RESPONSES OF PILES SUBJECTED TO BLAST-INDUCED LATERAL SPREADING

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INTRODUCTION

Liquefaction-induced lateral spreading has caused substantial damage to deep foundations, which in turn has resulted in damage to the superstructures. Recent researches from centrifuge tests (e.g., Wilson et al. 2000; Dobry and Abdoun 2001) and 1-g shake table tests (e.g., Tokimatsu et al. 2001) have provided valuable insight on behavior of piles during lateral spreading. However, several key aspects have not been clearly understood due to the complex mechanism of soil-pile interaction. In addition, scaling effect is often an issue when implementing the results from small-scale testing. Therefore, two full-scale blast-induced lateral spreading tests were conducted in the Port of Tokachi on Hokkaido Island, Japan. The overall research effort was lead by the Port and Airport Research Institute (PARI), with a primary objective of assessing the performance of two different quay walls which were subjected to lateral spreading (see Figure 1). The large test area enabled researchers to include additional experiments in the zone of liquefaction and lateral spreading without interfering with the primary objective of the quay wall test. UCSD installed three full-scale instrumented pile foundation systems: a single pile, a 4-pile group, and a 9-pile group. The main objective of this research is to evaluate loading conditions on the piles due to kinematic loading from laterally spreading soil.

This paper describes the details of test setup and the use of blasting technique to liquefy the soils and induce lateral spreading. In addition, the applicability of pushover analysis using the p-y method for piles subjected to lateral spreading problems is evaluated by comparing the analysis results with the test results from the full-scale lateral spreading experiments.

TEST SETUP

A layout of the test site for the first test is shown in Figure 1. The test site was approximately 25 m wide by 100 m long with 4% surface slope test bed. The site was bordered by traditional design quay walls (e.g., no consideration of seismic force in the design) on one end. The quay wall was anchored by tie-rods which were fixed to H-piles to reduce movement of the quay wall. The test piles consisted of a UCSD single pile, a group of Waseda University (WU) single piles, a 4-pile group, and a 9-pile group. The piles were instrumented with strain gauges to measure moments during lateral spreading. All the single piles had the free-head condition, while the pile groups were fixed to a rigid pile cap based on typical Caltrans design practice. Controlled blasting was used to liquefy the soil at the test site, and thus induce lateral spreading. Blast holes
were spaced at 6.0 m on center in the square grid pattern. The blasting starting from the southwest corner of the embankment and continuing toward the quay wall as shown in Figure 1.

The second lateral spreading test was carried out about one month after the first test with the same test piles and instrumentation from the first experiment still in place. The test was performed in an attempt to induce additional ground deformations and further evaluate the performance of the piles subjected to a higher level of soil deformation. More details on the site information and test setup can be obtained elsewhere (Ashford and Juinarnarongrit 2004).

![Diagram of site layout for 1st lateral spreading test]

Figure 1: Site layout for 1st lateral spreading test

**SOIL CONDITIONS**
The soil at the test site consisted of 7.5 m of hydraulic fill, underlain by 1 m of medium dense sand overlying a very dense gravel layer (see Figure 4). The ground water table was approximately 1 m below the ground surface. The hydraulic fill consisted of a 4-m layer of very loose to loose silty sand (SM) overlying a 3.5-m layer of very soft lean to fat clay with sand (CL to CH).

**TEST RESULTS**
Sand boils forming at the ground surface (see Figure 2) provided direct evidence that the ground had indeed liquefied as a result of the blasting. The array of pore pressure transducers was used
to provide the quantitative record of the blast’s effect on the pore water. A typical example of the observed excess pore pressure time-histories at various depths nearby the 9-pile group with their initial effective stresses ($\sigma''$) is presented in Figure 3. The results indicate that the excess pore water pressures at all depths reached a liquefaction plateau at about 25 seconds, confirming the success in using blasting technique to liquefy the soil. The results of pile response due to lateral spreading will be presented in the pile analysis section.

Figure 2: Sand boil after the first test  Figure 3: Excess pore pressure time-histories

**P-Y METHOD FOR PILES IN LATERAL SPREADING SOIL**

Application of this method in current design practice is mainly focused on the analysis of piles under inertial loading, with movement relative to a stationary soil mass (Reese et al. 1974; Matlock 1970). For the case of moving soil mass such as piles in lateral spreading soil, the moving soil mass will exert the load on the pile and displace the pile a certain amount depending on the relative stiffnesses between the pile and the soil. For this type of application, the free-field soil movement is required to input to the boundary ends of the Winkler springs as shown in Figure 4. Application of this method to piles in lateral spreading problems is rare in the literature due to the limited availability of physical data, particularly for the case of pile groups. As such, the applicability of pushover analysis using the p-y method for both single pile and pile group subjected to lateral spreading problems is evaluated as presented in the following sections.

**Single Pile**

To predict the behavior of single piles subjected to lateral spreading, the free-field soil displacement, $y_s$, is required to impose to the boundary ends of the Winkler soil springs along depths as shown in Figure 4. In this study, the free-field soil movement profiles (i.e., no influence from pile foundations) were obtained from the measured soil displacement profiles of a slope inclinometer between the pile groups, at the end of both the first and second experiments. Based on these data, simplified linear displacement profiles of the free-field soil movements were used for the boundary condition at the end of soil springs with the largest displacements at the ground surface of 0.43 m for the first test and additional 0.46 m from the second test for a total of 0.89 m. Soil springs at different depths were calculated based on standard p-y springs available in the literature (i.e., Reese et al.’s 1974 recommendations for non-liquefied cohesionless soil and Matlock’s 1970 recommendations for soft clay). Since the maximum response of the piles due to lateral spreading occurred at the end of the test, where the soil had
already been liquefied, the p-y curves for liquefied soil were used for the saturated sand layers. Details of p-y curves for liquefied sand used in this study are described below.

The characteristics of p-y curves for liquefied soil appear to depend on the soil relative density. For loose sand with relative density, Dr, of less than 40%, Wilson et al. 2000 and Tokimatsu et al. 2001 found that the p-y curves are flat, inferring that the soil pressure from liquefied soil is negligible, while for medium dense and dense sand with relative density of greater than 55% (i.e., Ashford and Rollins 2002; Wilson et al. 2000; Tokimatsu et al. 2001) the p-y curves are concave up due to soil dilation. The relative densities at the Tokachi test site were slightly over 30% for the first 4-m of the sand layer and about 45% for the second sand layer at depths between 7.5 m and 8.5 m. Therefore, zero soil spring stiffness was used for the liquefied soil layers (i.e., from depths of 1 m to 4 m and from depths of 7.5 m to 8.5 m). A summary of soil properties used in the pushover analyses is given in Figure 4. All the analyses in this paper were conducted using LPILE Plus 4.0m computer code (Reese et al. 2000).

Figure 4: Soil displacement profiles from 1st and 2nd tests at Tokachi, and p-y analysis model for single pile

Pile Groups
The approach used to analyze pile group behavior in this study was adopted from the method proposed by Mokwa (1999) and Mokwa and Duncan (2003). In this method, it is recommended that the piles in a group can be modeled as an equivalent single pile with a flexural stiffness equal to the number of piles in the group multiplied by the flexural stiffness of a single pile within the group. Figure 4 shows a schematic of the numerical model used for the analysis of 4-pile group subjected to lateral spreading. The p-multiplier approach was used to reduce the soil stiffnesses for each pile in the group to account for group effects. The reduced soil spring stiffnesses were then summed to develop the combined soil springs for the pile group. For the soil passive pressure acting on the pile cap, it was modeled using the sand p-y curves (Reese et
al. 1974), considering the width of pile cap as a pile diameter. In addition, the pile head boundary condition of the group-equivalent pile was determined by estimating the rotational restraint provided by the piles and pile cap and was represented by a rotational spring as suggested by Mokwa (1999). Finally, the group-equivalent pile, incorporating both the effect of pile head restraint and the effect of pile group behavior was analyzed by imposing the free-field soil movement profile at the boundary end of each soil spring. The input free-field soil movement profiles were the same as that used in the single pile.

Figure 5:  p-y analysis model for pile group

RESULTS of ANALYSES

Single Pile
Figure 6 presents a comparison between computed and measured pile responses of the single pile for the first and second experiments. It was found that the predicted pile head displacements (Figure 6a), pile head rotations (Figure 6b) and moment profiles (Figure 6c) were in very good agreement with the pile responses measured from both tests.

Figure 6a shows that for the first 8 m, the movement of the free-field soil mass was greater than the movement of the pile, which implies that the soil provided the driving force to the pile, as is also shown by the positive soil reaction in Figure 6d, except for the liquefied layer where zero reaction was assumed. Negative soil resistance indicates that the soil mass moved less than the pile, and therefore the soil provided the resistance force to the pile, as mostly occurred in very dense gravel layer (Figure 6d).
Pile Groups

Figure 7 and Figure 8 present the results of calculated and measured pile responses of the 4-pile group and the 9-pile group, respectively. The same soil properties and free-field soil displacement profile used in the case of single pile were also used for analyzing the behavior of pile groups. Three types of boundary conditions at the pile head were considered for the purpose of comparison; these include the free head condition, fixed head condition, and rotationally restrained pile head boundary condition. Neither the free head nor fixed head conditions provided a reasonable estimate of the measured pile behavior. The free-head case overestimated the maximum positive moment at depth, and gave zero moment at the pile head, while the fixed-head case under-predicted the maximum positive moment but overestimated the maximum negative moment. The deflections at the pile head obtained from the fixed-head case were smaller than that measured by approximately 50% for both the 4- and 9-pile groups. The free-head case over-predicted the pile head deflection by 53% and 60% for the 4-pile group and the 9-pile group, respectively.

The analysis results obtained using the rotationally restrained pile head boundary condition considerably improved the agreement between measured and computed responses for both 4-pile group and 9-pile group, as shown in Figure 7 and Figure 8, respectively. The computed pile moments were within a reasonable range of the measured moment from the test. The errors between computed and measured pile group displacements were 3% for the 4-pile group and 13% for the 9-pile group. Pile head rotation was somewhat overestimated on the 9-pile group as shown in Figure 8b, likely due to the difference in the amount of rotation between the pile heads and the pile cap.
SUMMARY AND CONCLUSIONS

Two full-scale tests using controlled blasting were conducted to study the behavior of a single pile and pile groups subjected to lateral spreading in the Port of Tokachi, Japan. The response of soil and piles were measured using various types of instrumentation. The results indicated that controlled blasting successfully liquefied the soil and induced lateral spreading. The magnitudes...
of free-field soil displacements between the test piles were approximately 40 cm for both tests. In addition, pushover analysis using p-y approach was used to predict the behavior of single pile and pile groups subjected to lateral spreading using a single set of baseline soil properties. The analysis results were compared to the results from the full-scale lateral spreading tests. Responses of the single pile subjected to lateral spreading were determined by imposing the known free-field soil movement profile measured during the tests to the boundary condition of Winkler springs. Standard p-y springs were used to model the stiffness of non-liquefied soil layers, while no soil resistance was used for the liquefied soil layer. For the case of pile groups, the piles in the groups were modeled as an equivalent single pile with a flexural stiffness equal to the number of piles in the group multiplied by the flexural stiffness of a single pile within the group. The rotational spring and a decrease of soil spring stiffnesses using p-multiplier approach were incorporated into the pile group model to account for the rotational pile group stiffness and the pile group effect, respectively. Then, the analyses for the case of pile group can be conducted in similar way as the single pile case. Computed pile responses for each pile foundation were compared to the measured responses obtained from the tests. Reasonably good agreement for all types of pile foundations considered in this study was obtained between the computed and measured responses. The results provide justification for the use of pushover analysis using p-y approach in piles subjected to lateral spreading problems. The method presented in this paper will benefit Caltrans in future design of single piles and pile groups subjected to lateral spreading.

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REFERENCES