise stated.

2.1. Test set-up

Six different pile foundations including three pile groups and three single piles were tested. A laboratory equivalent of Cement Deep Soil Mixing (CDSM) was used to improve the soft clay in the centrifuge tests. Photos of the fully constructed and instrumented model on the centrifuge arm are shown in Fig. 1. The details of the model including the locations of the instruments are given in Fig. 2. All the instruments referenced in this paper are also labeled in Fig. 2. The dimensions of CDSM blocks with respect to the outside diameter of a single pile (D) are presented in Table 1. Because the soil within the upper five to ten pile diameters dictate the lateral load response [63,64], it was decided to improve soft clay in all pile foundations to a depth of nine pile diameters. GIL and GIS denote the Large and Small Improved pile Groups, respectively. SIL and SIS similarly represent the Large and Small Improved Single piles, respectively. GU and SU represent the pile group and the single pile in unimproved soil, respectively.

The soil profile consisted of four clay layers with a total depth of 9.6 m, overlaying an 8.1 m dense sand layer (Fig. 2). The clay used in this test is a Kaolin/fine sand mix with equal amounts (by weight) of Kaolin and fine sand. Generally, the clay layers were lightly overconsolidated (OCR ≈ 1.1–2) except for the top layer with OCR varying from about 1.1–10 near the ground surface. The undrained shear strength profile of unimproved clay was estimated based on the assumption of normalized behavior implied by the following function [65,66]: