Article ID: 1671-3664(2002)02-0181-13

Response of single piles and pipelines in liquefaction-induced lateral spreads using controlled blasting

Scott A. Ashford† and Teerawut Juirnarongrit‡

Department of Structural Engineering, University of California, San Diego, La Jolla, CA 92093-0085

Abstract: Two full-scale experiments using controlled blasting were conducted in the Port of Tokachi on Hokkaido Island, Japan, to assess the behavior of piles and pipelines subjected to lateral spreading. Test specimens were extensively instrumented with strain gauges to measure the distribution of moment during lateral spreading. This allowed us to compute the loading condition, as well as to conduct damage and performance assessments on the piles and pipelines. This paper presents the test results and discussions on the response of single piles and pipelines observed from the full-scale experiments. Based on the test results, it can be concluded that using controlled blasting successfully liquefied the soil, and subsequently induced lateral spreading. The movements of the single pile, as well as the transverse pipelines, were approximately the same as the free field soil movement. Observed moment distribution of the single pile indicated that global translation of the liquefied soil layer provided insignificant force to the pile. In addition, the degree of fixity at the pile tip significantly affected the moment along the pile as well as the pile head displacement. The pile with a higher degree of fixity at the pile tip had smaller pile head displacement but larger maximum moment.

Keywords: piles; pipelines; pile tests; lateral spreading; liquefaction; soil-pile interaction

1 Introduction

In past earthquakes, lateral spreading has caused considerable damage to civil infrastructure including port facilities, buildings, bridges, and utilities. Examples of damage to deep foundations and lifeline utilities due to liquefaction-induced lateral spreading are the foundation piles of the Yachiyo and Showa bridges and NFCH building during the 1964 Niigata earthquake (Hamada, 1992a); the railway bridge foundations during the 1991 Limon earthquake (Youd *et al.*, 1992); the batter piles supporting the 7th Street Terminal Wharf in the 1989 Loma Prieta earthquake (Benuzca 1990); and the damage to numerous water and gas lines in the 1906 San Francisco earthquake (Bartlett and Youd, 1992). Therefore, it is extremely essential to understand the behavior of soil as well as lifeline utilities during lateral spreading in order to improve current design methods and prevent catastrophic failure during future earthquakes. Meanwhile, most liquefaction and lateral spreading research to date has focused on small-scale centrifuge studies (Abdoun *et al.*, 1996; Ramos *et al.*, 2000; Wilson *et al.*, 2000; Dobry and Abdoun 2001), limited area 1-g shake table tests (Tokida *et al.*, 1993; Hamada, 2000; Meneses *et al.*, 2002), or case histories (Hamada and O'Rourke, 1992b; O'Rourke, 1996). In addition, some full-scale testing has been carried out to study the behavior of deep foundations in sand liquefied by controlled blasting (Ashford *et al.*, 2000), but these tests do not account for global translations of the lateral spreading soil mass.

In light of this, full-scale instrumented lifeline components in controlled lateral spreading tests were carried out in the Port of Tokachi on Hokkaido Island, Japan in order to understand the behavior of lifelines and be able to implement the test results in engineering practice. The test results will be a valuable source of data for further development of the empirical methods and/or complex numerical models to use to design lifeline facilities subjected to lateral spreading.

This research project was a joint collaboration between the University of California, San Diego (UCSD) and several Japanese organizations. The overall research effort was led 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 Program with support from the California Department of Transportation (Caltrans), Pacific Gas & Electric, and the California Energy Commission. In all, UCSD installed six test specimens. The pile

Correspondence to: Scott A. Ashford, Department of Structural Engineering, University of California, San Diego, La Jolla, CA 92093-0085.
Tel: 858-822-0431; Fax: 858-822-2260;
Email: sashford@ucsd.edu
†Associate Professor; ‡ Graduate Student Researcher

specimens in the experimental program consisted of a single pile, a four-pile group, and a nine-pile group. The pipeline specimens included two natural gas pipelines and one electrical conduit. In addition, three single piles were also installed in the area by WU. This paper presents test results and discussion focused on the behavior of single piles and pipelines subjected to lateral spreading.

2 Site characterization

The test site was recent man-made land that was completed just a few years ago as a part of an expansion of the Tokachi port capacity. The land was built by hydraulically placing fill without any ground improvement; therefore, the soil was very loose and highly susceptible to liquefaction.

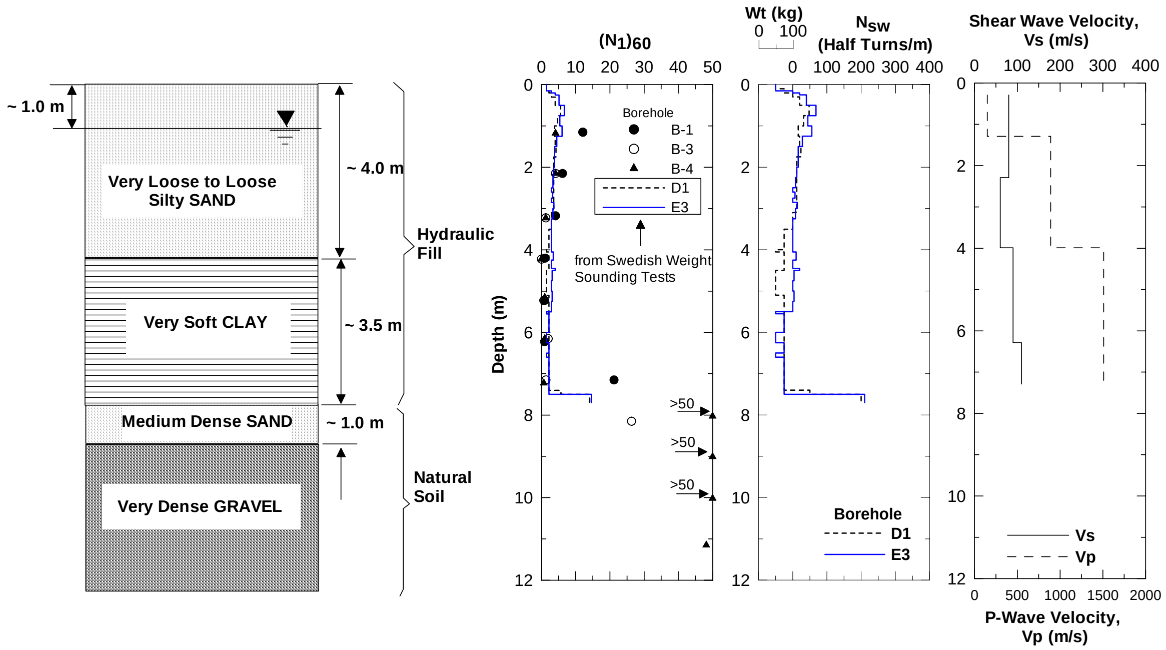

Fig. 1 Typical soil profile of test site

A subsurface soil exploration program was carried out in many areas throughout the test site to characterize the soil condition. Figure 1 presents a typical soil profile of the test site based on the soil boring investigation. Generally, the soil condition consisted of 7.5 m of hydraulic fill underlain by a 1 m of medium dense sand layer overlying a very dense gravel layer. The water table was approximately 1 m below the ground surface. According to the Unified Soil Classification System: ASTM D2487-93 (ASTM 1998), the hydraulic fill consisted of a 4 m layer of very loose to loose silty sand (SM) with the corrected SPT-N values, $(N_1)_{60}$, ranging from 1 to 12. The SPT N-values were corrected based upon hammer type and release system, sampler configuration, short rod lengths, and overburden stresses. This was underlain by a 3.5 m layer of very soft lean to fat clay with sand (CL and CH). Since this layer was cohesive soil, the effect of overburden stresses was not incorporated into the correction. The corrected SPT blow counts in this layer ranged from 0 to 2 blows per foot. In addition to the Standard Penetration Tests, Swedish weight sounding and shear wave velocity tests were conducted. The results of these tests are also shown in Fig. 1. The results of Swedish weight sounding methods were converted to the SPT N-values using the correlation based on Japanese Industrial Standard: JIS A 1221-1995 (Japanese, 1995), which yielded a good agreement with the measured SPT N-values. Shear wave velocity of less than 100 m/s for the hydraulic fill layer also supports that the strength of the soil in this layer was very low.

Fig. 2 presents the grain size distribution of the hydraulic fill plotted together with the Japanese standard curves for liquefaction potential evaluation (Port and Harbor Research Institute, 1997). Generally, the fines content gradually increased with depth. The first 4 m of the soil fell into a zone highly susceptible to liquefaction while below this layer, the liquefaction potential was less, due to the greater amount of fine contents. Only a thin layer of soil at depths between 7.0 and 7.5 m fell outside the liquefaction curves, indicating that this layer was not liquefiable. Based on the results of grain size analysis and the strength characteristic, the first 7 m layer of soil at the test site was susceptible to liquefaction, and therefore appropriate for conducting the full-scale lateral spreading test.

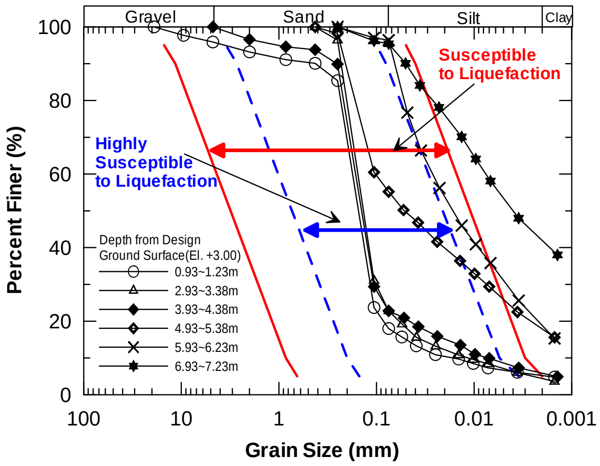

Fig.2 Grain size distribution of soil at test site

3 Site description and test setup

Two full-scale tests were carried out to study the behavior of piles and pipelines subjected to lateral spreading. The UCSD experiments were located in a zone of the test area where large global translation of the soil was expected. The following sections describe the details of the test site, test set up, and test specimens.

3.1 First Lateral Spreading Experiment

A layout of the test site for the first test is shown in Fig. 3. The test site was approximately 25 m wide by 100 m long. The front end was bordered by a waterway. 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 a series of tied rods which were fixed to H-piles to prevent movement of quay wall. The ground surface was at the elevation of +3.00 m at the quay wall and started to gently slope upwards at 25.2 m away from the quay wall with a slope of 4%. The test site was surrounded by sheet piles to tip elevations between -5.00 m and -8.00 m.

The UCSD single pile was located 19.0 m away from the quay wall. The pile diameter was 318 mm with a wall thickness of 10.5 mm, a nominal length of 11.5 m, and the yield strength of 400 MPa. Three similar free-head single piles were installed in the area by Waseda University (WU). The cross sections of the WU piles were the same as that of the UCSD pile, but the degree of fixity at the pile tip was different. The UCSD pile was driven full length into the ground corresponding to approximately 3.0 to 3.5 m penetration into the dense soils to obtain the fixity at the pile tips. This was to ensure that the movement of the soil due to lateral spreading could produce bending moments along the pile and hence allow quantifying the distribution of the soil pressure acting on the pile from strain gauge data. Conversely, the WU piles were driven into the ground to depths approximately 7.5 m to 8 m below the surface with their tips sitting just above the dense soils. The degree of fixity of these piles at the tips was therefore negligible.

In addition to pile specimens, pipelines oriented in transverse and longitudinal directions were installed. The objective of the test for transverse pipelines was to assess the pipeline performance subjected to bending due to the global translation of the soil, while the test for the longitudinal pipeline was to evaluate the pipeline performance subjected to axial frictional forces imposed by the soil moving relative to the pipeline.The pipelines in this study included two natural gas pipelines and one electrical conduit. 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 an 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. The transverse pipelines were installed by first excavating the ground and setting them on the compacted layer of fill material with a thickness of 20 cm. Then, both ends were anchored to the sheet pile wall using high strength bolts. This type of connection allowed some rotation at each end of the pipeline. Subsequently, the sand was backfilled with multiple compacted layers in accordance with Japanese Gas Association specifi-cations to achieve a compact dry unit weight of 90% of the maximum dry unit weight determined in the laboratory using the standard Proctor test ASTM D-698 (ASTM, 1998). The longitudinal gas pipeline was installed in the same way as the transverse pipelines but only one end was anchored to the sheet pile as shown in Fig.3.

Due to the success in using controlled blasting to induce soil liquefaction for the full-scale lateral load tests at Treasure Island (Ashford *et al.*, 2000), as well as the liquefaction testing at a saturated loess site in China (Wang *et al.*, 2002), the same technique was implemented at the test site to liquefy the soil, and thus induce lateral spreading. The blast holes were spaced at 6.0 m on center in the square grid pattern. The charges were installed at depths of 3.5 m and 7.5 m below the design ground surface (EL +3.00 m). The amount of charges varied from 2 kg nearby the pile specimens to 3 to 5 kg at other areas. The charge was reduced near the pile specimens to prevent damage to a large number of instruments installed in the vicinity. The sequence of the primary blasting

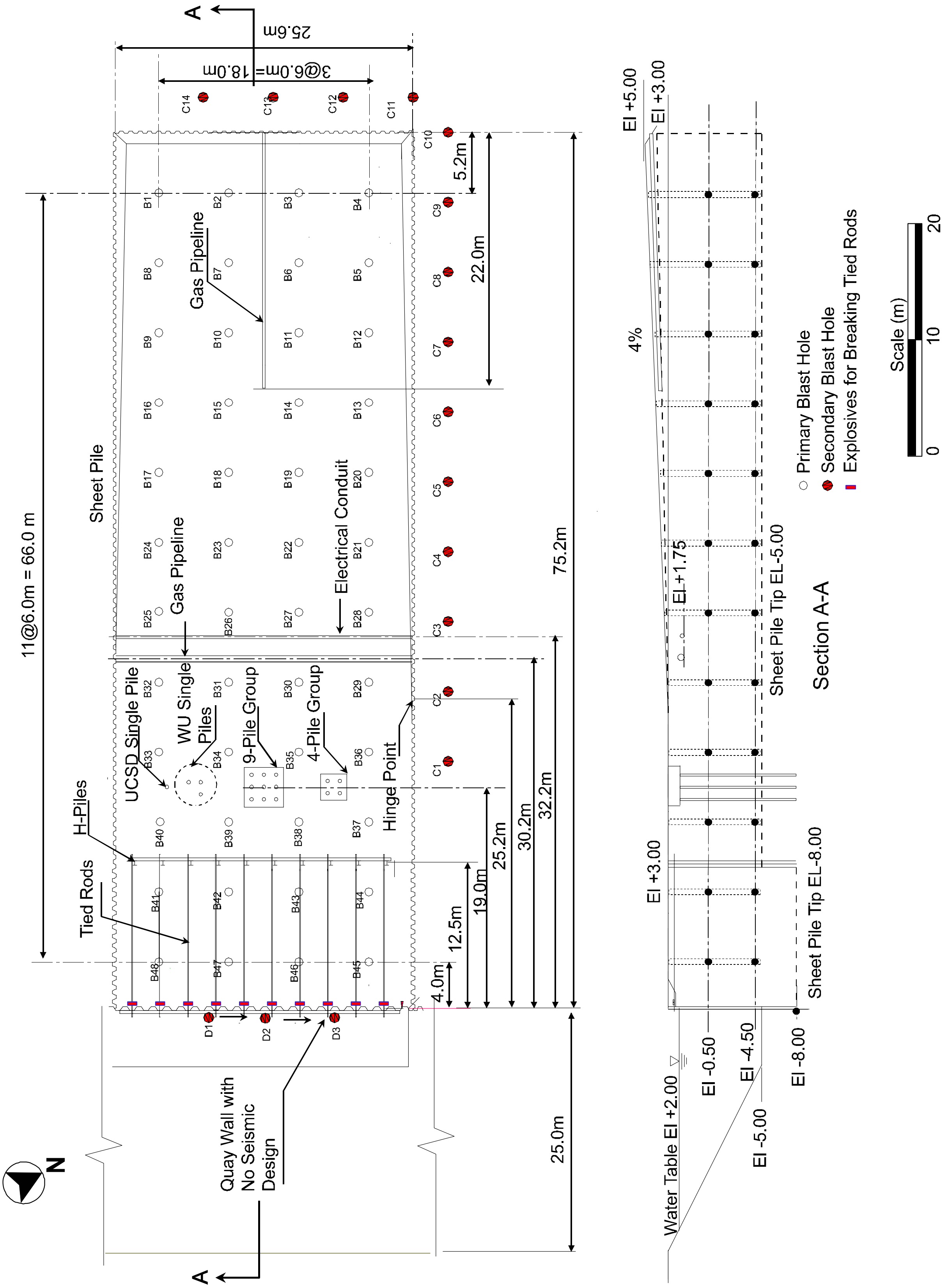

Fig. 3 Site layout for 1st lateral spreading test

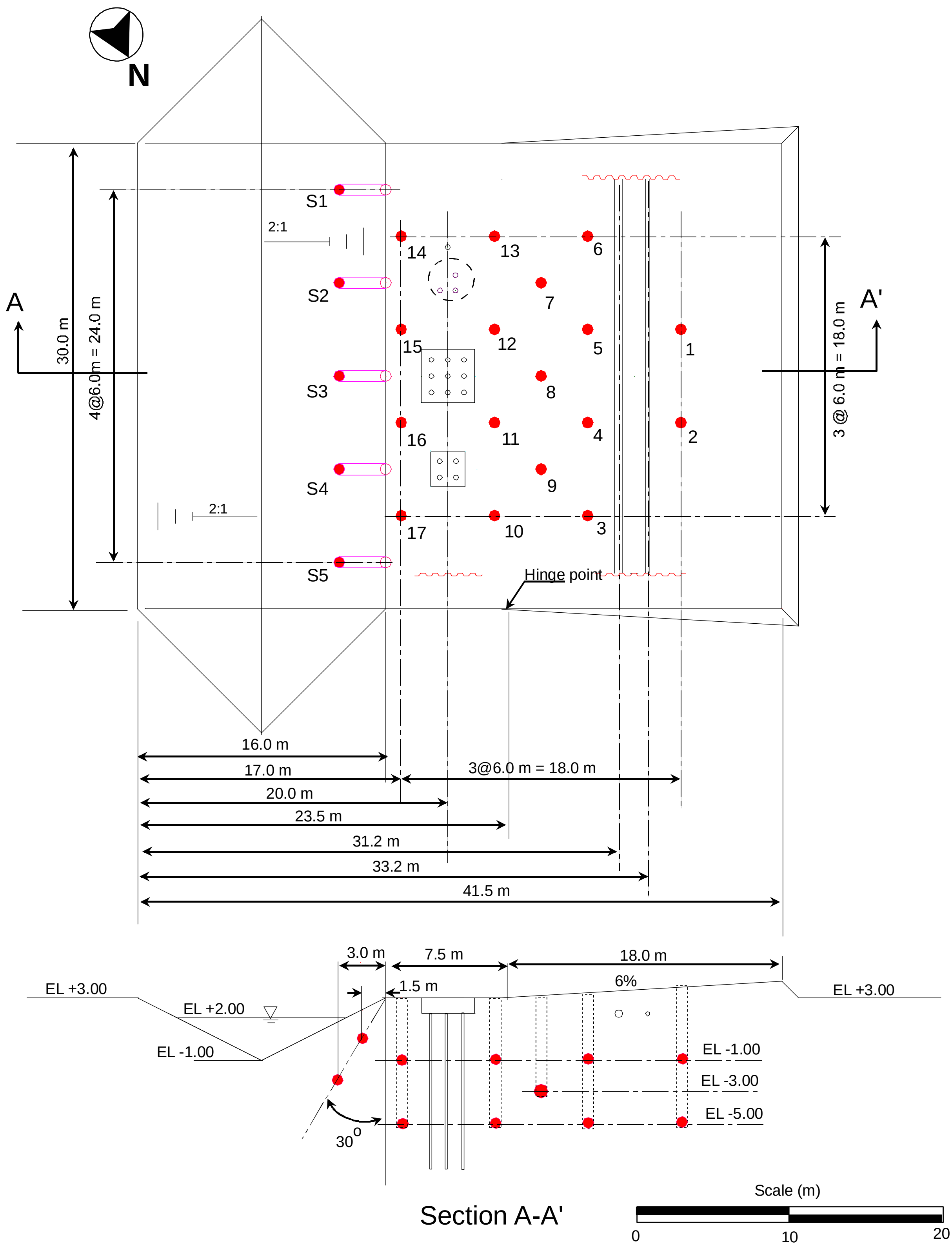

Fig. 4 Site layout for 2nd lateral spreading test

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 B45) as illustrated in Fig. 3. The blasting interval between two adjacent boreholes was approximately 0.7 seconds. Following the primary blasting was the detonation of secondary blast holes around the perimeter of the test site with a time interval between each blast hole of 1 second. The purpose of these explosives was to loosen the soil in the vicinity of the sheet pile to allow unrestricted flow of soil in these regions. Approximately 20 seconds after the completion of the secondary blasting, additional explosives were used to break the tied rods of the quay wall, which allowed the quay wall to move freely to create additional movement of the soil within the test area.

3.2 Second Lateral Spreading Experiment

The second lateral spreading test was carried out with the same test specimens and instrumentation still in place. The test was performed with an attempt to induce additional ground deformations and further evaluate the performance of the piles and pipelines subjected to a higher level of soil deformation. The test site for the second lateral spreading test was significantly modified from the first one as presented in Fig. 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 an elevation of -1.00 m with a slope of 2:1. However, it was observed that the actual slope was steeper than the design drawing with a slope of approximately 1:1. The water was then filled to an elevation of +2.00 m but the actual ground water table during the second test observed from the soil excavation adjacent to the test area was approximately at an elevation of +1.00 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.

Most blast holes were spaced at 6.0 m on center in a square grid pattern. Charges were installed at depths of 4.0 m and 8.0 m below the design ground surface (El +3.00 m). The size of charge 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 charge sizes 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 to increase the potential for down slope movements. The others were located between the pipelines and piles as denoted as blast holes No. 7 to No. 9 where 3 kg of explosives were installed at El. -3.00 m. The explosives on the steep slopes (S1 to S5) were detonated first. Approximately 15 second later, the primary sequence of the blasting was started. The primary blast began at blast hole No.1 on the rear of the embankment and proceeded sequentially to No.17.

The weather for the second lateral spreading experiment was quite poor, with heavy snowfall of about 0.50 m and wind speeds of 100 kph on the test day. The ground was frozen down to a depth of approximately 0.20 cm to 0.30 cm below the ground surface at 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 the test specimens.

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. Steel channels C 75 mm x 6.92 kg/m with yield strengths 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 pipelines were spaced between 1.0 m and 3.0 m along the top and the side of the pipelines to measure the bending moment due to both horizontal and vertical movement, respectively.

In addition to the strain gauges, other instrumentation was 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, and Global Positioning System (GPS) units. Layout of instrumentation for the first experiment is presented in Fig. 5. 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 Excess Pore Water Pressure

Sand boils forming at the ground surface (Fig. 6) 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 blast effect on the pore-pressure.

Typical examples of the observed excess pore pressure time-histories at different locations and multiple depths are presented in Fig.7. These transducers were located near the single pile, transverse pipelines, and longitudinal gas pipeline. The excess pore water pressure increased as the depth increased. The time required for the excess pore water pressure to reach its maximum value depended

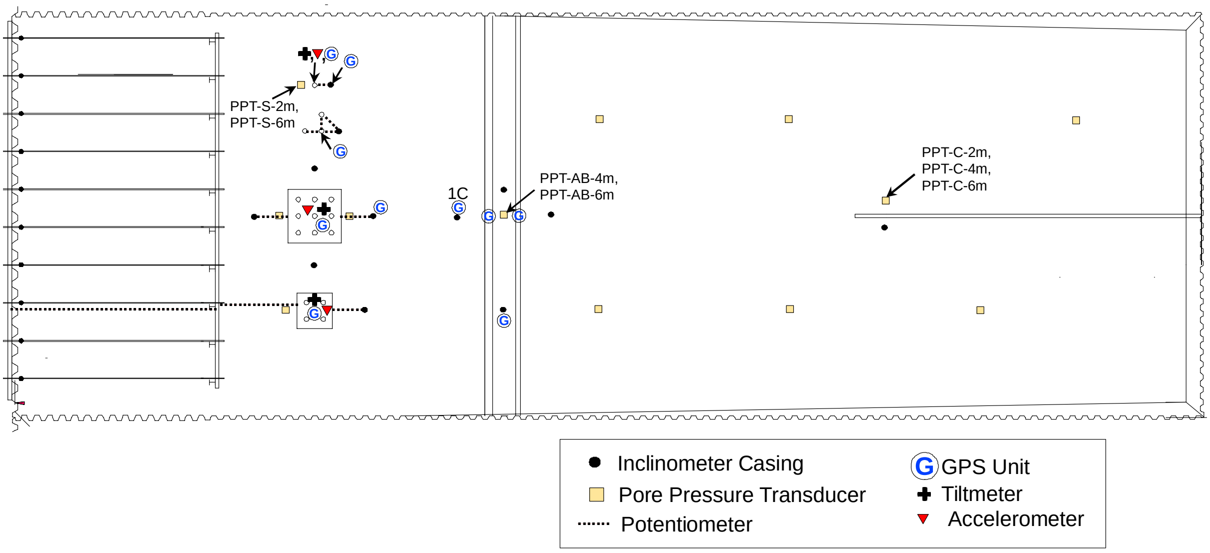

Fig. 5 Instrumentation plan for 1st lateral spreading test

upon the distance between the transducer and the first blast hole. In this case, the soil in the vicinity of the longitudinal gas pipeline liquefied first, while the soil near the single pile liquefied afterward. Fluctuation of pore pressure was obvious as the blasting occurred in the vicinity of the transducer location. The evidence of increased excess pore water pressure at about 40 seconds and 86 seconds was due to the effect of secondary blasting and blasting of tied rods, respectively.

Fig. 6 Sand boil after 1st test

Based on the measured excess pore pressure, the excess pore pressure ratio, R_u, at each location was calculated ($R_u = \dfrac{\Delta u}{\sigma_v'}$, where Δu is excess pore water pressure and σ_v' is the vertical effective stress). Fig. 8 presents the time-history of R_u at several locations. The results show that most of the soil in the vicinity of

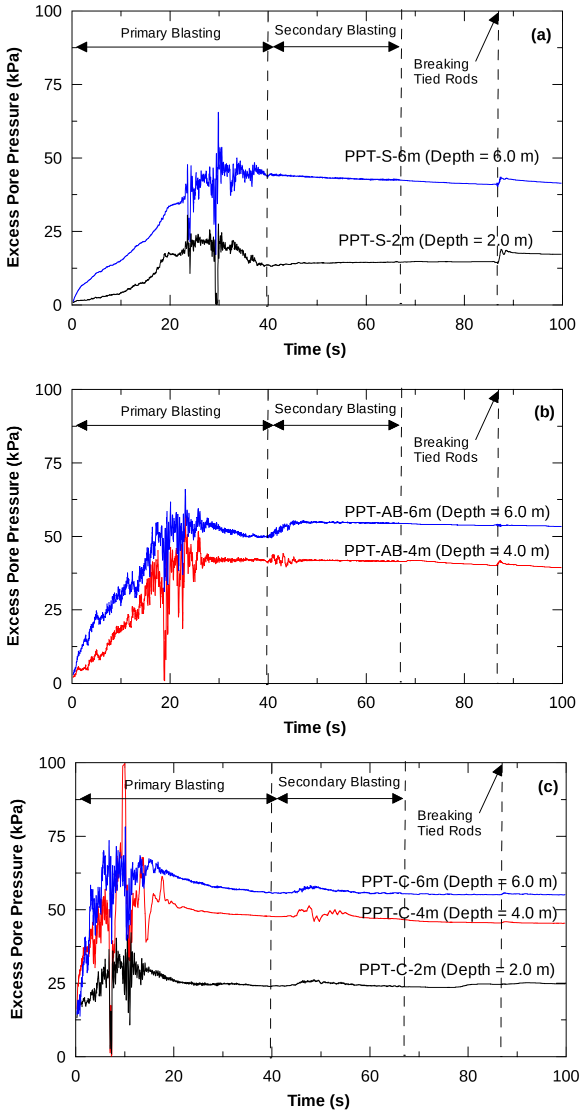

Fig. 7 Excess pore pressure vs. time nearby (a) single pile; (b) transverse pipelines; (c) longitudinal pipeline

the single pile and pipelines was liquefied as indicated by the maximum R_u exceeding 100%. The ratio was dropped below 100% at the end of primary blasting and proceeded to dissipate with time. Though some transducers show that the excess pore water pressure ratios were slightly above 100% at the end of the blasting, this may be due to: 1) some error in estimating the soil unit weight and depth of water table, or 2) some of the transducers might have moved downward during and after the blasting, resulting in an increase of R_u. In summary, R_u in the region of the pile and pipeline specimens at the end of the primary blasting ranged from 65% to slightly over 100%.

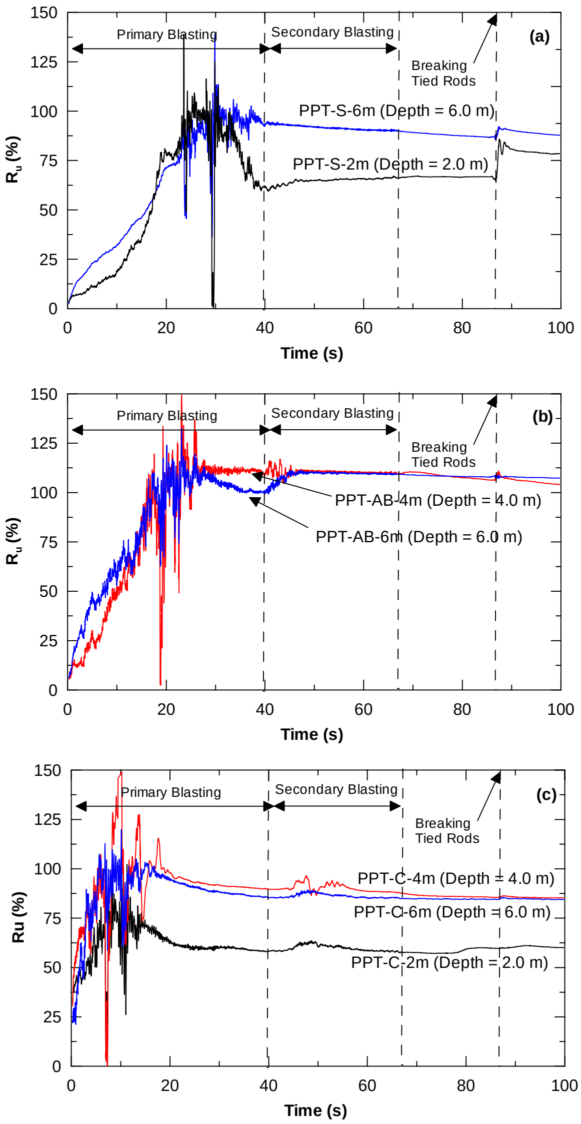

Fig. 8 Excess pore pressure ratio vs. time nearby; (a) single pile; (b) transverse pipelines; (c) longitudinal pipeline

The R_u in the second test appeared to be significantly less than those measured during the first test with values at the end of the blast ranging between 30% and 80%. Though some water was coming out from the blast hole after the blast stopped, no sand boils were observed in the second test. The lower R_u measured in the second test was because the soil was less susceptible to liquefaction for two reasons. First, the soil in the second test was denser because some settlement after liquefaction in the first test took place with the magnitude approximately 20 cm to 60 cm. Second, the ground water table in the second test was lower than that observed in the first test.

5.2 Deformations of Ground and Lifelines

GPS units were used to monitor the movements of both ground and test specimens during lateral spreading. The measurements in the vicinity of piles and transverse pipeline specimens were conducted by a research team from Caltrans (Turner, 2002), while those in the embankment area were carried out by WU (Takahashi, 2002b)

A typical example of time history of soil movements down slope of the transverse pipelines (denoted as unit 1C as shown Figure 5) in longitudinal, transverse, and vertical directions is presented in Fig.9a, together with the response of R_u in the same vicinity. The movements of GPS units were observed 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 forces from the blasting was not important as indicated by the insignificant movements in transverse and vertical directions. The longitudinal movement observed 27 seconds after the soil was liquefied (i.e., R_u reached 100%) was therefore primarily due to liquefaction-induced lateral spreading. Fig. 9b presents the displacement path of the soil in the horizontal plane showing that the horizontal movement mainly occurred in the longitudinal direction towards the quay wall.

Based on the GPS data (Turner, 2002; Takahashi, 2002b) and survey data provided by Sato Kogyo Co. and Tobishima Co. (Sato Kogyo, 2002), the displacement vectors in the horizontal plane for the first and second tests were plotted as presented in Figure 10.

5.2.1 First Test

The displacement vectors in the horizontal plane for the first test are presented in Fig.10a. The soil in the vicinity of the embankment moved considerably in the transverse direction as opposed to the expected flow direction. This is because the lateral confinement in the transverse direction was lower than that in the longitudinal direction. The displacements of the soil were mainly in the longitudinal direction in the level ground area with the soil movement being quite uniform in the vicinity of the test specimens. The average soil movement in this vicinity was approximately 30 cm. Beyond the range of pile specimens, the soil displacement continued to increase towards the quay wall where the maximum movement over 1 m had occurred.

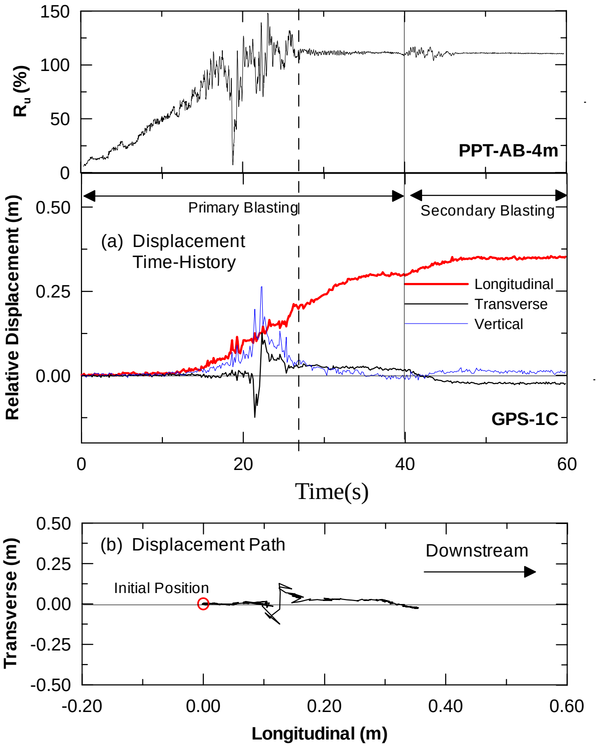

Fig. 9 Example of GPS data of unit GPS-1C

The longitudinal pipeline moved transversely 33 cm at the free end of the pipeline. The movements of both transverse pipelines were similar. The maximum movements of 35 cm occurred at the middle of both pipelines.The displacements of both transverse pipelines were slightly lower than the soil movement on the upstream side (35 to 40 cm). This is because both pipelines tended to impede the soil flow.

The UCSD single pile appeared to move with the same magnitude of the free field soil movement with a pile head displacement of 32 cm. The average horizontal displacement of the WU piles was, however, significantly higher than those measured for the UCSD single pile with the magnitude of average displacement approximately 43 cm. This was due to the fact that the tips of the WU piles were located just above the dense layer, while the UCSD pile penetrated about 3.5 meters into dense soil. The WU piles, therefore, likely behaved as rigid piles, where 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. Therefore, due to the effect of fixity at the pile tip, the displacement at the pile head of the UCSD single pile was less than those of the WU piles.

5.2.2 Second Test

Fig. 10b presents the horizontal displacement vectors of the second test. The horizontal soil movements on the upstream side that occurred in the second test were significantly lower than in the first test, with an average value of 15 cm, because the soil conditions in the second test were less susceptible to liquefaction as described in the previous section. Similar to the first test, the magnitude of soil movement increased as the location moved towards the waterway. The maximum soil movement was observed between both pile groups with a magnitude of 46 cm. However, it should be noted that 10 cm of this 46 cm of soil movement contributed to the failure of the slope immediately after the first set of the blast at the slope toe.

The movement of the single pile was 28 cm, while the WU piles moved significantly more, with the magnitude of 39 cm due to the lack of fixity at the pile tips. The movement of the gas pipeline and electrical conduit were about 50% of the first test.

5.3 Responses of Single Piles and Pipelines

5.3.1 Single Piles

Moment distribution along the length of the UCSD single pile at the end of the first and second blast is presented in Fig. 11. As expected for the case of the free-head pile, the test results indicate that the moments at the pile head were zero. Interestingly, the moment was insignificant for the first 4 m of a very loose liquefied sand layer, indicating that the resultant force on the pile produced by the liquefied soil was negligible. However, the moment increased with depth for the next 3.5 m, where a very soft clay layer existed. Though the data indicates that the excess pore water pressure ratio in this layer also reached 100%, the clay layer appeared to behave differently from liquefied sand in which it applied some pressure to the pile as indicated by the increase in the pile moment. The maximum moment occurred in a dense soil layer below the bottom of the liquefied soil layer, at a depth of about 9 m below the ground surface. This dense soil layer resisted the bending action produced by the lateral soil pressure from global translation. Based on the moment data, the single pile yielded after the second test with a plastic hinge length of more than 1 m. It should be noted that the moments along WU piles were negligible compared to the UCSD single pile (Takahashi, 2002a). This is due to the difference in degree of fixity at the pile tip.

5.3.2 Pipelines

The strain distributions along the transverse pipelines for the first and second tests are presented in Fig.12. It should be noted that the strain of the second test presented herein is the cumulative strain obtained from both the first and second tests. The strain data of pipelines was somewhat irregular because they were subjected to non-uniform soil pressure along their entire length, produced by compression waves

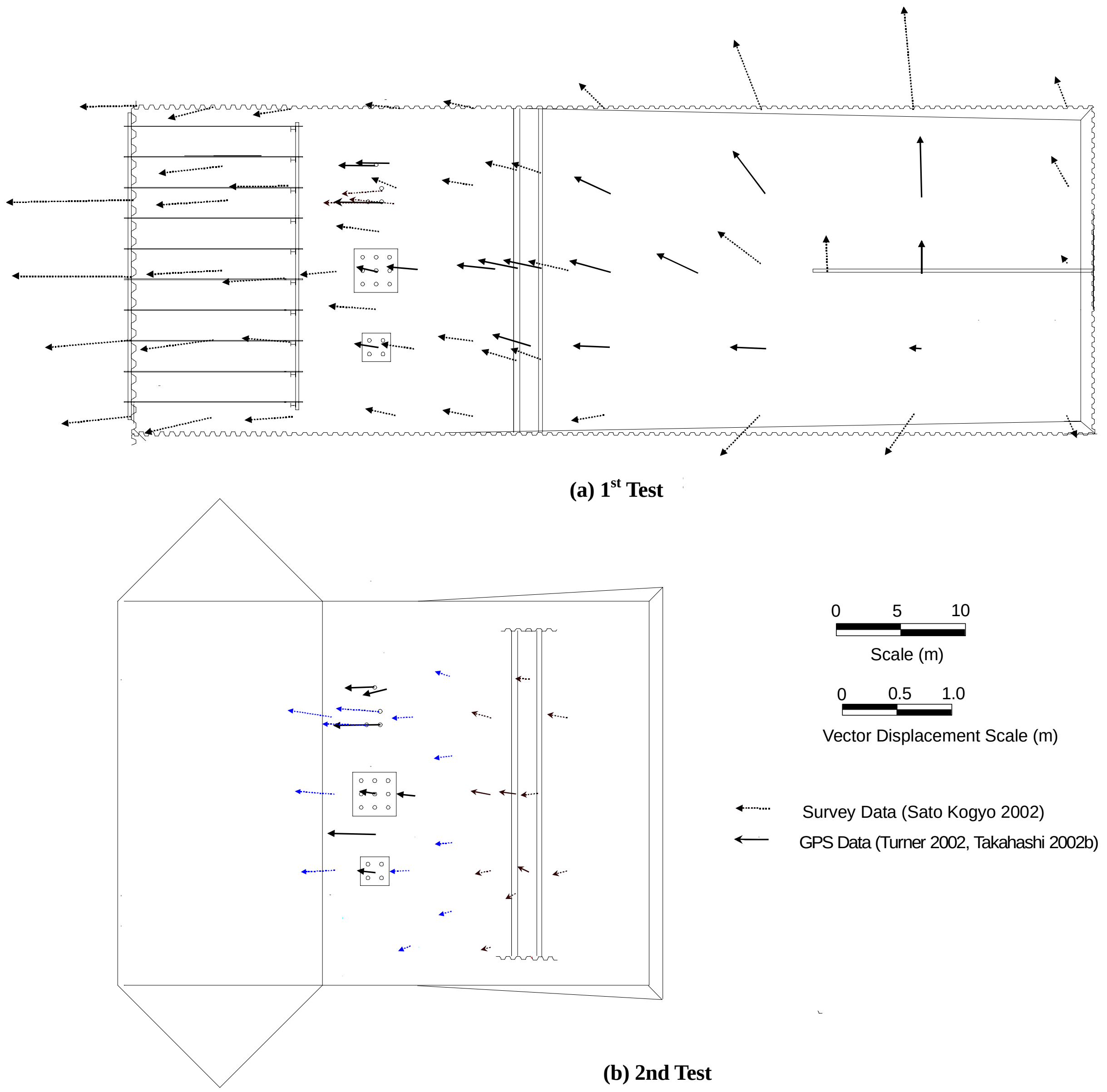

Fig. 10 Horizontal displacement vectors for (a) 1st test; (b) 2nd test

from the blasting. The strain distribution along the side of the electrical conduit was smaller than the gas pipeline. The reason is that for the same pipeline curvature distribution (i.e., both pipelines experienced the same movement as shown in Fig.10), the larger diameter pipeline produces larger strain. Strain data also shows that both pipelines performed relatively well without yielding.

The strain distribution along side of the longitudinal gas pipeline is shown in Fig.13a. Initially, it was aimed at measuring the axial strain along the pipeline due to the axial frictional forces imposed by the soil movement relative to the pipeline. Theoretically, if the soil moves parallel to the direction of the pipeline, the maximum strain should occur at the support and gradually decrease to zero at the end of the pipeline. However, the measured strain distribution shows that the maximum moment occurred at the middle of the pipelines. This is because the soil movement produced by the blasting caused the pipeline to move significantly in the transverse direction compared to the longitudinal direction as shown by the survey data in the previous section (Fig.10). Small strain observed in the vicinity close to the support indicates that the frictional forces imposed by the soil movement in this case was negligible and would not cause damage to the pipeline. However, the larger amount of strain along the top and bottom of the pipeline due to settlement was noticed as shown in Fig.13b. The symmetry of strain gauge data

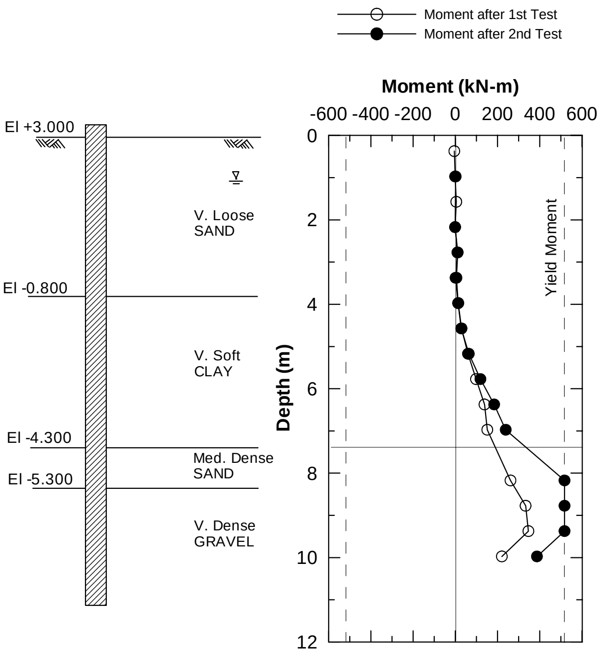

Fig. 11 Moment distribution along UCSD single pile

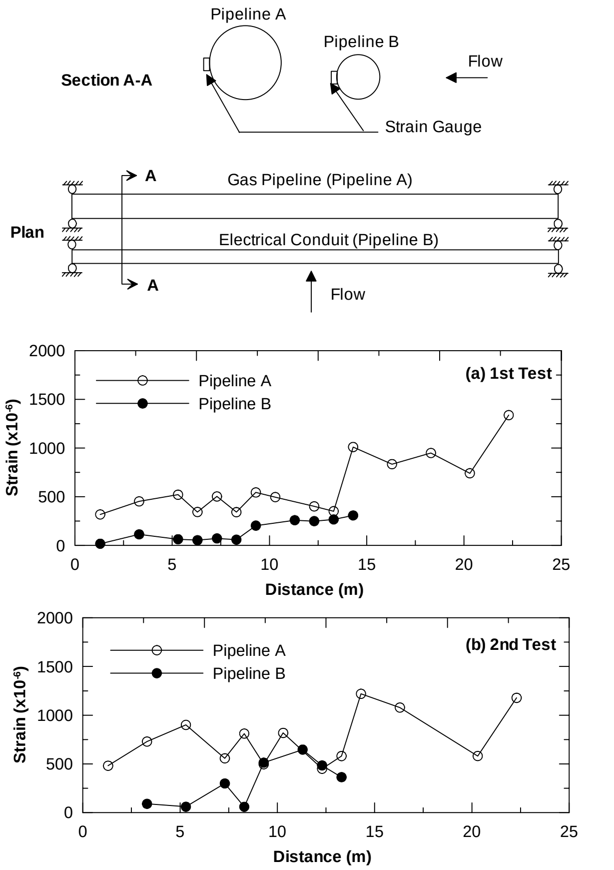

Fig. 12 Strain distribution along transverse pipelines; (a) 1st test; (b) 2nd test

along the top and bottom indicated the consistency of data. This bending strain, due to the soil settlement, appeared to be more important than that due to the frictional forces, and therefore, should be considered in the design.

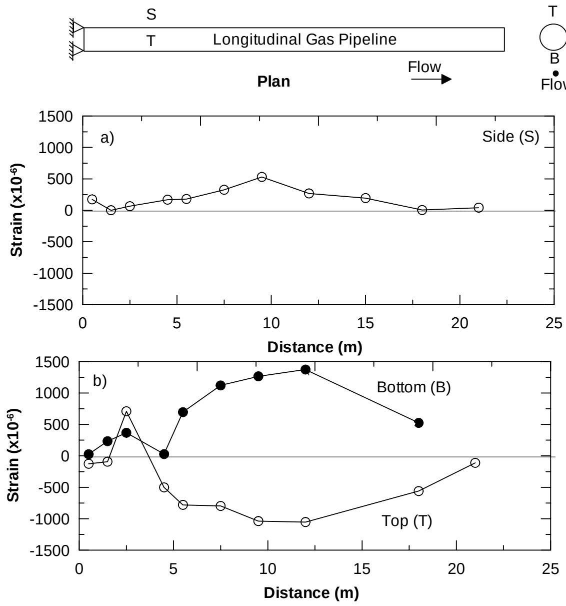

Fig. 13 Strain distribution along longitudinal gas pipeline for 1st test; (a) side strain gauge; (b) top and bottom strain gauges

6 Conclusions

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

(1) Controlled blasting successfully liquefied the soil and induced lateral spreading.

(2) The excess pore water pressure ratios varied between 65% and slightly over 100% for the first test. The degree of liquefaction in the second test was lower than the first one with excess pore pressure ratios ranging between 30% and 80%.This is probably because the soil conditions in the second test were less susceptible to liquefaction.

(3) The average soil movement on the upstream side in the second test was approximately 50% of the first test. The movements increased as the location moved towards the quay wall, where the soil movement was the highest.

(4) Increasing the degree of fixity at the pile tip decreased the pile head displacement but increased the moment along the pile. The UCSD single pile, which had a higher degree of fixity at the pile tip, yielded at the end of the second test, while no yielding was observed for the WU single piles.

(5) Observed moment distribution of the UCSD single pile indicated that global translation of the liquefied soil layer provided insignificant force to the pile.

(6) Both transverse and longitudinal pipelines performed well without yielding.

7 Acknowledgement

The authors wish to express their sincere gratitude to the sponsors of the Performance of Lifelines Subjected to Lateral Spreading project, the PEER Lifelines Program with support from Caltrans, Pacific Gas & Electric and the California Energy Commission.

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 in this test, as well as effectively managing the project is gratefully acknowledged. Dr. Akio Abe (Tokyo Soil Research Co.) is acknowledged for providing the test specimens and facilitating the instrumentation work in Japan.

Loren Turner, Cliff Roblee and Tom Shantz from Caltrans' Division of New Technology and Research are gratefully acknowledged for their help in monitoring the movements of soil and lifelines using GPS equipment.

8 References

Abdoun TH, Dobry R, O'Rourke TD and Chaudhuri D (1996), "Centrifuge Modeling of Seismically-induced Lateral Deformation During Liquefaction and Its Effect on a Pile Foundation," *Proc. Sixth Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Soil Liquefaction, Technical Report NCEER-96-0012*, NCEER, SUNY-Baffalo, NY, pp. 525-539.

Ashford SA, Rollins KM, Bradford SC, Weaver TJ and Baez JI (2000), "Liquefaction Mitigation Using Stone Columns Around Deep Foundations: Full-scale Test Results," *Soil Mechanics 2000, Transportation Research Record No. 1736*, TRB, Washington D.C., pp.110-118.

ASTM Standard (1998), *Annual Book of ASTM Standards*, Vol. 04.09, West Conshohocken, PA, pp. 217-227.

Bartlett SF and Youd TL (1992), "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., Hamada M. and O'Rourke TD(eds.), August 17.

Benuzca L (1990), "Loma Prieta Earthquake Reconnaissance Report", *Report by EERI and NRC, Supplement to Vol.6 of Earthquake Spectra*, May, 448 p.

Dobry R and Abdoun TH (2001), "Recent Studies on Seismic Centrifuge Modeling on Liquefaction and Its Effect on Deep Foundations," *State-of-the-Art Paper, Proc. 4th Intl. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics and Symposium to Honor Prof. W.D.L. Finn* (*S. Prakash, ed.*), San Diego, CA, March 26-31, SOAP-3, Vol. 2., pp.1-30.

Hamada M (1992a), "Large Ground Deformations and Their Effects on Lifelines: 1964 Niigata Earthquake," *Ch.3 of Case Studies of Liquefaction and Lifeline Performance during Past Earthquakes*, *Report No. NCEER-92-0001*, Vol.1, (Hamada and O'Rourke, eds.) National Center for Earthquake Engineering Research, Buffalo, N.Y, pp. 3-1 to 3-123.

Hamada M and O'Rourke T (1992b), "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.

Hamada M (2000), "Performance of Foundations Against Liquefaction-induced Permanent Ground Displacement," *Proc. 12th World conference on Earthquake Engineering*, Auckland, New Zealand, Paper No. 1754.

Japanese Geotechnical Society (1995), *Method for Swedish Weight Sounding Test (JIS A 1221-1995), Ground Survey Method*, pp.213-220 (in Japanese)

Meneses J, Hamada M, Kurita M and Elgamal A (2002), "Soil-pile Interaction Under Liquefied Sand Flow in 1g Shake Table Tests," *Proceedings, International Conference on Advances and New Challenges in Earthquake Engineering Research*, August 15-20, Harbin and Hong Kong, China.

O'Rouke TD (1996), "Lessons Learned for Lifeline Engineering from Major Urban Earthquakes," *Proceedings, Eleventh World Conference on Earthquake Engineering*, Elsevier Science Ltd., 18 p.

Port and Harbor Research Institute (1997), *Handbook on Liquefaction Remediation of Reclaimed Land*, A.A. Balkema, Rotterdam, Brookfield.

Ramos R, Abdoun TH and Dobry R. (2000), "Effect of Lateral Stiffness of Super Structure on Bending Moments of Pile Foundation Due to Liquefaction-induced Lateral Spreading," *Proc. 12th World conference on Earthquake Engineering*, Auckland, New Zealand, Paper No. 0902.

Sato Kogyo Co. (2002), (Personal Communication), Japan.

Takahashi Y (2002a), "Consideration of the Measured Action of Piles," *Internal workshop on Full-Scale Lateral Spreading Tests*, Japan, June 24-25

Takahashi Y (2002b), "A Result of the Ground Surface Displacement Measured by GPS and an Investigation on the Flowing Characteristics of the Liquefaction Soil," *Internal workshop on Full-Scale Lateral Spreading Tests*, Japan, June 24-25.

Tokida K, Iwasaki H, Matsumoto H and Hamasa T (1993), "Liquefaction Potential and Drag Force Acting on Piles in Flowing Soils," *Soil Dynamic and Earth-*

quake Engineering, Computational Mechanics, South Hampton, England, pp. 349-364.

Turner LL (2002), "Application of Real-Time Kinematic Global Positioning System to Measure Liquefaction-Induced Ground Surface Deformations," *thesis* presented to University of California, Davis, In partial satisfaction of the requirements for the degree of Master of Science.

Wang L, He K, Shi Y and Wang J (2002), "Study on Liquefaction of Saturated Loess by In-situ Explosion Test," *Journal of Earthquake Engineering and Engineering Vibration*, **1**(1): 50-56

Wilson DW, Boulanger RW and Kutter BL (2000), "Observed Seismic Lateral Resistance of Liquefying Sand," *J. Geotechnical and Geoenvironmental Engineering*, ASCE, **126**(10): 898-906.

Youd TL, Rollins KM, Salazar AF and Wallace RM (1992), "Bridge Damage Caused by Liquefaction During the 22 April 1991 Costa Rica Earthquake," *Proc. Tenth World Conference on Earthquake Engineering*, Madrid, Spain, July 19-24, pp. 153-158