13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1822

PERFORMANCE AND ANALYSIS OF A LATERALLY LOADED PILE IN STONE COLUMN IMPROVED GROUND

Thomas WEAVER[1], Scott ASHFORD[2], Kyle ROLLINS[3]

SUMMARY

Results of full-scale lateral load tests performed on a 0.6-m diameter cast-in-steel-shell (CISS) pile in liquefied sand and stone column improved ground are presented. Cone penetration test results show up to 5 times greater tip resistance after stone column installation. The lateral stiffness of the soil-pile system for the stone column improved ground increased by approximately 44% under static load conditions and approximately 350% when compared to the pile head response after pore water pressures were increased by detonation of down-hole explosives. Analyses show a poor correlation between measured and predicted pile response for the static load case after installation of stone columns. However, the analyses compare reasonably with the measured pile response after detonation of the down-hole explosives. Variation of the calculated pile response resulting from variation in the estimated soil properties was quantified and found to be negligible at this site.

INTRODUCTION

The lateral response of deep foundations is of great concern in seismically active regions. Many deep foundations at bridge sites penetrate through potentially liquefiable soils in these regions. As a result of liquefaction, the lateral load capacity of these foundations may reduce significantly during an earthquake. Observation of reduced lateral soil resistance from centrifuge model tests and full-scale load tests have previously been reported [1, 2]. One option to increase the lateral capacity of deep foundations in a liquefiable soil is to mitigate the potential for liquefaction by installing stone columns. This paper presents results from a series of full-scale lateral load tests on a 0.6-m diameter cast-in-steel-shell (CISS) pile. The lateral load tests were performed at Treasure Island in San Francisco, California. The tests included static tests in non-liquefied soil, cyclic tests in blast induced liquefied sand, and a test in stone column improved soil where down-hole explosives were detonated in an effort to increase pore water pressures prior to application of lateral loads. Results from each test are presented to illustrate the improved lateral load capacity of piles in stone column improved ground. Lateral load analyses for the CISS pile in stone column improved ground were performed and are compared with the full-scale test results.

[1] Assistant Prof., Dept. of Civil Eng., Univ. of Idaho, Moscow, ID, USA. Email: tweaver@uidaho.edu
[2] Associate Professor, Dept. of Struct. Eng., Univ. of California San Diego, La Jolla, CA, USA
[3] Professor, Dept. of Civil and Env. Eng., Brigham Young University, Provo, Utah, USA

SITE CONDITIONS

A subsurface investigation at the test site was performed prior to foundation installation and after installation of stone columns. The investigation prior to foundation installation consisted of one soil boring and six cone penetration test (CPT) soundings. The site investigation after installation of the stone columns consisted of ten CPT soundings.

The soil profile shown in Figure 1 consists of medium dense, poorly graded sand at the ground surface (elevation +3.4 m), approximately 1.5 m thick, underlain by loose, poorly graded sand with intermittent clay zones to an elevation of -10.4 m (approximately 12.3 m thick). The loose sand is underlain by Young Bay Mud and extends to the bottom of the borehole. Ground water was encountered at an elevation of +1.9 m (1.5 m below the ground surface) at the time of the boring excavation.

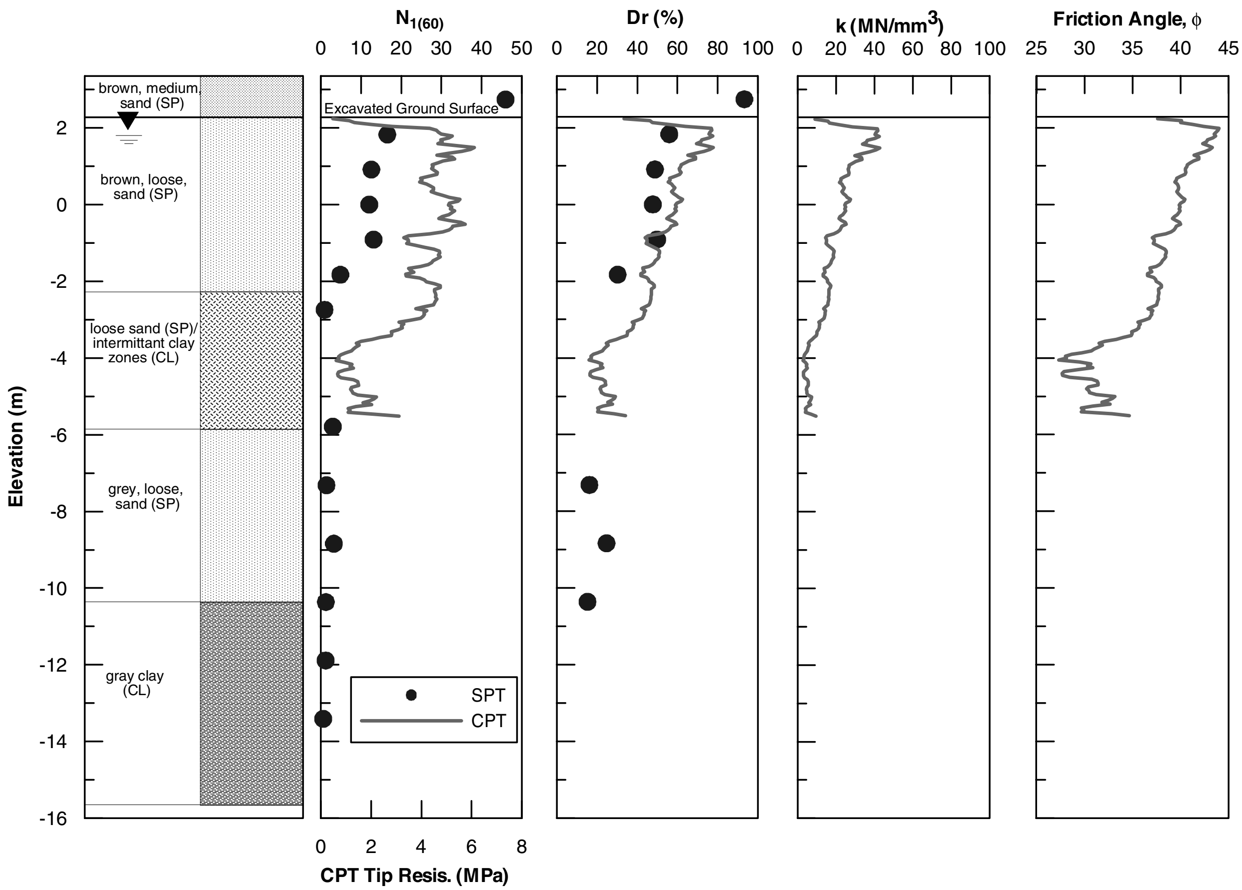


Figure 1 Soil Profile, SPT Data, CPT Tip Resistance, and Interpreted Soil Properties with Depth Before Stone Column Installation

Before Stone Column Installation

Standard Penetration Test (SPT) results and typical CPT sounding data prior to foundation and stone column installation are shown in Figure 1. The SPT results are shown as $(N_1)_{60}$ values, having been corrected for field procedures and overburden. Correlations presented by Kulhawy and Mayne [3] were used to estimate the friction angle and relative density from CPT tip resistance and $(N_1)_{60}$ values. In addition, a correlation proposed by Peck, Hanson, and Thornburn [4] was used to estimate friction angle from $(N_1)_{60}$ values. A significant difference in friction angle is observed between the two correlations.

However, the correlations using the SPT and cone penetration data to estimate relative density compare favorably. The constant of horizontal subgrade reaction, k, was estimated using a correlation proposed by the American Petroleum Institute [5].

After Stone Column Installation

Average tip resistance values from the CPT soundings near the 0.6-m diameter CISS pile after installation of the stone columns are shown in Figure 2. As anticipated, these results show a significant increase in tip resistance compared to tip resistance values measured prior to stone column installation. Interpreted soil properties after installation of the stone columns were also estimated using the CPT data and correlations presented by Kulhawy and Mayne [3]. Estimated relative density, friction angle, and constant of horizontal subgrade reaction obtained using CPT data before and after stone column installation are compared in Figure 2.

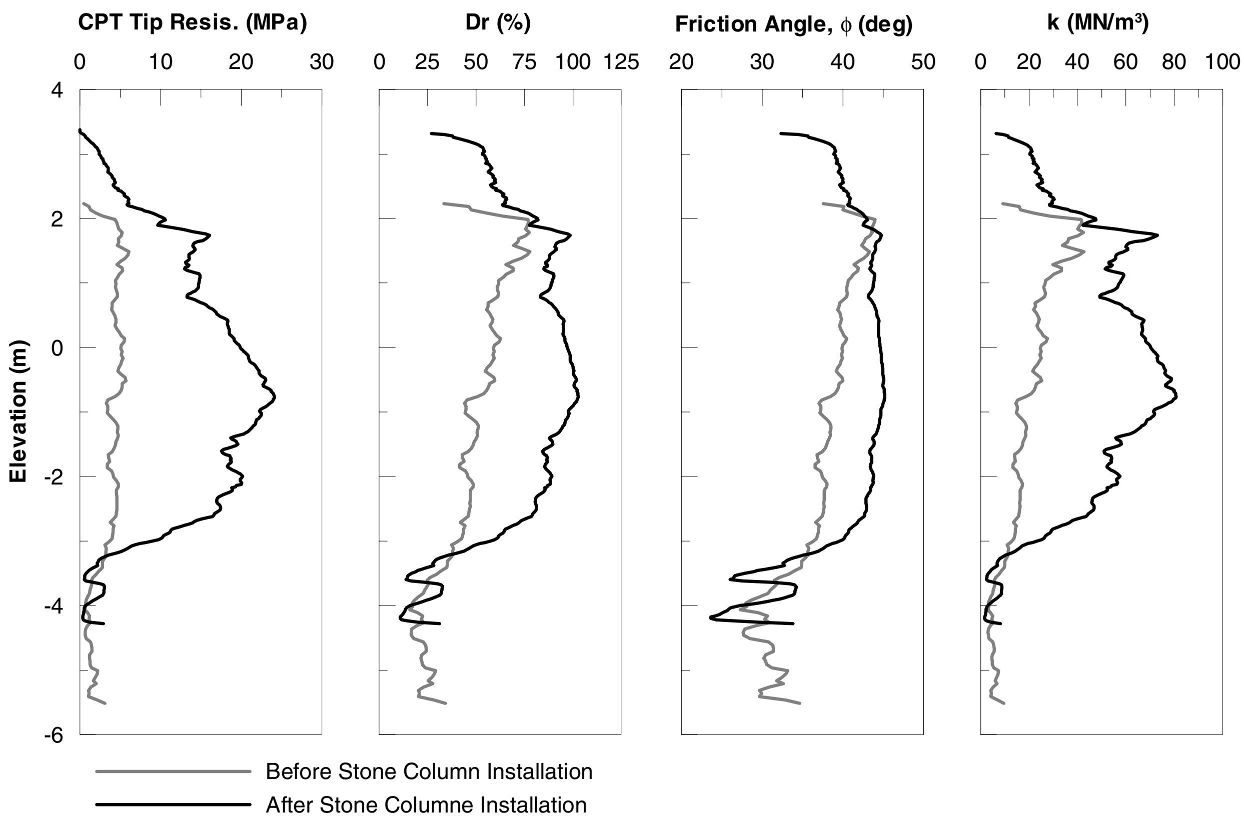


Figure 2 CPT Tip Resistance and Interpreted Soil Properties with Depth Before and After Stone Column Installation

TEST SETUP

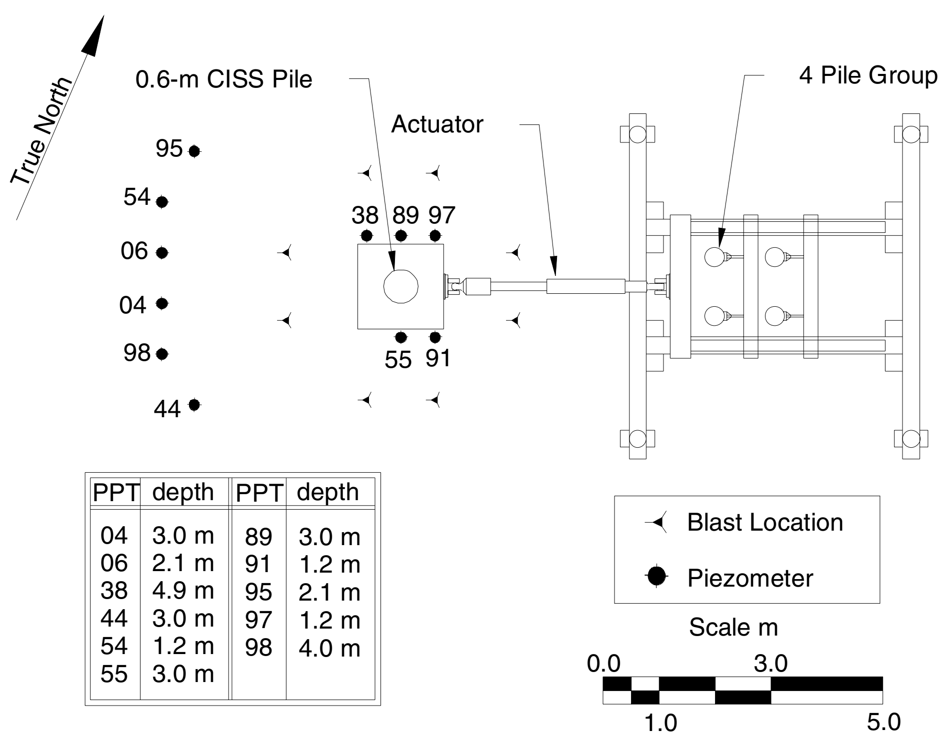
The test site at Treasure Island consisted of an excavation with plan dimensions of approximately 16 m by 22 m and 1 m in depth. The CISS pile was installed near the center of the excavation. A group of four closely spaced piles were also installed, and a hydraulic actuator was placed approximately 1 m above the excavated ground surface between the two foundation systems to apply the lateral loads (Figure 3). The CISS pile installation consisted of driving a 0.6-m outside diameter steel pipe having a wall thickness of 13 mm to an elevation of -11.5 m. Soil inside the steel shell was drilled out to allow for placement of a steel reinforcing cage and concrete. Strain gages were attached to the steel reinforcing cage along the length of the pile prior to placing the cage inside the steel shell, and a displacement transducer was attached to the top the pile to measure pile head displacement during the tests. A moment curvature analysis was used to calculate the flexural stiffness (EI = 291,800 kN-m^2) of the pile.

Setup without Stone Columns

After installation of the piles and prior to load testing, down-hole explosives were strategically placed in a circular array around the CISS pile and 4 pile group in an effort to produce a zone of liquefied sand around the piles after blast detonation. The explosives near the CISS pile were placed at a depth of approximately 3.2 m below the excavated ground surface and a radial distance of 2 m from the center of the pile. In addition, a series of piezometers were installed near the CISS pile to measure pore water pressures during testing. The test setup is shown in Figure 3; however, the location of down-hole explosives and piezometers near the 4 pile group are not shown.

Setup with Stone Columns

After completing lateral load tests in the blast-induced liquefied sand, the 1 m deep excavation was filled with soil and 24 stone columns with an approximate diameter of 0.9-m were installed to densify the liquefiable soils. Stone columns were installed in a grid pattern consisting of four rows and six columns with a spacing of approximately 2.4 m on center (Figure 4). Prior to performing additional load tests, the 1 m of fill was excavated, and down-hole explosives were again placed in an array around the piles. In addition, down-hole explosives were placed around the perimeter of the stone column improved soil. As a result, the explosive energy at the time of the blast detonation was significantly greater for the lateral load tests performed in stone column improved ground compared to lateral load tests prior to installing stone columns.



PPT	depth	PPT	depth
04	3.0 m	89	3.0 m
06	2.1 m	91	1.2 m
38	4.9 m	95	2.1 m
44	3.0 m	97	1.2 m
54	1.2 m	98	4.0 m
55	3.0 m		

Figure 3 Test Setup and Location of Peizometers and Explosives Before Stone Column Installation

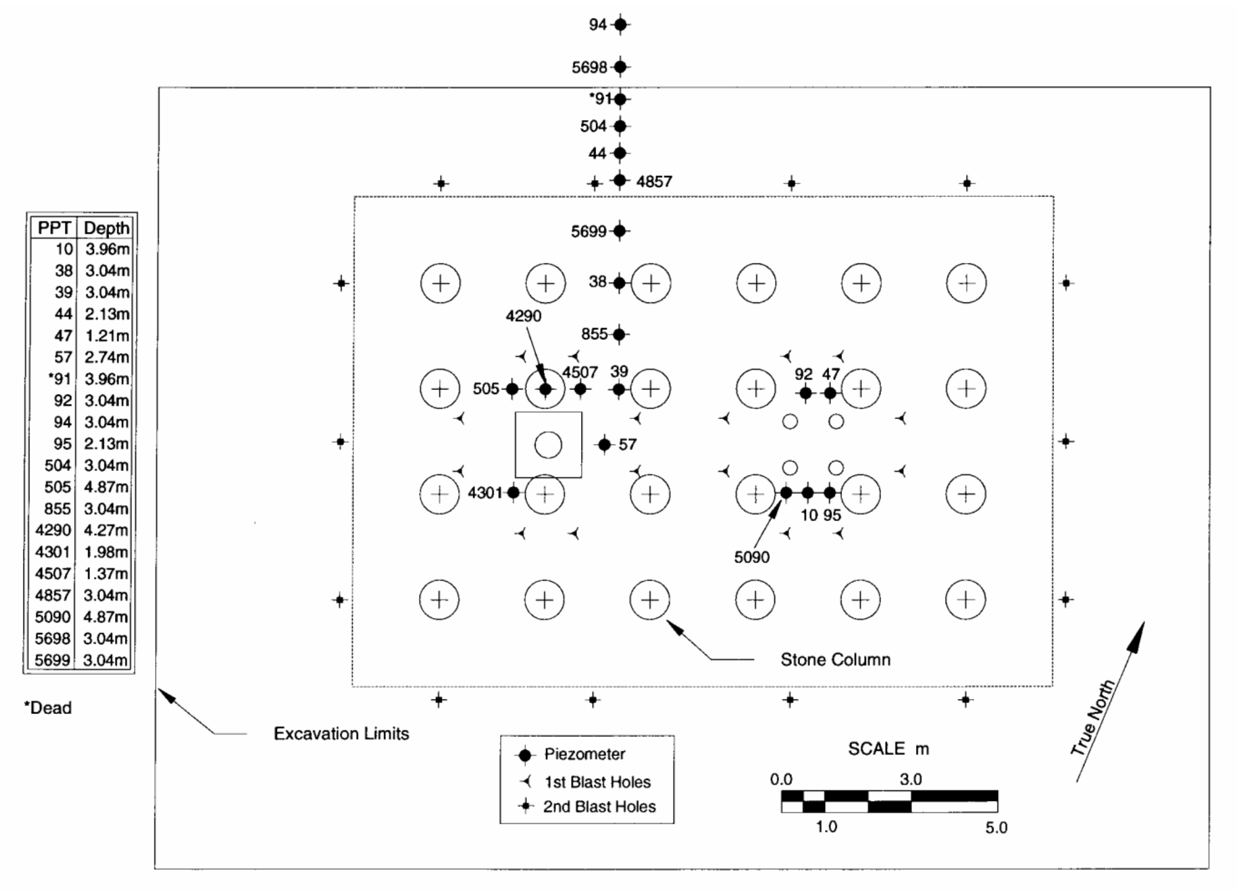


Figure 4 Location of Stone Columns, Piezometers, and Down-hole Explosives for Lateral Load Tests after Stone Column Installation

RESULTS

Load Tests without Stone Columns

A static load test was performed prior to detonating explosives at the test site. The test procedure consisted of pulling the CISS pile towards the pile group. The CISS pile was displaced up to 38 mm at the load point. Shortly after the static load test was completed, the down-hole explosives were detonated and a series of half cycle displacement controlled loads were applied to the pile. During these post-blast load cycles, the CISS pile was pushed away from the pile group. The first series of cycles consisted of displacing the CISS pile to 75 mm, 150 mm, and then 225 mm, followed by 10 more half cycles at a displacement of 225 mm. Results from the static and cyclic load test prior to installing stone columns are shown in Figure 5. After detonation of the explosives and during the first post-blast load series, excess pore water pressure ratios near the CISS pile ranged from 70% to 100%. Excess pore water pressure ratios calculated from the piezometer data near the CISS pile for the first post-blast load series are provided by Rollins et al. [6].

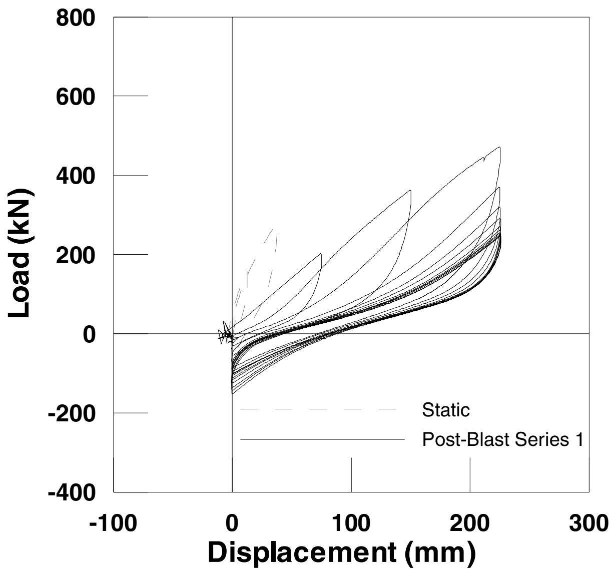


Figure 5 Load-displacement Response of CISS Pile without Stone Columns

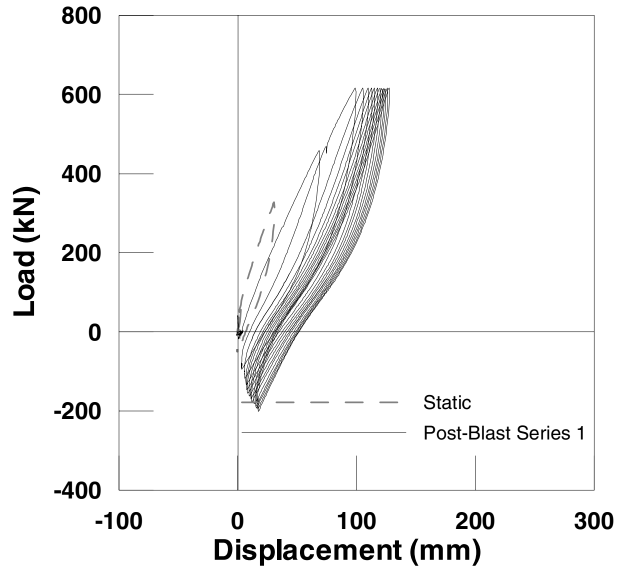


Figure 6 Load-displacement Response of CISS Pile with Stone Column Improved Ground

Load Tests with Stone Columns

A similar test protocol was followed for load tests after installation of the stone columns. The static load test consisted of pulling the CISS pile toward the pile group prior to detonating explosives. Again the maximum pile displacement at the load point was 38 mm. Explosives were detonated and cyclic lateral loads were applied. As a result of the increased stiffness in the lateral load response of the piles after installing the stone columns, load controlled tests were performed to prevent exceeding the capacity of individual load cells within the pile group. Results from the static load test and first load series after blast detonation are shown in Figure 6. Excess pore water pressures dissipated quickly after blast detonation and excess pore water pressure ratios near the CISS pile were generally less than 20% during the first post-blast load series. Excess pore pressure ratios near the CISS pile with stone column improved ground for the first post-blast load series are provided by Rollins et al. [6].

ANALYSIS

A lateral load analysis was performed for the CISS pile to compare analytical results with the full-scale pile response after stone column installation. The analysis was performed using the program COM624P. The analysis consisted of using a series of non-linear, Winkler type springs along the length of the pile to model the soil resistance that develops when lateral loads are applied to the pile. The non-linear spring response is called a *p-y* curve and is a force per tributary length of the pile. Since excess pore water pressures were relatively small during the lateral load test, standard static sand *p-y* curves developed by Reese et al. [7] were used to model the stone column improved soil. Soil properties required to calculate *p-y* curves for sand include soil unit weight (γ), constant of horizontal subgrade reaction (k), and friction angle (ϕ). Average values for the friction angle and constant of horizontal subgrade reaction along the pile length were are shown in Figure 2.

Due to some uncertainty in estimating the soil properties required for calculating standard *p-y* curves, an attempt to quantify the variation of each parameter was made and subsequent analyses for each varied parameter were performed. The standard deviation associated with the correlation used to obtain each soil parameter was greater than the standard deviation obtained from the variation in CPT tip resistance near the CISS pile. Therefore, the standard deviation for the correlations between CPT tip resistance and estimated soil property was used to estimate the variation in friction angle and relative density. The relative density plus or minus one standard deviation was used to define the upper and lower values of the constant of horizontal subgrade reaction. The standard deviation for the soil unit weight was estimated using a reasonable coefficient of variation of 5% as presented by Duncan [8]. The average soil properties used to define the sand *p-y* curves are shown in Table 1. The standard deviations for the friction angle, relative density, and total unit weight are 3 degrees, 10 percent, and 1.02 kN/m^3, respectively. Variation in the calculated pile head displacement resulting from the variation of soil properties was used to calculate the standard deviation of pile head displacement vs. applied load. Calculated pile head displacement vs. load and the measured pile head response are presented in Figure 7. Estimates of the maximum moment vs. load are presented in Figure 8.

TABLE 1: Soil Properties for Lateral Load Analysis

	Average Values			
Depth Below Excavated Ground Surface (m)	Friction Angle, ϕ (deg)	Relative Density, Dr (%)	Const.. of Horiz. Subgrade Reaction, k (MN/m^3)	Bouyant Unit Weight (kN/m^3)
0.00 – 0.30	42	74	39.4	20.42
0.30 – 0.61	42	74	39.4	10.61
0.61 – 4.57	44	93	64.1	10.61
4.57 – 5.49	42	73	38.2	10.61
5.49 – 13.80	31	25	6.5	10.61

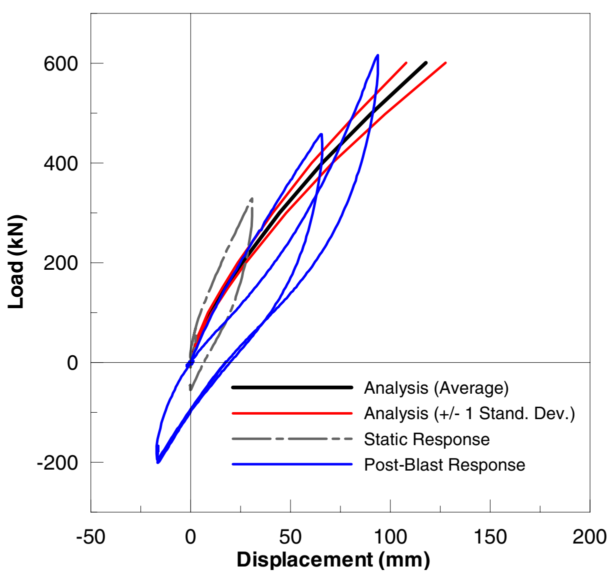


Figure 7 Pile Head Displacement vs. Load from Analyses and Measured Response After Stone Column Installation

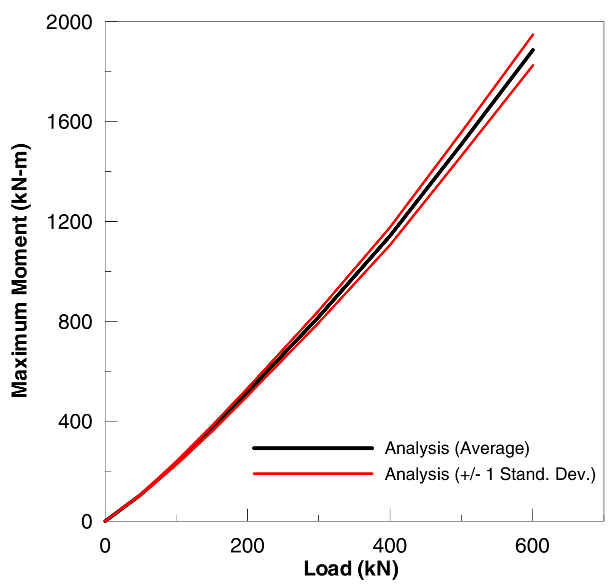


Figure 8 Load vs. Maximum Moment from Analyses After Stone Column Installation

DISCUSSION

Test Results

A comparison of the test results from Figures 5 and 6 show an increase in stiffness of the load-displacement response of the CISS pile after installation of stone columns. Prior to blasting, the lateral secant stiffness for the CISS pile was 7.5 kN/mm at a pile head displacement of 38 mm. After stone column installation, the secant stiffness at a pile head displacement of 38 mm increased by 44% to 10.8 kN/mm. A more dramatic increase in stiffness is observed when comparing the post-blast secant stiffness with and without stone columns. The post-blast secant stiffness without stone column improved ground was approximately 1.5 kN/mm after approximately three 225 mm displacement cycles; whereas, the secant stiffness with stone columns was over 350% greater at 7.0 kN/mm (Rollins et al. [5]). The dramatically increased stiffness for the pile in stone column improved ground is attributed to the increased density and friction angle of the soil and the fact that excess pore water pressures ratios were less than 20% during the load cycles compared to excess pore water pressure ratios between 70% and 100% during load cycles without stone column improved ground immediately following detonation of the down-hole explosives.

Analysis

Results from the analyses show greater pile head displacement for a given lateral load compared to the measured response. The average pile head displacement from the lateral load analysis is approximately 68% greater than the measured static displacement at a load of 300 kN. Detonation of the explosives produced a moderate increase in pore water pressure and a reduction in pile head stiffness. As a result, the average pile head displacement from the lateral load analysis is approximately 30% greater than the measured pile head displacement at a load of 600 kN. Although there is a substantial difference between the analysis and measured static response, the analysis appears to provide a reasonable estimate of the pile head response after detonation of the explosives.

The analyses also show that variation in the soil properties associated with the correlations used to obtain the soil properties did not produce a significant deviation from the average results. The standard deviation from the average pile head displacement is approximately 8% of the average value; whereas, the standard deviation from the average maximum moment is approximately 3% of the average value. Greater variability in measured CPT tip resistance could produce a significant deviation from average results at other sites.

CONCLUSIONS

Based on a review of data collected for the lateral load testing at Treasure Island and subsequent analyses, the following conclusions can be made:

1. The installation of stone columns significantly increased the relative density of the loose to medium dense sand.
2. The lateral pile head stiffness increased significantly after installation of stone columns as a result of increased soil strength and lower pore water pressures after detonation of the down-hole explosives.
3. The Reese et al. [3] sand *p-y* curves did not model the stone column improved soil response accurately under static loading.
4. The Reese et al. [3] sand *p-y* curves provided a reasonable estimate of soil response after detonation of the explosives.
5. The standard deviation for correlations between CPT tip resistance and soil properties do not result in large enough differences in the soil property to produce significant differences in the calculated pile response.

AKNOWLEDGMENTS

The authors wish to express their gratitude to the sponsors who made this project possible. These sponsors include: Caltrans (lead agency), Alaska DOTPF, Missouri DOT, Oregon DOT, Utah DOT, and Washington DOT. Several companies donated services to the project including Hayward Baker Inc., Geotechnics America/Mustang Construction, Geneva Steel, Condon-Johnson & Associates, Foundation Constructors, Subsurface Consultants, Kleinfelder & Associates, and Pacific Western, LLC. We also greatly appreciate the cooperation of the U.S. Navy and the City of San Francisco.

REFERENCES

1. Rollins KM, Ashford SA, Land JD. "Full-scale lateral load testing of deep foundations using blast-induced liquefaction". Proceedings: Fourth International Conference on Recent Advances in Geotechnical Engineering and Soil Dynamics, San Diego, CA, Paper No. SPL-3, 2001.

2. Wilson DW, Boulanger RW, Kutter BL. "Observed seismic lateral resistance of liquefying sand". Journal of Geotechnical and Geoenvironmental Engineering, 126(10), 2000: 898 – 906.

3. Kulhawy FH, Mayne PW. "Manual on estimating soil properties for foundation design". Electric Power Research Institute, 1990.

4. Peck RB, Hanson WE, Thornburn TH. "Foundation Engineering, 2nd Edition". Wiley & Sons, 1974.

5. American Petroleum Institute, "Recommended practice for planning, designing and constructing fixed offshore platforms, API recommended practice 2A (RP 2A)". Seventeenth Edition, 1987.

6. Rollins KM, Ashford SA, Lane JD, Hryciw RD. "Controlled blasting to simulate liquefaction for full-scale lateral load testing". 12th World Conference on Earthquake Engineering, Paper no. 1949, New Zealand, 2000.

7. Reese LC, Cox WR, Koop FD. "Analysis of laterally loaded piles in sand". Proceedings, Offshore Technology Conference, Houston, Texas, Vol. II, Paper No. 2080, 1974: 473 – 484.

8. Duncan JM, Navin M, Patterson K. "Manual for geotechnical engineering reliability calculations". Virginia Polytechnic Institute and State University, Center for Geotechnical Practice and Research, Blacksburg, VA, 1999.