Observed Lifeline Performance in the Port of Tokachi Lateral Spreading Experiment

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Abstract: Two full-scale experiments using controlled blasting were conducted in the Port of Tokachi on Hokkaido Island, Japan, to assess the performance of lifeline facilities subjected to the lateral spreading. Lifeline specimens in this study included a single pile, a 4-pile group, a 9-pile group, two natural gas pipelines, and one electrical conduit. All of them were extensively instrumented with strain gauges to measure the distribution of moment during the lateral spreading. This allowed computing the loading condition, as well as conducting the damage and performance assessments on the lifeline facilities. Other instrumentation including pore pressure transducers, GPS units, and slope inclinometers, were also installed to measure the degree of liquefaction as well as the movements of soil and lifelines. This paper presents some of the test results and the discussions on the performance of piles and pipelines observed from the experiments.

1. INTRODUCTION

Lateral spreading, which usually refers to global displacements of gently sloping ground due to liquefaction, is one of the primary earthquake hazards. In past earthquakes, lateral spreading has caused considerable damage to civil infrastructure including port facilities, buildings, bridges, and utilities. Good examples are the damage of quay walls and buildings in the 1995 Kobe earthquake; damage of pile foundations in the 1964 Niigata earthquake; the damage of over 250 bridges and numerous embankments along the Alaskan Railroad and Highway during the 1964 earthquake; the damages of numerous water and gas lines in the 1906 earthquake; and the significant damage in the San Francisco area in 1989 Loma Prieta earthquake (Bartlett and Youd 1992b; Seed 1987; Youd and Hoose 1976; Bardet and Kapuskar 1993; Clough *et al*. 1994; and O' Rourke and Pease 1992). Therefore, it is extremely essential to understand the behavior of soil as well as structures during the lateral spreading in order to improve the current design method for structures and lifeline utilities to prevent the catastrophic failure for the future earthquakes. Meanwhile, most lateral spreading research to date has focused on small-scale centrifuge studies (e.g. Abdoun *et al*. 1996), limited area 1-g shake table tests (e.g. Tokida *et al*. 1993), or case histories (e.g. Hamada and O'Rourke 1992; O'Rourke 1996). In addition, some full-scale has been carried out to study behavior of deep foundations in sand liquefied by controlled blasting (e.g. Ashford et al. 2000), but these tests do not account for the global translations of the lateral spreading soil mass. In light of this, the full-scale instrumented lifeline components in controlled lateral spreading tests were carried out in order to understand the performance of lifelines and be able to implement the test results in engineering practice. The test results will be an invaluable source of data for further development of the empirical methods and/or complex numeral models to use to design lifeline facilities subjected to lateral spreading.

Two full-scale experiments using controlled blasting were conducted in November and December 2001 in the Port of Tokachi on Hokkaido Island, Japan, to study the performance of lifeline facilities subjected to the lateral spreading. This research project was the joint collaboration between the University of California San Diego (UCSD) and several Japanese organizations. This overall research effort was lead by Dr. Takehiro Sugano of the Port and Airport Research Institute (PARI). The primary objective of the test was to assess the performance of quay walls subjected to the lateral spreading using controlled blasting. One quay wall was of traditional design and new seismic design criteria was applied to the other. Since the test area was so large, it 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. The University of California, San Diego, together with Waseda University (WU) lead by Professor Masanori Hamada, collaborated with other Japanese researchers to install the lifeline specimens in the zone of lateral spreading through the PEER Lifelines Program with support from Caltrans, Pacific Gas & Electric and the California Energy Commission.

In all, UCSD installed 6 test specimens. The pile specimens in the experiment program consisted of a single pile, a 4-pile group, and a 9-pile group. In addition, two natural gas pipelines and one electrical conduit were installed. The objectives of this study is to conduct damage and performance assessments of those lifelines subjected to lateral spreading, as well as evaluate loading conditions on the structures during the lateral spreading.

2. SITE CHARACTERIZATION

The test site was a recent man-made land and the construction was completed just a few years ago. The land was built by hydraulically placing fill without any ground improvement; therefore, the soil was very loose and highly susceptible to liquefaction.

Subsurface soil exploration program was carried out in many areas throughout the test site to characterize the soil condition. Generally, the soil condition consisted of a 7.5 m of hydraulic fill underlain by a 1 m of medium dense sand overlying a very dense gravel layer as presented in Figure 1. The hydraulic fill was comprised of a 4-m layer of very loose silty sand with uncorrected 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. Uncorrected SPT blow counts ranged from 0 to 2 blows per foot in this layer. The water table was approximately 1 m below the ground surface. Figure 2 presents the grain size distribution of the hydraulic fill plotted together with the Japanese standard curves for liquefaction potential evaluation. The first 4 m of the soil fell into a zone of highly susceptible to liquefaction. Below this layer, fine contents increased with depth. Only a thin layer of soil at depths between 7.0 to 7.5 m was not liquefiable. Based on the results of grain size analysis and the strength characteristic, the soil at the test site was highly susceptible to liquefaction, and therefore appropriate for conducting the full-scale lateral spreading test.

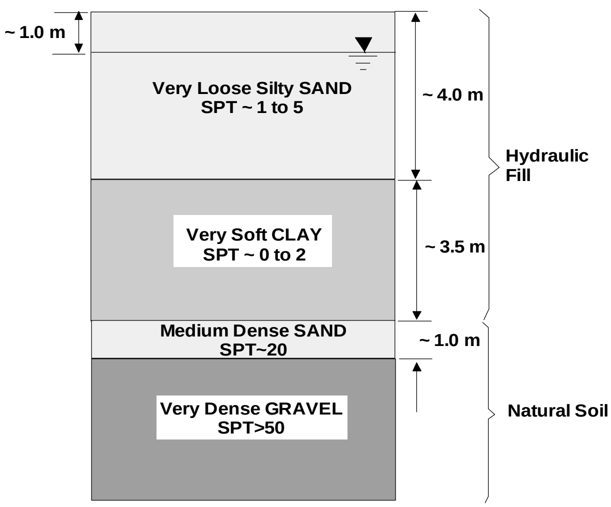

Figure 1 Typical Soil Profile of Test Site

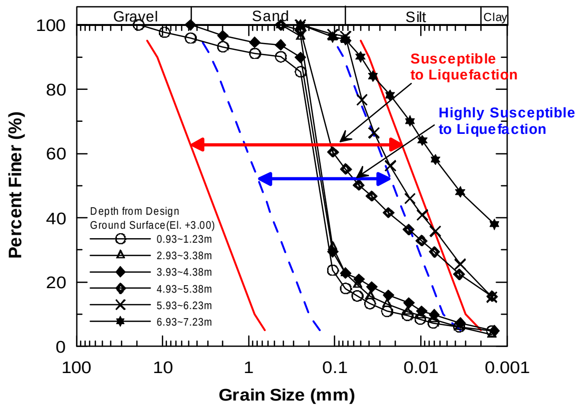

Figure 2 Grain Size Distribution of Soil at Test Site

3. SITE DESCRIPTION AND TEST SETUP

The UCSD experiments were located in a zone of the unapplied seismic design quay wall where the large global translation of the soil was expected. A layout of the test site for the first experiment is shown in Figure 3. The test site was approximately 25 m wide by 100 m long. The front face was bordered by a water way. The water elevation was approximately +2.00 m on the test day. The sheet pile quay wall was driven to the elevation of -8.00 m and was anchored by the tie rods which were fixed to H-piles to prevent the movement of the quay wall. The quay wall retained approximately 7.5 m of hydraulic fill. The ground surface started to gently elevate upwards at 25.2 m away from the quay wall with the embankment slope of 4%. The test site was surrounded by the sheet piles to tip elevations between -5.00 and -8.00 m.

The UCSD pile specimens were located at 19.0 m away from the quay wall. The pile specimens consisted of a single pile, a 4-pile group, and a 9-pile group. A group of free head single piles of WU were also located in this region. The pile diameters were 318 mm with wall thickness of 10.5 mm, and a nominal length of 11.5 m. The yield strength of these steel pipe piles was 400 MPa.

In addition, two natural gas pipelines and one electrical conduit were installed. The gas pipeline consisted of a 500 mm diameter pipe with wall thickness of 6 mm and yield strength of 400 MPa. The electrical conduit consisted of a 268 mm diameter with wall thickness of 6 mm and yield strength of 400 MPa. Both pipelines were about 25 m long and located across the test sites at 30 m and 32.2 m away from the quay wall. The bottoms of both pipelines were installed at the elevation of +1.75 m. The other gas pipeline was 22 m long and installed parallel to the direction of the flow. The center of the pipeline was 1 m below the ground surface along its entire length.

As success in using the controlled blast to induce liquefaction of the soil in several tests in Japan as well

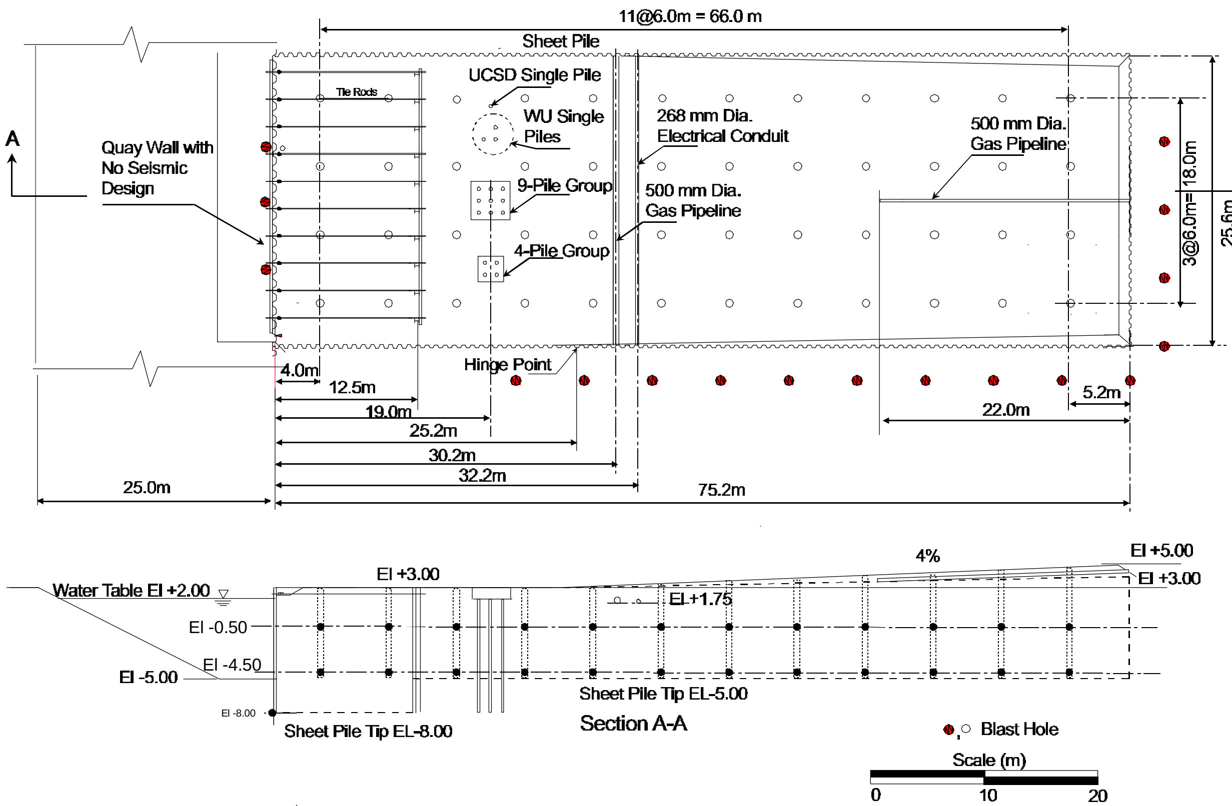

Figure 3 Site Layout of 1st Lateral Spreading Experiment

as the full-scale lateral load tests at Treasure Island (Ashford *et al.* 2000), the same technique was implemented to liquefy the soil at the test site, and thus induce the lateral spreading. The blast holes were spaced at 6.0 m on centers in the regular grid pattern. The charges were installed at depths of 3.5 m and 7.5 m below the ground surface. The amount of charges varied from 2 kg nearby the pile specimens to 3-5 kg at other areas. It was done this way so as to prevent damage to a large number of instruments installed in the vicinity of pile specimens. The first experiment was carried out on November 13, 2001. The sequence of the blasting started from the back corner of the embankment and then proceeded successively towards the quay wall. This was followed by the detonation of the secondary blast holes around the perimeter of the test site. The purpose of these explosives was to loosen the soil in the vicinity of sheet pile to allow unrestricted flow of the soil in such region. Approximately 10 seconds after a completion of secondary blasting, the additional explosives were used to break the tie rods of the quay wall and allowed the quay wall to move freely to create additional movement of the soil within the test area.

The second lateral spreading test was performed with an attempt to induce additional ground deformations and further evaluate the performance of lifeline facilities subjected higher level of soil deformation. The test site for the second lateral spreading test was significantly modified from the first one as presented in Figure 4. The test site was approximately 30 m wide by 40 m long. The quay wall and sheet piles surrounding the test site were removed to allow the soil to move freely. The waterway was excavated on one end of the test site to the elevation of -1.00 m with the slope 1:2 and then filled with the water to the elevation of +2.00 m. The ground surface was level for a distance of 7.5 m away from the edge of the waterway and then started to rise up with the embankment slope of 6% over a distance of 18.0 m. The blast holes were spaced at 6.0 m on centers in a regular grid pattern. The charges were installed at depths of 4.0 m and 8.0 m below the design ground surface (El +3.00m). The amount of charges varied from 2 kg to 4 kg. Two additional rows of blast holes were drilled. One was located on the steep slope adjacent to the waterway with the amount of explosives ranging from 1 to 3 kg. The purpose of these explosives was to loosen the soil at the slope toe prior to the primary blasting sequence such that the embankment soil behind it had a high potential to move freely with larger deformation once the primary blasting initiated. The other was located between the pipelines and pile as denoted as blast holes No. 7 to No. 9. Three kilograms of explosives were installed at El. -3.00 m.

The weather condition for the second lateral spreading experiment was awful as presented in Figure

5 due to the heavy snowfall with the snow thickness of about 0.50 m, and a new record of wind speed of 100 kph on the test day. The ground was frozen throughout the test site which would likely impede the global translation of the soil mass. In an attempt to mitigate this, jackhammers were used to break up the frozen ground in the vicinity of test specimens to depths of approximately 20 to 30 cm below the ground surface as presented in Figure 6. The second test was carried out on December 14, 2001. The explosives on the steep slopes were detonated initially from S1 to S5. Approximately 15 second later, the primary sequence of the blasting was started. The primary blast began at blast hole No.1 on the back of the embankment. Then, the blasting proceeded to the next holes of the same rows, and then continued to the next row towards the waterway (i.e., from No.1 to No.17).

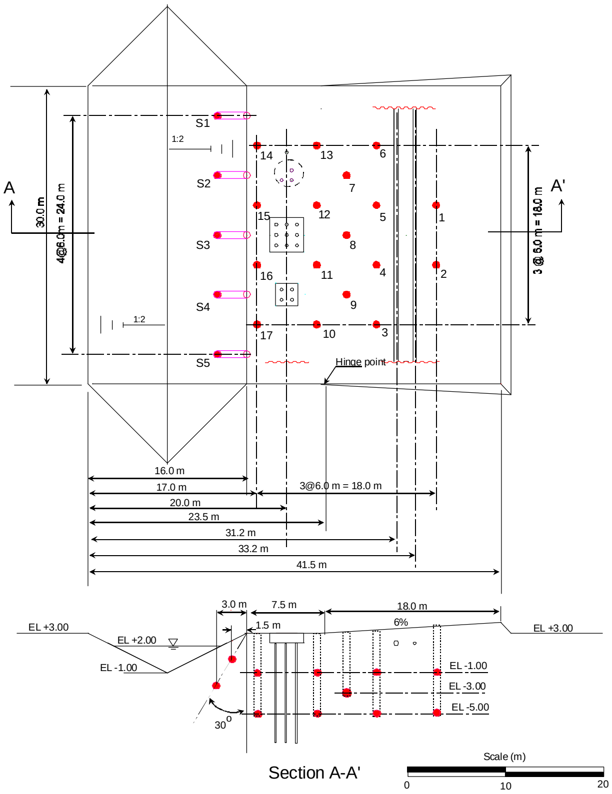

Figure 4 Site Layout of 2nd Lateral Spreading Experiment

Figure 5 Awful Weather Condition during 2nd Lateral Spreading Experiment

Figure 6 Breaking up Frozen Ground Surface Using Jack Hammer

4. INSTRUMENTATION

Piles and pipelines were extensively instrumented with electrical strain gauges. The strain gauges of pile specimens were located at 0.6 m intervals on both upstream and downstream sides of the piles to measure the bending moment along the length of the pile. A series of tiltmeters at various depths were also installed on one pile of each foundation system to use as backup data for strain gauges. Unfortunately, all of them were damaged during the pile installation. The 75x40x5 steel channels with the yield strength of 400 MPa were welded to the steel pipe piles to protect the strain gauges from damage during the pile installation. The strain gauges of the gas pipeline were spaced between 1.0 m and 3.0 m along the top and the side of the pipelines to measure the bending moment along the pipelines in both horizontal and vertical directions, respectively.

Apart from the strain gauges, other instrumentation was also installed to capture behaviors of soil and lifelines in more details. These include pore pressure

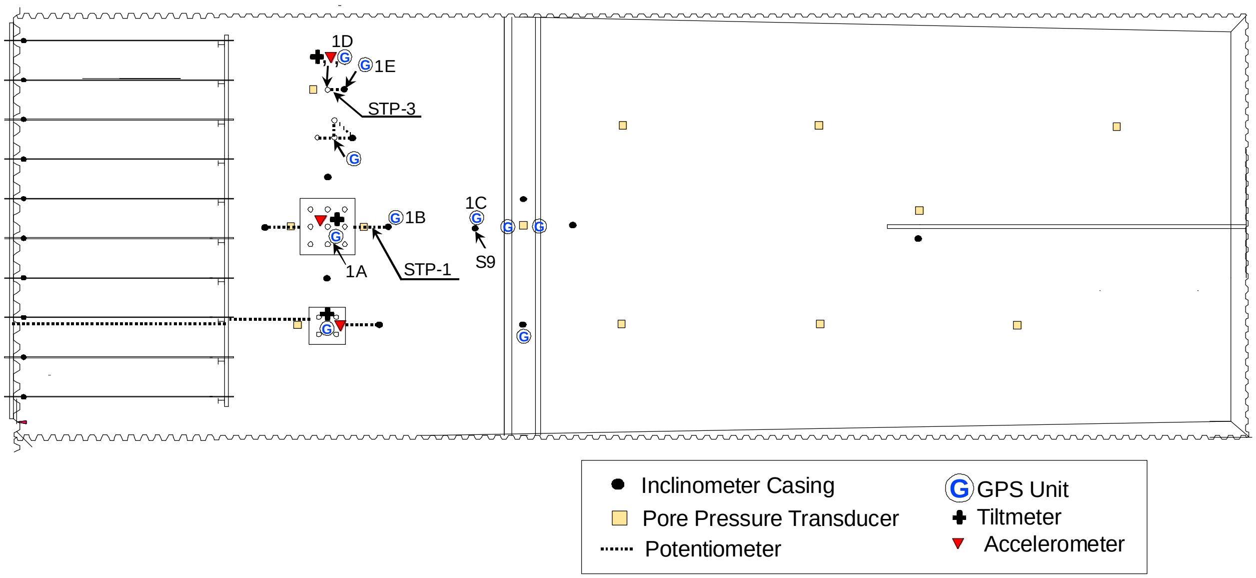

Figure 7 Instrumentation Plan for 1st Lateral Spreading Test

transducers, soil pressure cells, string-activated linear potentiometers, accelerometers, slope inclinometer casings, and Global Positioning System (GPS) units. Layout of instrumentation for the first experiment is presented in Figure 7. The instrumentation for the second experiment was essentially the same as the first test; therefore, it is not shown in this paper.

5. TEST RESULTS

5.1 Moment Distribution

Moment distribution along the length of the single pile at the end of the test is presented in Figure 8. The test results indicate that the moments at depths between 0 and 4 meters were insignificant. One of possible explanations on this phenomenon is that after the soil was liquefied, it becomes to behave like viscous fluid material being able to flow around the pile without significant force acting on the pile. The soil resistance began to increase with depth for the next 3.5 m where very soft clay layer existed. The maximum moment occurred in a dense soil layer at depth about 9 m below the ground surface. The pile was yielded after the second test.

Figure 9 presents the moment distribution of pile No.5 in the 9-pile group. The shape of moment profile from the experiment agreed well with a typical analysis of a pile with fixed head condition showing that the results from the test were reasonable and appropriate for further analysis to estimate the loading distribution of liquefiable soil on the pile. Figure 10 presents the moment profile of each pile in the group after the first experiment. The moment distribution of all piles in the 9-pile group was more or less similar, except for pile No. 2 and No.4 where the moments were smaller than the others. This is likely due to the fact that both piles were shorter in length, and thus the degree of fixity into the dense soil layer was less, resulting in smaller moment developed in the piles. It is noted that pile No. 2 and No.4 reached refusal during the pile installation, likely due to the presence of boulder at that particular depth. The similarity of moment distribution of each pile in the group indicates that shadowing effect was unimportant in liquefied soil. This conclusion was similar to that of a recent research on the behavior of pile group in liquefied soil conducted at Treasure Island (Ashford and Rollins 2002).

After the second test, all piles in the 4-pile group and the 9-pile group remained elastic with the maximum moment below 70% and 60% of yield moment, respectively. No any structural damage was observed on piles to pile cap connections on both pile groups though both pile groups experienced the total movements of nearly 40 cm. In addition, based on the strain gauge data of pipelines (not presented in this paper), all the pipelines performed comparatively well without any yielding.

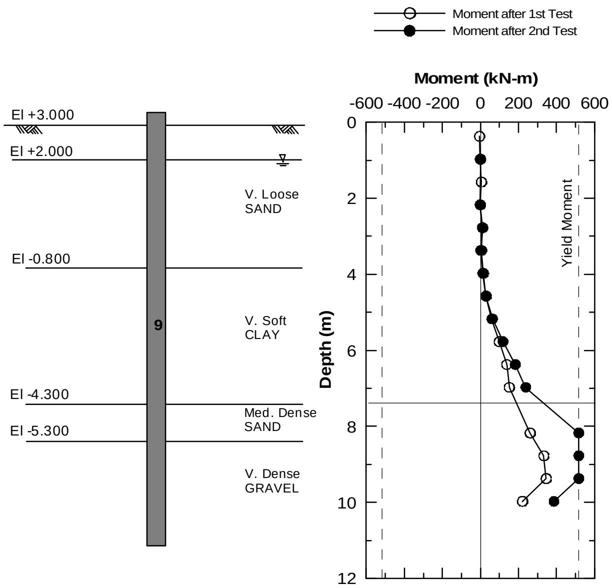

Figure 8 Moment along Single Pile

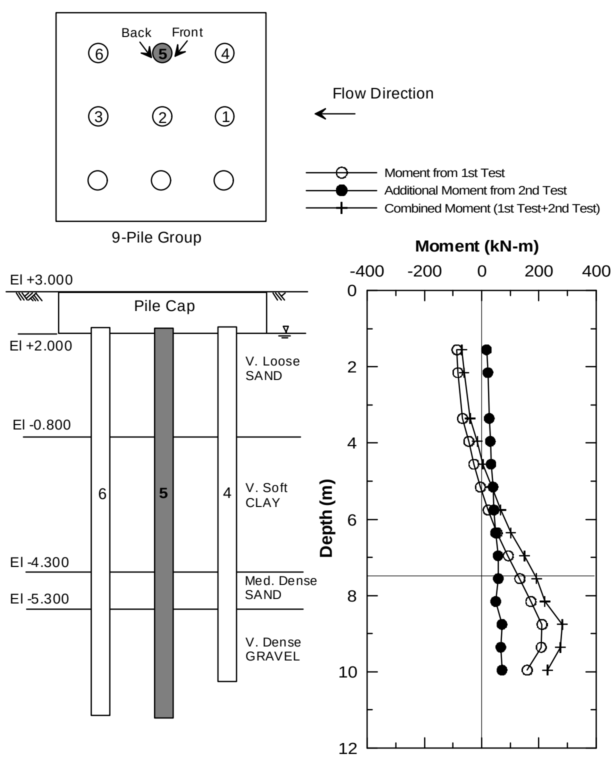

Figure 9 Moment along Pile No.5 of 9-Pile Group

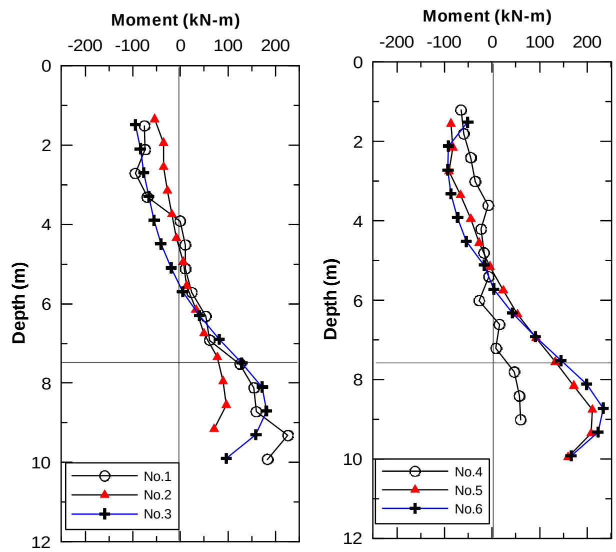

Figure 10 Moment Distribution of Each Pile in 9-Pile Group (Data Extracted from 1st Experiment)

5.2 Excess Pore Water Pressure

An example of excess pore water pressure ratio time-history nearby the 9-pile group at depth of 2 m below the ground surface is presented in Figure 11. The excess pore water pressure ratios built up immediately after the blast though this transducer was located about 40 m away from the first blast hole. The rate of increase in pore water pressure become more rapidly as the blast moved closer to the transducers. The increase in pore water pressure ratios proceeded to reach the maximum values at approximately 30 seconds. Fluctuation of pore pressure ratios was obvious as the blasting occurred in the vicinity of transducer location. The results show that the soil in the vicinity of the 9-pile group was liquefied with the maximum excess pore pressure ratios exceeded 100%. The ratios dropped to about 80% after the blast stopped, then proceeded to dissipate with time. The characteristics of excess pore water pressure ratios in other locations were basically the same as the one presented herein. The excess pore water pressure ratios throughout the entire test site exceeded 70%. Some of them were slightly over 100%. Sand boil was observed following the blasting as presented in Figure 12 confirming that the liquefaction had occurred.

The excess pore pressure ratios in the 2nd test appeared to be much less than those measured during the first test with values ranging between 30% and 80%. No any sand boil was observed in the 2nd test. Two possible reasons can be explained regarding the lower excess pore pressure ratios. Firstly, the soil was less susceptible to liquefaction because some settlement took place after the first experiment and caused the soil become denser. Secondly, frozen ground decreased the liquefaction potential.

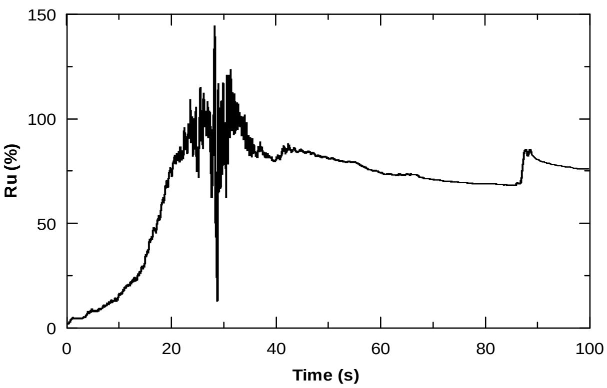

Figure 11 Excess Pore Pressure Ratio vs. Time nearby 9-Pile Group

Figure 12 Sand Boil after the 1st Experiment

5.3 Deformations of Ground and Lifelines

The GPS units were used to monitor the movements of both ground and lifeline facilities during the lateral spreading. The measurements were conducted by a research team from the California Department of Transportation (Caltrans) (Turner 2002). An example of time history of soil movements on the downstream side of the gas pipeline (denoted as unit 1C) in longitudinal, transverse, and vertical directions is presented in Figure 13a. The movements of GPS units were observed at time about 10 seconds after blasting initiated. As the blasting moved closer to the GPS location, more movements in all directions were observed. The lateral movements between 10 seconds and 27 seconds were due to not only the liquefaction-induced lateral spreading but also the dynamic forces generated by the blasting. With the blasting past the location of GPS units (at about 27 seconds), the effect of dynamic forced from the blasting was not important as indicated by the insignificant movements in transverse and vertical directions. The longitudinal movement observed after 27 seconds was therefore primarily due to liquefaction-induced lateral spreading. Figure 13b presents the displacement path of GPS unit in the horizontal plane showing that the horizontal movement mainly occurred in the direction of flow. The vector displacements in horizontal plane throughout the test site for the first test are presented in Figure 14a. The largest horizontal displacement was about 55 cm occurring at the pile head of WU pile. Taking into the consideration of the pile rotation and pile height above the ground surface, the movement of WU pile at the ground surface was computed as 42 cm. The UCSD single pile moved only 32 cm, which was significantly less than the WU piles. This was likely due to the fact that the WU piles were shorter in length and the pile tips were located just above the dense layer; while the UCSD pile was penetrated about 3.5 meters into the dense soils. The WU piles were therefore likely behaved as a rigid pile, in which the rotation and movement at the pile tip were expected. In contrast, the UCSD pile acted as a flexible pile where the rotation and the movement at the pile tip was insignificant. As a result, the displacement at the pile head of the UCSD single pile was less than those of the WU piles. The 4-pile group and the 9- pile group moved 21 and 18 cm, respectively. The data from the GPS units in the vicinity of the pipelines show that the movements of the gas pipeline and electrical conduit were similar with the magnitude of about 38 cm. The average of soil movement was about 35 to 40 cm.

As presented in Figure 14b, the horizontal movements occurred in the second test were generally lower than those occurred in the first test, especially in the vicinity of the pipelines. This was mainly due to the weather condition that decreased the liquefaction potential and thus impeded the global translation. The movement of the gas pipeline was about 50% of that occurred in the first test. The average soil movement in the second test ranged from 10 to 23 cm. One GPS units installed between two pile caps showed the soil movements as high as 45 cm. However, 10 cm of 45 cm attributed to the movement of slope toe due to the effect of initial blasting along the slope edge. The movement of pile groups in the 2nd test ranged from 16 cm to 18 cm, slightly less than that in the first test. The movements of WU and UCSD single piles at the ground surface were 42 cm and 28 cm, respectively.

The accuracy of real time kinematic GPS (RTK-GPS) data was checked against the relative displacements obtained from string-activated linear potentiometers in two locations as presented in Table 1. The locations of GPS units and potentiometers involved in Table 1 are illustrated in Figure 7. Excellent agreement between measurements from GPS units and linear potentiometers were observed with the difference being within 1 cm, which is the accuracy typically associated with RTK-GPS methods. This confirmed the consistency of measurements using the RTK-GPS method.

Table 1 Verification of GPS Measurements with Data from Potentiometers

GPS Location	Potentiometer Location	GPS Measurement (m)	Potentiometer Measurement (m)
GPS 1A and 1B	STP-1	0.106	0.096
GPS 1E and 1D	STP-3	0.012	0.002

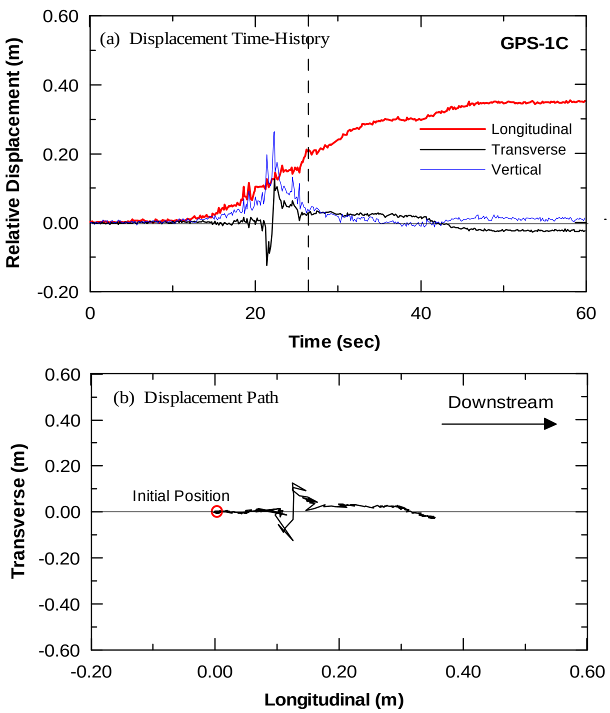

Figure 13 GPS Data (a) Displacement Time-History, and (b) Displacement Path

Profiles of soil displacement at the end of the experiments were measured from the slope inclinometer readings. Figure 15 presents the results of the soil movement in the vicinity of pipeline (S9). The results indicated that the maximum movement of the soil occurred at the ground surface as expected. The displacement at ground surface obtained from slope inclinometer measurement was in good agreement with that from GPS data. Slightly less displacement obtained from inclinometer data was possibly because an absolutely fixed boundary condition at the tip of the casing was assumed to compute the soil displacement profile. In fact, some movement of inclinometer casings at the tips might occur resulting in underestimating the soil displacements. Based on the slope inclinometer data, the movement of the soil at the ground surface for the 1st experiment ranged approximately from 18 to 41 cm with the average values of 30 cm. The soil movements within the embankment area observed from the second test varied between 9 cm and 22 cm. Large displacements were observed in the zone of slope failure with the average values of 40 cm.

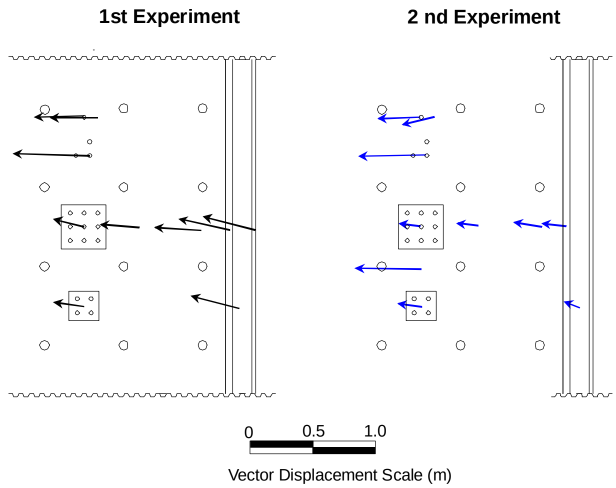

Figure 14 Vector Displacements from GPS Data for (a) 1st Experiment and (b) 2nd Experiment

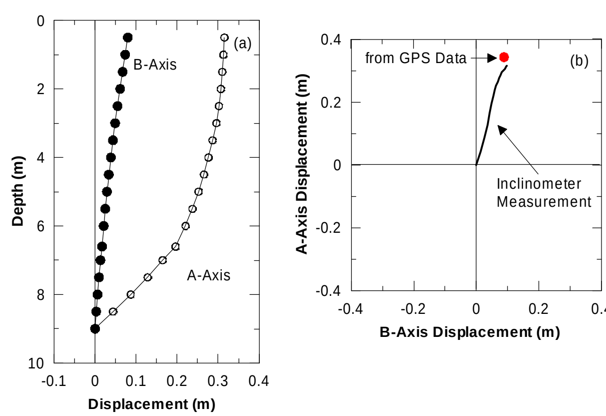

Figure 15 Soil Displacement Obtained from Slope Inclinometer Data (a) Section View, and (b) Plan View

6. CONCLUSIONS

Based on the results obtained from two full-scale experiments, the following conclusions can be obtained:

1. Using controlled blasting successfully liquefied the soil and induced lateral spreading.

2. The excess pore water pressure ratios exceeded 70% for the first experiment. The degree of liquefaction in the second experiment was much lower than the first one with the excess pore pressure ratios ranging between 30% and 80%, likely due to the weather condition.

3. The average soil movements of the first experiment were about 35 cm to 40 cm, while about 9 cm to 22 cm of ground movements were observed in the second test.

4. The total movements of the single pile, 4-pile group, and 9-pile group were 60 cm, 39 cm, and 34 cm, respectively.

5. The total movements at the middle of gas pipeline and electrical conduit were about 54 cm.

6. Shadowing effect of the 9-pile group in liquefied soil was not observed.

7. Both 4-pile and 9-pile groups performed well during both experiments. Piles remained in elastic range with the maximum moments less than 70% of yield moment. No damaged was observed on piles to pile cap connection.

8. Single pile was yielded at the end of the second experiment.

9. All pipelines also performed very well without any yielding.

7. ACKNOWLEDGMENTS

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The assistance from Dr. Takehiro Sugano of Port and Airport Research Institute (PARI) and Professor Masanori Hamada from Waseda University, the original leaders of this project, in providing a great opportunity for us to participate for this test, as well as effectively managing the project is gratefully acknowledged.

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8. REFERENCES

Abdoun, T. Dobry, R., and O-Rourke, T. D. (1997). "Centrifuge and numerical modeling of soil-pile interaction during earthquake induced soil liquefaction and lateral spreading," *Observation and Modeling in Numerical Analysis and Model Tests in Dynamic Soil-Structure Interaction Problems, Geotechnical Special Publication No. 64,* ASCE, New York, pp.64-70.

Ashford, S. A., Rollins, K. M. (2002). *TILT: Treasure Island Liquefaction Test Final Report, Report No. SSRP-2001/17,* Department of Structural Engineering, UCSD.

Ashford, S. A., Rollins, K. M., Bradford, S. C., Weaver, T. J., and Baez, J. I. (2000). "Liquefaction mitigation using stone columns around deep foundations: fill-scale test results," *Soil Mechanics 2000, Transportation Research Record No. 1736*, TRB, Washington D.C., pp.110-118.

Bardet, J. P., and Kapusker, M. (1993). "Liquefaction sand boils in San Francisco during 1989 Loma Prieta earthquake." *J. Geotech. Engrg.,* ASCE, Vol. 119, No.3, March, pp. 543-562.

Bartlett, S. F. and Youd, T. L. (1992b). "Empirical analysis of horizontal ground displacement generated by liquefaction-induced lateral spreads." *Tech. Rep. NCEER-92-0021*, National Center for Earthquake Engineering Research, Buffalo, N.Y., M. Hamada and T.D. O'Rourke (eds.), August 17.

Clough, G. W., Martin, J. R., II, and Chameau, J. L. (1994). "The geotechnical aspects." *Practical Lessons from the Loma Prieta Earthquake*, National Research Council, National Academy Press, Washington, D.C., pp. 29-63.

Hamada, M., and O'Rourke, T., Editors. (1992). *Case Studies of Liquefaction and Lifeline Performance During Past Earthquakes, Report No. NCEER-92-0001, Vol.1*, National Center for Earthquake Engineering Research, Buffalo, N.Y.

O'Rouke, T. D. (1996). "Lessons learned for lifeline engineering from major urban earthquakes," *Proceedings, Eleventh World Conference on Earthquake Engineering, Elsevier Science Ltd*., 18 p.

O'Rouke, M. J., and Pease, J. W. (1992). "Large ground deformations and their effects on lifeline facilities: 1989 Loma Prieta earthquake." *Case Studies of Liquefaction and Lifeline Performance During Past Earthquakes, Volume 2: United States Case Studies, Tech. Rep. NCEER-92-0002*, M. Hamada and T.D. O'Rourke (eds.), February 17, 85 pages.

Seed, H. B. (1987). “Design problems in soil J. *Geotech. Engrg.*, ASCE, Vol. 113, No. 8, August, pp. 827-845.

Tokida, K., Iwasaki, H., Matsumoto, H., and Hamasa, T. (1993). “Liquefaction potential and drag force acting on piles in flowing soils,” *Soil Dynamic and Earthquake Engineering*, Computational Mechanics, South Hampton, England, pp. 349-364.

Turner, L. L., (2002). *Measurements of Lateral Spread Using a Real Time Kinematic Global Positioning System*, California Department of Transportation Division of New Technology &Research, April 2002, 94 pages..

Youd, T. L., and Hoose, S. N. (1976). “Liquefaction during 1906 San Francisco Earthquake.” *J. Geotech. Engrg. Div.*, ASCE, Vol. 102, No. GT5, May, pp. 425-439.