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Investigation of liquefaction-induced lateral load on pile group behind quay wall

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ABSTRACT

This paper presents the results of a shake-table test on a 2 × 2 pile group behind a quay wall. The main objective is to study the behavior of the pile group under the liquefaction-induced lateral spreading and the liquefied soil pressure exerted on the individual pile in the pile group. The test results are presented and discussed. Significant pile group effect is observed through a comparison of the monotonic bending moments of the individual pile in the pile group. In this regard, a simple finite element model is developed to evaluate the liquefied soil pressure on the individual pile in the pile group, in which, both the uniform and triangular soil pressures are calibrated based on the tested monotonic bending moments of the piles. The liquefied soil pressure on the pile near the quay wall is about twice as much as that on the pile far from the quay wall. Next, the liquefied soil pressure on the individual pile in the pile group is compared to that obtained from the shake-table test on single pile. Further, a parametric study is conducted to investigate the effect of the pile rotational stiffness and the pile diameter on the pile group response. Finally, the concluding remarks are drawn based upon the presented results.

1. Introduction

Numerous case histories of the pile foundation damage or failure caused by liquefaction-induced lateral spreading have been reported in major earthquakes, such as 1964 Niigata earthquake [1], 1989 Loma Prieta earthquake [2], 1995 Kobe earthquake [3], 1999 Chi-Chi earthquake [4], 2001 Arequipa earthquake [5], and 2011 Christchurch earthquake [6]. Documentation and analysis of these case histories have highlighted the importance of the kinematic soil-pile interaction, i.e., the lateral load on the pile foundations caused by the lateral spreading of liquefied soil [7,8]. Therefore, a proper consideration of the kinematic effect is one of the most important aspect of pile design, however, this effect has not been fully understood in current study, which is either ignored or crudely approximated in the pile design code.

To investigate the soil-pile kinematic interaction, the pile behind water front structures was studied using laboratory tests and field investigations. With the results of centrifuge tests on a 2 × 3 pile group behind a wall in the lateral spreading ground, Sato [9] suggested that piles near the quay wall suffered more severe damage than those far from the quay wall. Sato and Tabata [10] studied soil liquefaction and the lateral spreading of saturated sand behind a sheet-pile wall using a large-scale shake-table test, and they found that the excess pore water pressure increased after a few cycles of shaking, and the loss of effective stress could further lead to the lateral spreading of liquefied sand. Motamed et al. [11] conducted shake-table tests on a 3 × 3 pile group behind a sheet-pile quay wall, and they concluded that the lateral soil pressure on the pile foundations induced by the lateral spreading was dependent upon the position of the individual pile in the group. Motamed et al. [12,13] performed large-scale shake-table tests on a 2 × 3 pile group behind a sheet-pile quay wall, the test results indicated that the piles close to the quay wall experienced larger lateral forces than the piles far from the quay wall. Ashford et al. [14] conducted a full-scale field test in which the controlled blasting was adopted to induce liquefaction and liquefaction-induced lateral spreading, and assessed the behavior of single pile and pile group subjected to lateral spreading, the results showed that the bending moment developed in the pile was mainly caused by the lateral movement of the soft clay layer.

Previous studies on the behavior of the single pile or pile group located behind waterfront structures were often conducted in a qualitative manner; whereas, the quantitative study on the liquefied soil pressure on the pile caused by the lateral soil movement was limited. Dobry et al. [15] conducted six centrifuge model tests on the single pile foundations and calibrated two limit equilibrium methods, it was suggested that the liquefied soil pressure on pile was about 10.3 kPa. Japan Road Association (JRA) [16] and Japan Sewage Works Association (JSWA) [17] guidelines suggested a triangular pattern for the liquefied

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soil pressure, and lateral load on pile was expressed as a function of the sand density, pile length and pile diameter. With the shake-table test results, He et al. [18] advocated that the liquefied soil pressure on the pile approximately equaled to the total overburden stress. Haigh et al. [19] and Haigh and Madabhushi [20] suggested a uniform liquefied soil pressure of 16 kPa on the pile in liquefiable layer. Similarly, Gonzalez et al. [21] recommended a uniform pressure of 10 kPa. Tang et al. [22] suggested a uniform pressure of 19.5 kPa based on the measured data of a single pile shake-table test.

Although liquefied soil pressure induced by lateral spreading were suggested based on tests, the soil pressures suggested by different scholars could be significantly inconsistent [15,18–22]; thus, it is challenging to determine which pressure should be adopted in the practical design of pile foundations [23–28]. In addition, most of the existing lateral soil pressure were developed based upon the test results of the single pile, which are not able to be applied to the scenario in which the pile group is adopted [18–21]. In such a circumstance, JRA [16] and JSWA [17] proposed the liquefied soil pressure on the pile groups, in which the pressures on the individual pile in the pile group were assumed to be the same. The outcome of this assumption is that the bending moments of the individual pile in the pile group would be the same, however, this inference could not agree with the test results of the pile group [12,13]. Thus, one of the most important aspect of this study is to discuss and overcome this limitation, and give recommendation of liquefied soil pressure on the individual pile in the pile group.

In the following sections, a shake-table test on a 2 × 2 pile group behind a quay wall subjected to the lateral spreading is conducted, and the results are presented and analyzed first. On this basis, a finite element (FE) analysis is conducted to evaluate the liquefied soil pressure on the individual pile in the pile group. The liquefied soil pressures suggested by JRA and JSWA are then compared to that obtained from the FE analysis and the test results. Next, a comparison between the liquefied soil pressure on the individual pile in the group and that on single pile obtained from the single-pile test [29] is conducted. Further, a parametric study is conducted to investigate the influence of the pile diameter and the pile rotational stiffness on the behavior of the pile group. Finally, the concluding remarks are drawn based upon the presented results.

2. Description of shake-table test

A shake-table test of a pile group behind a sheet-pile quay wall embedded in the saturated sand (Fig. 1) was performed in this study, and this test was conducted at the Institute of Engineering Mechanics, China Earthquake Administration. In this shake-table test, a rectangular laminar container was used with the dimension of 1.7 m in height, 2.2 m in width, and 3.5 m in length, and detailed information of this laminar container was described in Sun et al. [30].

The soil profile in the test consisted of a saturated 1.5-m thick sand stratum behind the quay wall (Fig. 2). The thickness of the saturated sand stratum in the front of the quay wall was 1.0 m. The water table was at the ground surface. The sand stratum was prepared using the water sedimentation method [31]. The sand material employed in the shake-table test was obtained from Harbin, China and its properties are listed in Table 1. The relative density (D_r) of the sand stratum was 45–50%, and the saturated density of this sand was approximately 1900 kg/m^3.

Prior to the preparation of the sand stratum, a 2 × 2 pile group of steel pipe piles, the outer diameter of which was 0.088 m, was installed behind the quay wall. In an attempt to achieve a fixed-end condition, the pile was inserted into a socket which was firmly connected to the base of the laminar container using ethoxyline resin. Static lateral pushover tests were performed on the pile group before the preparation of the sand stratum to evaluate the actual degree of fixity at the top and bottom of the pile group. The obtained rotational stiffness at the top and bottom of pile group is summarized in Table 2. The pile space in the pile group was 3 times of the pile diameter in both longitudinal and transverse directions. The Young's modulus (E) of the pile group was obtained from two tension tests, and the result is listed in Table 2.

The quay wall was placed in the laminar container before the preparation of the soil stratum, and was connected to the container base using a pin connection. The top of the quay wall was temporarily constrained before and during the preparation of the sand stratum. Before the shaking, the constraint on the top of the quay wall was removed, which would lead to the lateral spreading of the sand behind the quay wall. The material property of the quay wall is listed in Table 3.

Various sensors were installed to record the different response of the soil-pile system throughout the shaking (Fig. 2). For example, raster displacement meter was used to record the displacement of the liquefied soil. The base excitation was a sinusoidal wave with a frequency of 2 Hz, and the amplitude of which was approximately 0.18 g (bottom plot of Fig. 3). During the first 2 s, the base excitation was gradually increased from 0 to 0.18 g. Note that the base excitation was applied in the direction of perpendicular to the quay wall.

3. Test results

To facilitate the analysis of the test results, the time history results were divided into three stages: Stage 1 (i.e., 0–2.5 s): prior to liquefaction; Stage 2 (i.e., 2.5–5.6 s): development of the liquefaction-induced lateral spreading; and Stage 3 (i.e., 5.6–15 s): convergence of the lateral spreading. In this study, the soil and pile displacements towards the waterside are defined as positive.

3.1. Excess pore pressure and acceleration

Figs. 3 and 4 show the free-field (see Fig. 2) acceleration and excess pore pressure (u_e) time histories, respectively. In Stage 1, the amplitude of the free-field acceleration increased rapidly and attained the peak acceleration. The recorded u_e built up rapidly and much of the stratum reached the initial liquefaction (i.e., u_e is equal to the initial effective vertical stress) during first few cycles of shaking. In Stages 2 and 3, the amplitude of free-field acceleration decreased gradually as the soil liquefaction then maintained constant at a low amplitude until the shaking ended, indicating that the liquefied sand lost most of the shear strength. The liquefaction level u_e maintained constant until the shaking ended. It was noted that the u_e time history at the 1.4 m depth only reached about 90% of initial vertical effective stress. In addition, only slight fluctuations were observed in the acceleration and u_e time histories, showing an absence of significant dilation in the free-field liquefied soil response [32].

Due to the soil liquefaction, the period of the ground surface is longer than that of base excitation, which showed a longer period response. The recorded u_e was slightly greater than the initial effective vertical stress at depth of 0.2 m, which may be caused by the sinking of pore pressure sensors [22,29].

Fig. 5 shows the acceleration time histories at the pile cap, ground surface, and the base. In Stage 1, the acceleration of the pile cap increased gradually as the base excitation increased, and the maximum acceleration amplitude of the pile cap was reached at the end of this stage. The maximum acceleration of the pile cap was approximately 0.3g, which was much larger than the amplitude of base excitation, and this showed an amplification effect of the soil-pile system. In Stage 2, the pile cap acceleration decreased gradually while the amplitude of base excitation maintained constant. In Stage 3, lower ground surface acceleration was observed, because of the liquefaction of the saturated sand and the loss of the shear strength. As a result, the pile group vibrated as the piles were mounted at the base and under the free vibration, thus, the acceleration amplitude of pile cap increased gradually and then maintained a constant of about 0.25g.