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ANALYSES OF PILE RESPONSES BASED ON RESULTS FROM FULL-SCALE LATERAL SPREADING TEST: TOKACHI BLAST EXPERIMENT

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SUMMARY

Full-scale lateral spreading experiments using controlled blasting were conducted at the Port of Tokachi, Hokkaido Island, Japan to study pile behavior during lateral spreading. The pile foundations included a single pile, a 4-pile group and a 9-pile group. This paper presents some results from the tests. Furthermore, the *p-y* analysis method was used to analyze the pile responses during lateral spreading. A generally good agreement was observed between the calculated and measured responses of all types of pile foundations, which verified the use of *p-y* analysis method in piles subjected to lateral spreading problems.

INTRODUCTION

It is widely recognized from past earthquakes that lateral spreading has caused tremendous damages to pile foundations supporting structures. Understanding the soil-pile interaction during lateral spreading is therefore very important. Research to date based on small-scale centrifuge studies (e.g., Abdoun *et al*. [1]; Ramos *et al.* [2]; Dobry and Abdoun [3]; Taboada [4]), limited area 1-g shake table tests (e.g., Tokida *et al.* [5]; Hamada 2000 [6]; Meneses *et al.* [7]), and case histories (e.g., Hamada and O'Rourke [8]; O'Rourke [9]) has provided valuable insight on soil-pile interaction during lateral spreading. However, some major aspects have not been clearly understood because the mechanism of soil-pile interaction is quite complex.

To improve our understanding of the behavior of piles subjected to lateral spreading and minimize scale effect that may occur in small scale testing, several full-scale instrumented piles were subjected to blast-induced lateral spreading in experiments carried out in the Port of Tokachi on Hokkaido Island, Japan. This research project was the joint collaboration between the University of California, San Diego (UCSD) and several Japanese organizations. The overall research effort was lead by the Port and Airport Research Institute (PARI). UCSD, together with Waseda University (WU), collaborated with other Japanese researchers to install the lifeline specimens in the zone of lateral spreading through the PEER Lifelines

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Program with support from the California Department of Transportation (Caltrans), Pacific Gas & Electric, and the California Energy Commission.

In all, UCSD installed three foundation test specimens. These consisted of a single pile, a 4-pile group, and a 9-pile group. This paper presents the field test results and discussion on the pile behavior subjected to lateral spreading. Furthermore, the potential of using the *p-y* analysis method in piles subjected to lateral spreading problems was assessed by comparing the results from numerical modeling using the *p-y* approach with the results from the full-scale lateral spreading experiments.

SITE DESCRIPTION

Two full-scale lateral spreading tests were conducted in this study. However, only the first one is presented herein. A layout of the test site for the first test is shown in Fig. 1. The test site was recently man-made land that was completed about 2 years before conducting the test. It was built as a part of an expansion of the Tokachi port capacity by hydraulically placing fill without any ground improvement; as a result, the soil was very loose and highly susceptible to liquefaction.

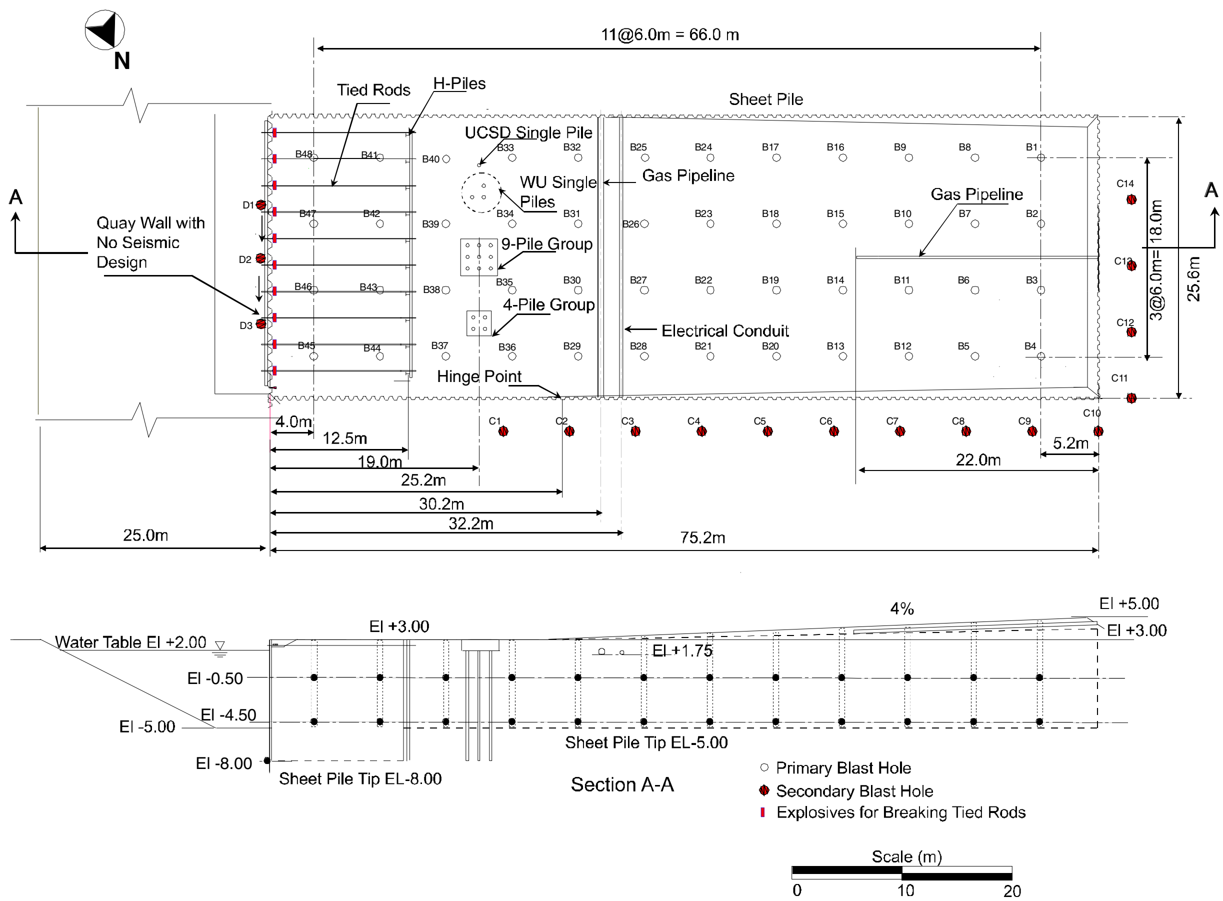

FIG. 1. Site Layout for 1st Lateral Spreading Test

The test site was approximately 25 m wide by 100 m long and was bordered by a traditional design of quay walls (e.g., no consideration of seismic force in the design) on one end. The quay wall was fixed to H-piles using a series of tie-rods to prevent the movement of quay wall. The UCSD pile specimens were located 19.0 m away from the quay wall. The test specimens consisted of a UCSD single pile, a group of Waseda University (WU) single piles, a 4-pile group, and a 9-pile group. All the single piles had the free-head condition, while the pile groups were fixed to a rigid pile cap. Controlled blasting was used to liquefy the soil at the test site, and thus induce lateral spreading. The blast holes were spaced at 6.0 m on center in the regular grid pattern as shown in Fig. 1. Others were installed surrounding the sheet pile wall. The sequence of the primary blasting started from the southwest corner of the embankment as denoted by blast hole B1, and then proceeded to the next hole to the north, and continued successively to the next rows towards the quay wall (from B1 to B48). Following the primary blasting, the secondary blast holes located around the perimeter of the test site (as denoted by C1 to C14) were detonated. Approximately 20 seconds after the completion of the secondary blasting, additional explosives were used to break the tie-rods of the quay wall, which allowed the quay wall to move freely to generate additional movement of the soil within the test area. More details on the site information can be obtained elsewhere (Ashford and Juirnarongrit [10]).

SOIL PROPERTIES

Fig. 2 presents a typical soil profile of the test site based on the soil boring investigation. The hydraulic fill was composed of a 4-m layer of very loose silty sand (SM) with Japanese SPT-N values ranging from 1 to 5. This was underlain by a 3.5-m layer of very soft lean to fat clay with sand (CL and CH). The SPT blow counts ranged from 0 to 2 blows per 30 cm in this layer. The water table was approximately 1 m below the ground surface.

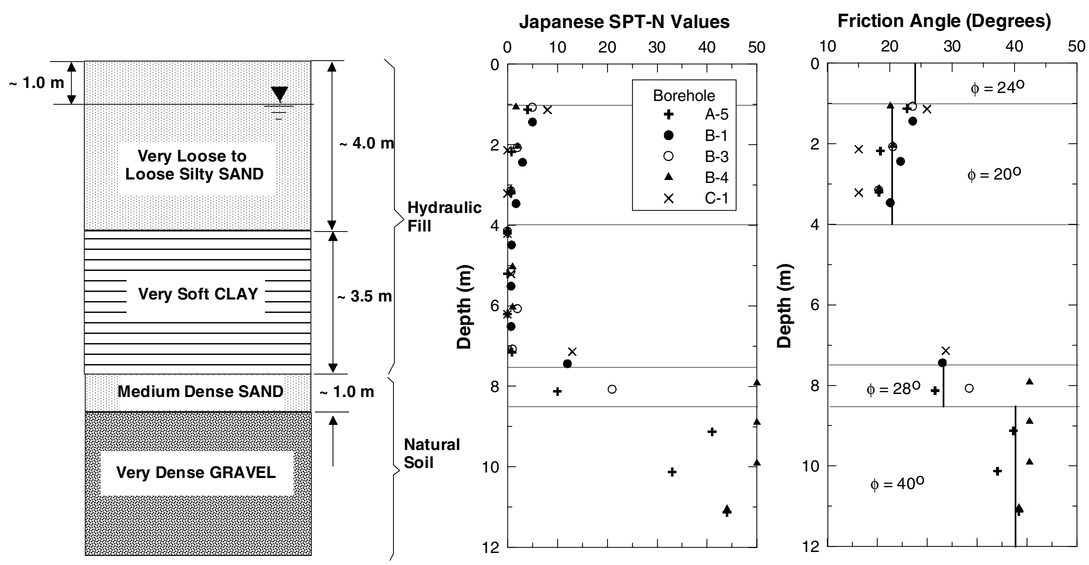

FIG. 2. Soil Properties of Japan Lateral Spreading Test Site

Since the experiment was conducted in Japan, the field exploration was carried out using the local standards of practice. In order to be consistent with our Japanese colleagues, soil friction angles as shown in Fig. 2 were calculated based on the Japanese standard correlation between the SPT-N value and friction angle, ϕ, suggested in the specification for highway bridges (Japan Road Association [11]). Cohesion of the clay layer was determined based on the SPT N-values using the correlation proposed by NAVFAC [12] for high plasticity clay, as well as data from cone penetration tests, which yielded the cohesion of 16 kN/m^2.

PILE PROPERTIES AND INSTRUMENTATION

All test pile specimens were 318 mm in diameter with wall thickness of 10.5 mm, a nominal length of 11.5 m, and a yield strength of 400 MPa. A total of 9 piles were extensively instrumented with electrical strain gauges: the single pile, two for the 4-pile group, and six for the 9-pile group. Steel channels C 75 mm x 6.92 kg/m (yield strengths = 400 MPa) were welded to the steel pipe piles to protect the strain gauges installed along the piles from damage during pile installation. The total pile stiffness, accounting for the stiffness of steel channels, was 36,300 kN-m^2.

The UCSD single pile was 11.2 m long, and was founded about 3 m into the dense gravel layer to ensure that it would behave as a flexible pile. For the pile groups, the piles were spaced at 3.5 pile diameters, center-to-center, corresponding to 1.11 m. The pile heads were restrained against rotation by reinforced concrete pile cap based on typical California Department of Transportation (Caltrans) design practice. The dimensions of pile caps of the 4-pile group and the 9-pile group were 2.33 m x 2.33 m x 1.00 m and 3.45 m x 3.45 m x 1.00 m, respectively. Since the piles were driven until refusal, the pile lengths varied according to the presence of the cobbles in the pile vicinity. The pile lengths in the group varied between 10.2 m and 11.5 m.

In addition to the strain gauges, other instrumentation was also installed to capture the behavior of soil and piles in more detail. These included pore pressure transducers at several depths, string-activated linear potentiometers, accelerometers, tiltmeters, slope inclinometer casings, soil pressure cells, and Global Positioning System (GPS) units. Layout of instrumentation in the region of pile specimens is presented in Fig. 3.

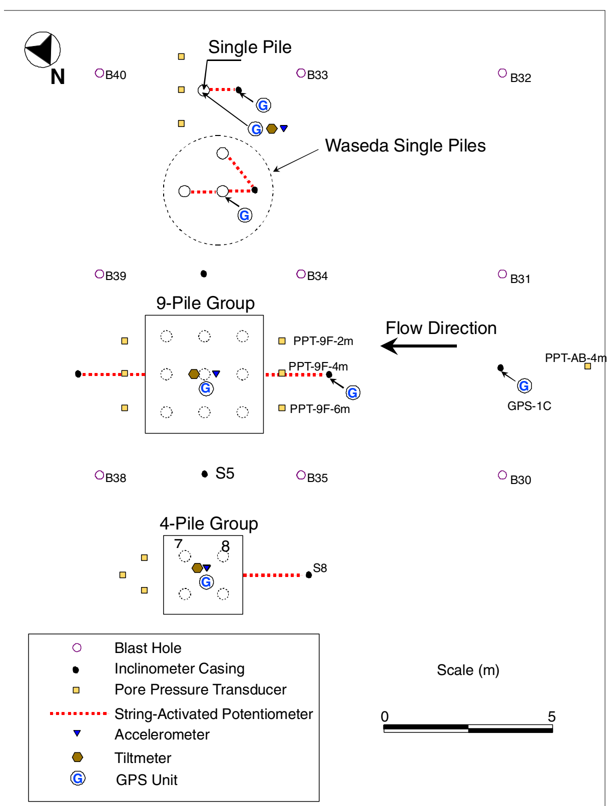

FIG. 3. Locations of Instrumentation

TEST RESULTS AND DISCUSSION

Excess Pore Water Pressure

Sand boils forming at the ground surface (Fig. 4) provided qualitative evidence that the ground had indeed liquefied as a result of the blasting. However, we relied on the array of pore pressure transducers to provide the quantitative record of the blast effect on the pore water.

FIG. 4. Sand Boil

Based on the measured excess pore pressure, the excess pore pressure ratio, R_u, at each location was calculated (e.g., $R_u = \frac{\Delta u}{\sigma_v'}$, where Δu is excess pore water pressure and σ_v' is the vertical effective stress). Fig. 5 presents a typical example time-history of R_u. This transducer, denoted as PPT-9F-2m, was located at depth of 2 m below the ground surface on the upslope of the 9-pile group. The results show that the soil in the vicinity of the 9-pile group was liquefied with the maximum R_u slightly exceeding 100%. The ratio dropped to about 90% at the end of the primary blasting, and then proceeded to dissipate with time. The evidence of increases in the excess pore water pressure ratio at about 40 seconds and 86 seconds was due to the effect of secondary blasting and blasting of tie-rods, respectively. The characteristics of R_u time-histories in other locations nearby the pile specimens were quite similar to the one presented herein. In summary, the R_u in the region of the pile specimens at the end of the primary blasting ranged from 75% to slightly over 100%.

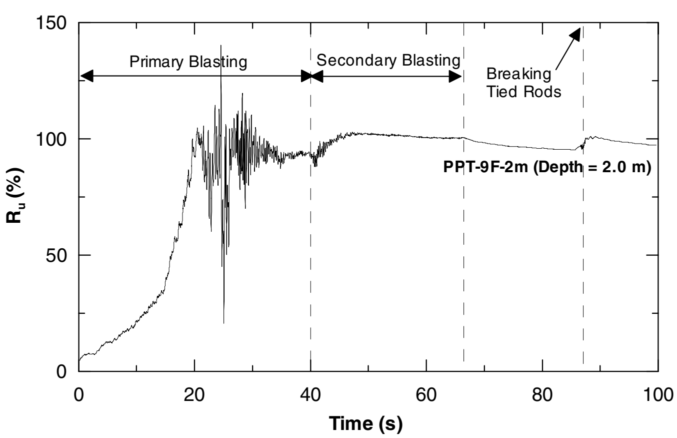

FIG. 5. Excess Excess Pore Water Pressure Ratio vs. Time nearby 9-Pile Group

Deformations of Ground and Pile Specimens

The GPS units were used to monitor the real-time movements of both ground and pile specimens during lateral spreading (Turner [13]). An example of time history of soil movements on the upslope side of the 9-pile group (denoted as unit 1C) in longitudinal, transverse, and vertical directions is presented in Fig. 6a, together with the excess pore water pressure ratio nearby that GPS unit (denoted as PPT-AB-4m). An increase in the excess pore water pressure was observed immediately after the blasting was initiated and continued to build up with time. Once the *Ru* reached 50% (about 10 seconds), the soil strength apparently dropped lower than the driving force of the soil mass. As a result, the translation of soil mass began to occur. As the blasting approached the GPS location, more movements in all directions were observed (i.e., spikes on the displacement time-histories). The rate of longitudinal movement between 10 seconds and 27 seconds (i.e., time at the blasting past the location of GPS unit) was fairy constant, about 1 cm/s. Beyond 27 seconds, the effect of dynamic force from the blasting was not important as indicated by the insignificant movements in transverse and vertical directions. However, the rate of lateral spreading of 1 cm/second still continued for another 5 seconds, likely due to the inertia effect of the soil mass. After 32 seconds, the rate of lateral movement began to die out. Increase in the soil movements at time of 40 seconds was due to the effect of secondary blasting around the sheet pile wall. Fig. 6b presents the displacement path of a GPS unit in the horizontal plane showing that the horizontal movement mainly occurred in the longitudinal direction towards the quay wall.

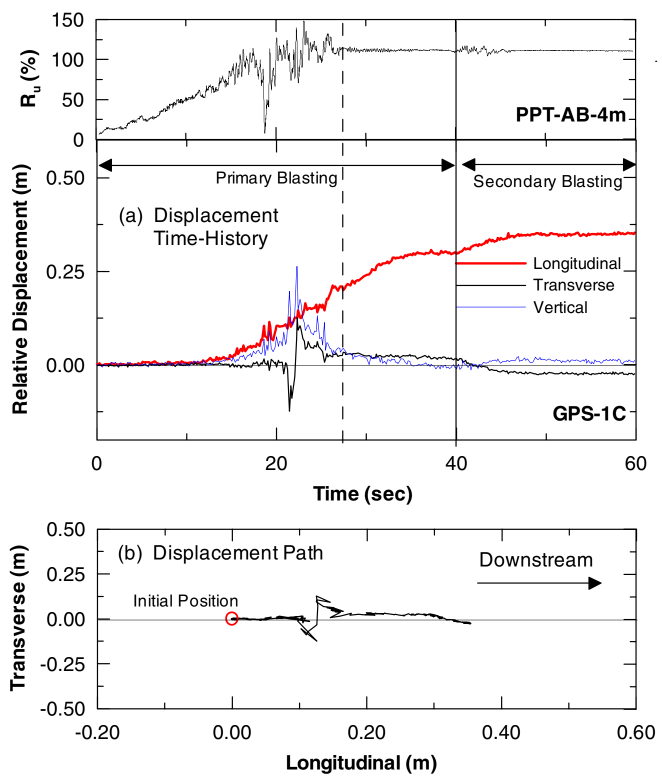

FIG. 6. Excess Pore Pressure vs. Time nearby 9-Pile Group at Various Depths for 1st Test

Based on the GPS data (Turner [13]) and survey data provided by Sato Kogyo Co. and Tobishima Co. (Sato Kogyo [14]), the displacement vectors in the horizontal plane for the first test are presented in Fig. 7.

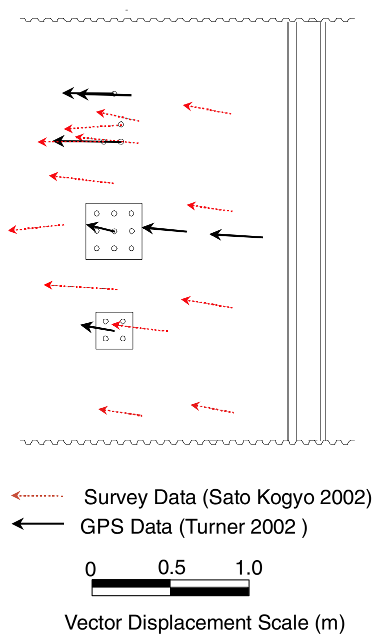

FIG. 7. Displacement Vectors from GPS Data

As shown in Fig. 7, the average displacement of the soil on the upslope side of the pile groups was approximately 30 cm. The movement of this upslope soil was somewhat impeded by the pile foundations located in front of the movement direction. Without this influence, as for the case of soil between the groups, the soil movement was approximately 30% greater than that of the upslope soil movement with the magnitude varying between 40 and 43 cm. The soil movement between the pile foundations likely represented the free-field soil movement. On the downslope side of the piles, the soil movement continued to increase towards the quay wall where the highest movement, over 1 m, had occurred.

The pile head displacement of the UCSD single pile (i.e., free-head pile) was 32 cm, while the 4-pile group and the 9-pile group (fixed-head pile), moved about 21 cm and 18 cm, respectively. The movements of both pile groups were approximately about 50% of the free-field soil movement. The movements of the pile foundations appeared to be dependent on the pile head condition. The pile with fixed-head condition moved less than the free-head pile due to the effect of pile head restraint in the groups contributing to resist the moment induced by the lateral soil pressure. This phenomenon was also observed from a recent research in the centrifuge testing conducted at Rensselaer Polytechnic Institute, where the test results indicated that the pile head displacement and moment in the single pile were significantly higher than those observed in the group (Abdoun [15]).

Response of Pile Foundation Systems

Based on the strain gauge data, the maximum strain in the piles occurred at the end of the test, which corresponded to the highest lateral movements of the piles. Therefore, to evaluate the pile performance in the worst case scenario, the strain gauge data at this period were chosen for calculating the moment distribution along the piles. The moments were calculated by multiplying the curvature data obtained from the strain gauges with the flexible rigidity of the pile that incorporated the stiffnesses of steel channels (EI = 36,300 kN-m^2). The samples of pile moment distribution will be shown in the results of analyses section, which will also be compared to the computed response using the *p-y* analysis method.

P-Y ANALYSIS METHOD

Application of the *p-y* analysis method in current design practice is mainly focused on an analysis of piles under lateral load moving against the stationary soil mass. However, in many cases, such as a pile subjected to lateral spreading, the soil mass itself will move toward and exert the load to the pile, which will displace the pile a certain amount depending on the relative stiffnesses between the pile and the soil. In this case, the soil loading must be considered by taking into account the relative movement between the soil and the pile. The response of the pile can then be obtained by solving the following differential equation:

$$EI\frac{d^4 y_p}{dz^4} - p(y_p - y_s) = 0 \tag{1}$$

where EI is the pile stiffness, y_p is the pile displacement, z is the depth, and y_s is the free-field soil movement

To predict the behavior of pile subjected to lateral spreading, the free-field soil displacement, y_s, needs to be known first, and then assigned to the boundary end of the soil spring at each depth as shown in Fig. 8.

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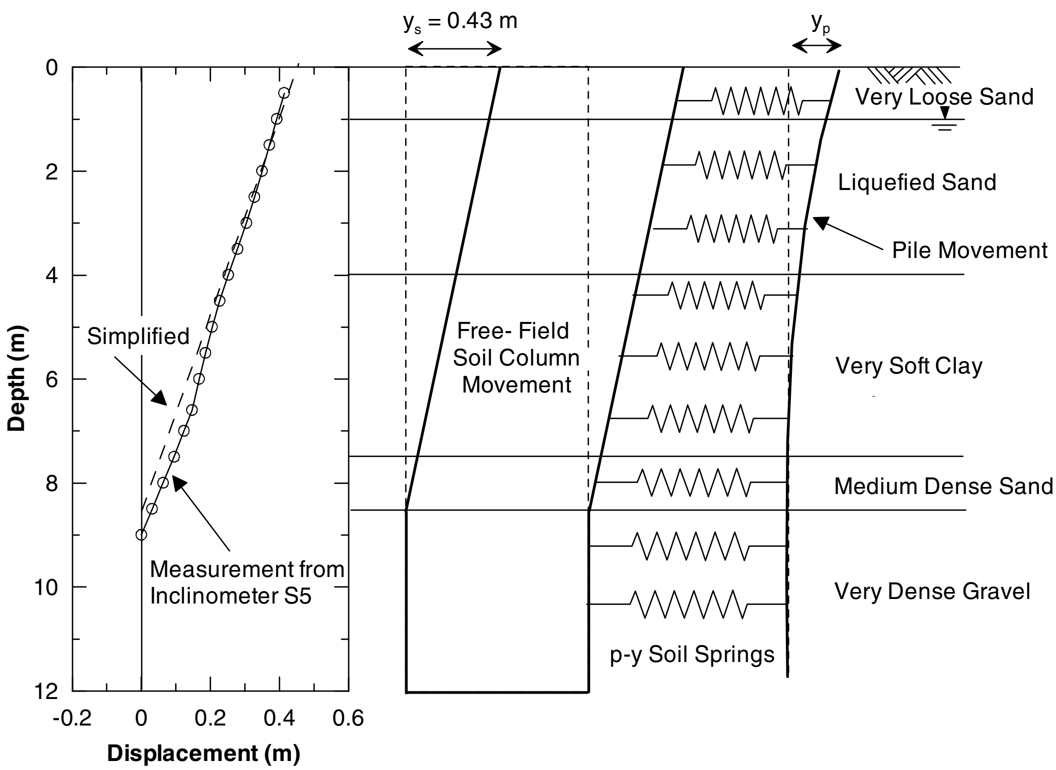

FIG. 8. *p-y* Analysis Model for Single Pile

This concept was incorporated into the LPILE Plus 4.0m computer code (Reese *et al.* [16]) using the finite difference technique to solve the above differential equation. The program also facilitates users by automatically generated *p-y* curves based on the soil types that are user specified. All the analyses in this paper were conducted using this program.

Analysis of Single Pile

In the analysis of the single pile subjected to lateral spreading, the free-field soil movement profile as presented in Fig. 8 was obtained from the measured soil displacement profile of a slope inclinometer between the pile groups (denoted as S5 in Fig. 3). The simplified linear displacement profile of the free-field soil movement was used for the boundary condition at the end of soil springs with the largest displacement of 0.43 m at the ground surface and zero movement at the interface layers between medium dense sand and very dense gravel. Soil springs at different depths were calculated based on standard *p-y* springs available in design practice. The *p-y* curves for sand were developed based on Reese *et al.* [17]'s recommendations, while the *p-y* curves for clay were obtained based on Matlock's [18] recommendations. 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 very loose saturated sand layer rather than using the *p-y* curves of non-liquefied soil.

The characteristics of *p-y* curves for liquefied soil depend on the soil relative density (Wilson *et al.* [19]). Results from the centrifuge tests at University of California at Davis indicated that for the soil relative density of 40%, the *p-y* curves are flat, inferring that the soil pressure of liquefied soil is negligible, while for the soil relative density of 55% the *p-y* curves showed the dilation behavior as that obtained from the full-scale lateral load test at Treasure Island (Ashford and Rollins [20]). Based on the SPT N-values, the relative density of the sand at the Japan test site, obtained from the equation proposed by Kulhawy and Mayne [21], was slightly over 30%. As a result, the liquefied soil layer in the Tokachi should not provide any loading or resistance to the pile. Therefore, zero soil spring stiffness was used for the liquefied soil layer (i.e., from depth 1 m to 4 m).

Analyses of Pile Groups

Two special considerations incorporated into the analysis of the pile groups were the effect of pile groups and the effect of pile head restraint at the pile cap. The approach used to analyze the behavior of the pile groups subjected to lateral spreading in this study was adopted from the method proposed by Mokwa [22].

In the analyses of the pile groups, the piles in a group were modeled as an equivalent single pile with four times the flexural stiffness of a single pile for the 4-pile group and nine times the flexural stiffness of a single pile for the 9-pile group. Fig. 9 shows a sample of the numerical model used for analyzing the behavior of 4-pile group subjected lateral spreading.

The soil springs for each pile were adjusted to account for group effects using the *p*-multiplier approach. The *p*-multiplier was obtained from results of previous research on 9-pile groups (Brown and Reese [23]; Morrison and Reese [24]; McVay *et al.* [25]; McVay *et al.* [26]; Rollins *et al.* [27]; Ashford and Rollins [20]). The adjusted soil springs were then summed to develop the combined soil springs for the group of piles. The group-equivalent pile *p-y* curves were determined by summing the *p*-multiplier of each pile in the group times the "*p*" values as:

$$p = \sum_{i=1}^{N} p_i f_{mi} \tag{2}$$

where p_i is the *p*-value for the single pile, f_{mi} is the *p*-multiplier, and N is the number of piles in the group.

The soil pressure acting on the pile caps was modeled by using the sand *p-y* curves (Reese *et al.* [17]) with the use of pile cap width as the pile diameter. The pile head boundary condition of the group-equivalent pile was determined by estimating the rotational restraint provided by the pile cap using a rotational spring. In general, a pile with a fixed head condition is usually assumed in analysis of a pile group, however, in reality some rotation of the pile cap often occurs in the pile group, mainly caused by the vertical movement of the piles in the group. Using the approach in estimating the pile head rotation by Mokwa [22], the rotational spring stiffnesses for the 4-pile group and the 9-pile group can be determined as 16,000 kN-m/rad and 71,000 kN-m/rad, respectively.

Finally, the group-equivalent pile, incorporating both the effect of pile head restraint and the effect of pile group behavior was analyzed with specifying the free-field soil profile at the boundary end of each soil spring. The process was the same as that for the single pile.

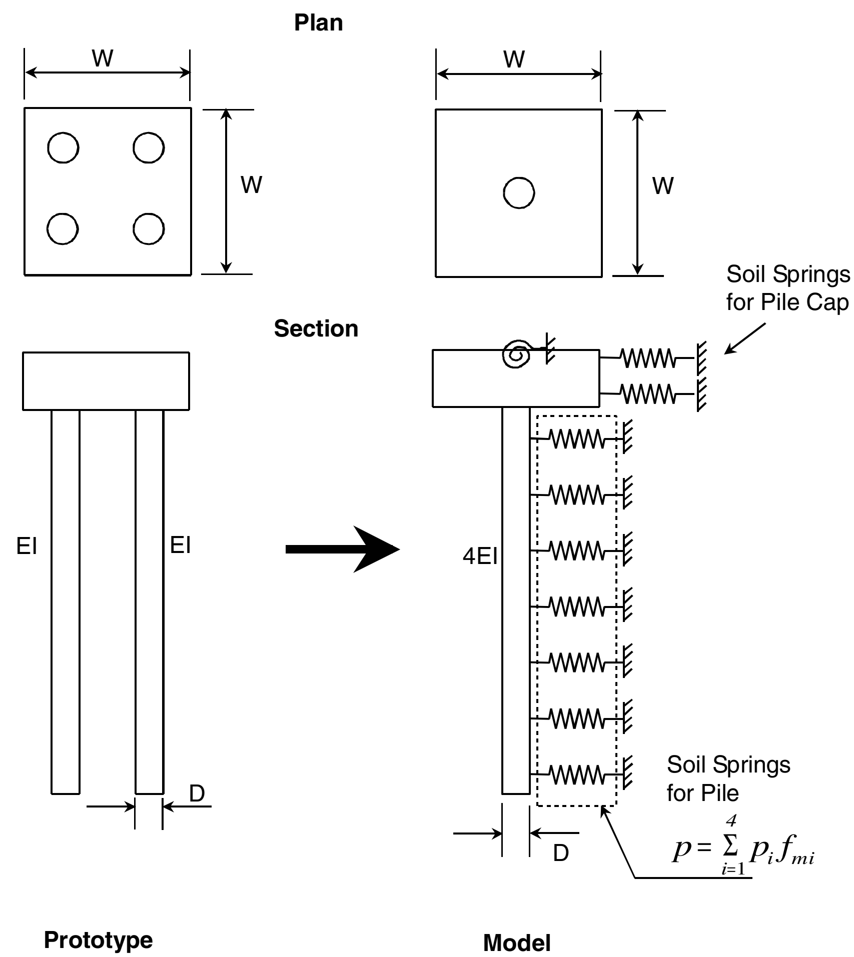

FIG. 9. ***p-y*** **Analysis Model for Pile Group**

RESULTS OF ANALYSES

Fig. 10 presents a comparison between computed and measured pile responses of the UCSD single pile (i.e., flexible pile). It should be noted that the measured moment (Fig. 11c) was insignificant for the first 4 m of the very loose liquefied sand layer indicating that the resultant force on the pile produced by the liquefied soil was negligible. This characteristic of liquefied soil was incorporated into the *p-y* springs for liquefied layer as discussed in the earlier section.

Predicted pile head displacement (Fig. 10a), pile head rotation (Fig. 11b) and moment profile (Fig. 10c) were in excellent agreement with the measured responses from the test. Fig. 10a show that for the first 8 m, the movement of 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 resistance force in Fig. 10d. Negative soil resistance indicated 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 (Fig. 10d).

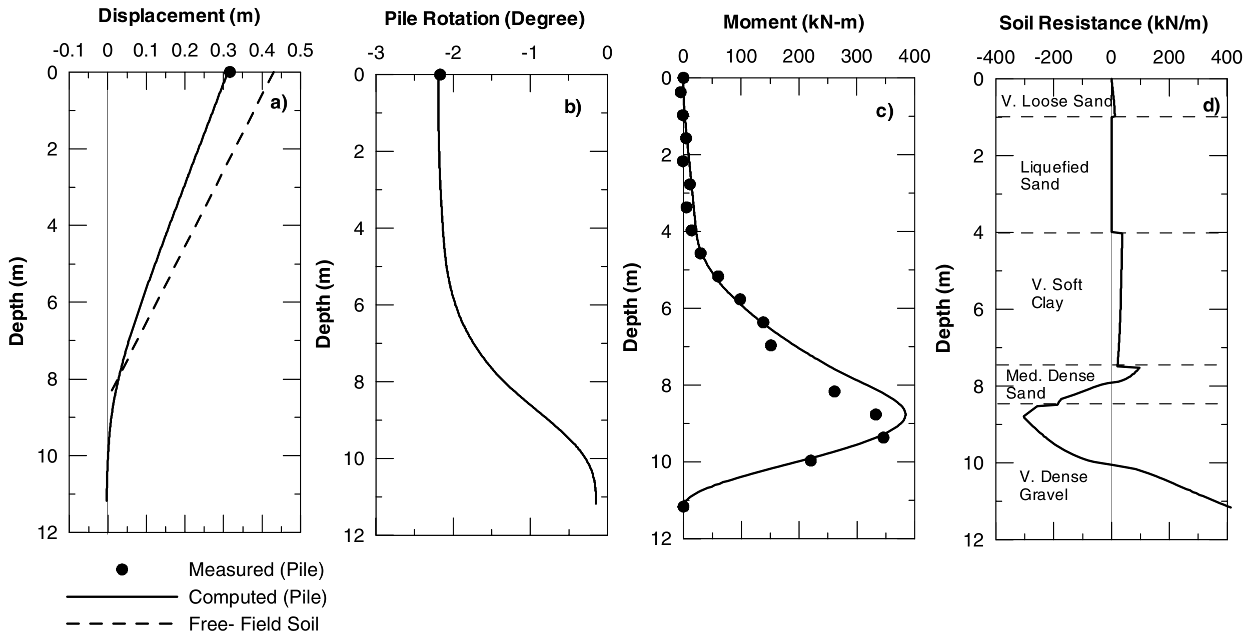

FIG. 10. Comparison between Measured and Computed Pile Responses for UCSD Single Pile (Flexible Pile)

Fig.11 presents the results of calculated and measured pile responses of the 4-pile group. The same set of baseline soil properties used in the case of single pile was also used for analyzing the behavior of the 4-pile group. Three types of boundary conditions at the pile head were considered in this study 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 48% for the 4-pile group. The free-head case over-predicted the pile head deflection by 46%. The analysis results obtained using the rotationally restrained pile head boundary condition considerably improved the agreement between measured and computed responses. The computed pile moment was in a reasonable range of that measured moment from the test. An excellent agreement between computed and measured pile head displacements was observed when using the rotational spring at the pile head.

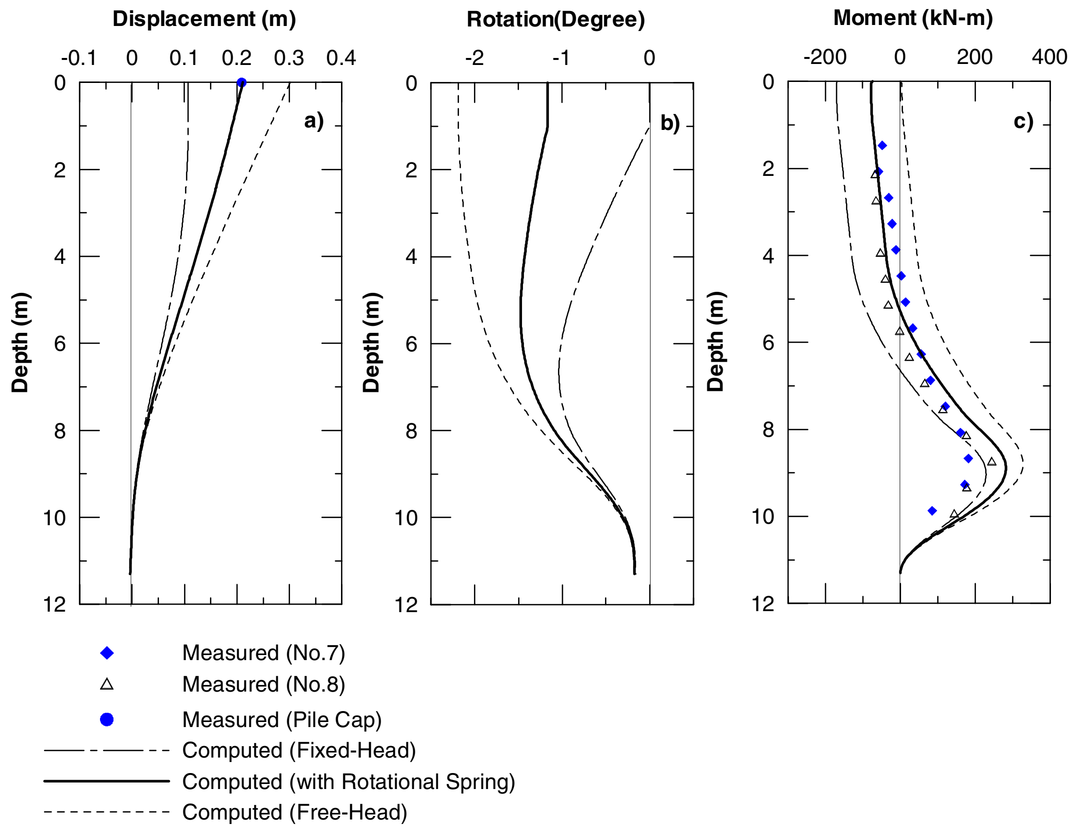

FIG. 11. Comparison between Measured and Computed Pile Responses for 4-Pile Group

Reasonably good agreement was observed between measured and computed pile responses for both single pile and pile group considered in this study using a single set of baseline soil properties in the analyses. This leads to the conclusion that the *p-y* analysis method was able to model the behavior of piles subjected to lateral spreading with reasonable degree of accuracy.

CONCLUSIONS

The main findings of this paper can be summarized as the followings:

- Controlled blasting successfully liquefied the soil and induced lateral spreading with the magnitudes of free field soil movements of 43 cm.
- Observed moment distribution of the single pile indicates that global translation of liquefied soil layer provided insignificant force to the pile.
- Reasonably good agreement for all types of pile foundations considered in this study was obtained between the computed and measured responses. The results provide verification for the use of *p-y* analysis method in piles subjected to lateral spreading problems.

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