

MICROPILE PERFORMANCE VARIABILITY: A FUNDAMENTAL CASE STUDY

Walter E. Vanderpool, Kleinfelder, Inc, Las Vegas, NV, USA

Donald A. Bruce, Geosystems, LP, Venetia, PA, USA

Edward E. Rinne, Kleinfelder, Inc, Oakland, CA, USA

Fred H. Kulhawy, Cornell University, Ithaca, NY, USA

Type 1B micropiles (Bruce and Juran, 1997) can be installed rapidly and efficiently. During an 85-day period, 536 micropiles were installed to depths of 61 meters within a site 41 meters by 43 meters. Their performance, as measured by load test, varied even where the materials, construction procedure, and installation technique were closely monitored and controlled. The variation in displacement, as measured by the standard deviation, exceeded 50 percent of the mean test-load displacement for all loads up to the maximum load applied (2670 kN). This study provides a statistical basis to predict variability with respect to the construction method, micropile materials, and installation technique. The variability of displacement at all test loads is a fundamental characteristic of this type of drilled deep foundation system.

INTRODUCTION

During an 85-day period in 1998, 536 micropiles were installed at a site in Las Vegas measuring 41 meters by 43 meters. Each Type 1B micropile was drilled through a soil profile of aged alluvial sediment to a depth of 61 meters. Type 1B micropiles are arguably the most common variant installed in North American soil profiles (Bruce et al., 1999). One contractor installed the micropiles using one technique. Each micropile was load tested by one method to the same maximum test load. The load tests and 75 retests were performed three to 110 days after installation.

The "static" pile load test is perhaps the highest quality test method available for design of deep foundations and verification of compliance with the design model. The construction and testing are expensive and time-consuming; therefore, the load testing generally is limited to a small number of piles in the foundation system or to critical and non-redundant foundation elements. In many cases, the load test is performed at the start of construction on piles that may or may not become part of the final works.

In this case study, the micropiled area was the central core of a much larger foundation system. The general characteristics of the foundation system and the purpose of the micropile installation are described elsewhere (e.g., Pearlman, 2000).

There were no geotechnical exploration data available for the lower one-half of the micropiled depth until 10 days before the decision was made to install micropiles. Geotechnical exploration to below the tip elevation was not available until after 16 micropiles had been installed and two had been load tested. The micropiles were being installed in quantities, at rates, and to depths without precedent. For these reasons, it was recommended that each micropile should be load tested as an acceptance criterion.

The potential load to be supported by the micropiled area was estimated to be approximately 1110 MN (Pearlman, 2000). Based on the potential load and the decision to load test each micropile, a maximum test

load of 2670 kN per micropile was established. The contractor prepared an installation plan and procedure, which was reviewed and modified, as necessary, before micropile installation was allowed to begin. A quality control plan was developed during installation of the early micropiles, and this plan was followed closely and was enforced throughout the remainder of the construction.

GEOTECHNICAL CONDITIONS

The Las Vegas Valley is a sediment-filled basin located in a desert environment (Matti and Bachhuber, 1985). The alluvium is over 100 meters thick at the site and consists of clayey sand with gravel and finer-grained zones of low to high plasticity clays. Limestone mountains border the Las Vegas Valley and the alluvial deposition in the desert environment results in soils with variable carbonate content and cementation.

Where the soils are uncemented, they are generally characterized as soft to very stiff in consistency. Most are partially cemented with thin layers (0.3 to 1.0 meter) of highly cemented soils. The highly cemented soils, locally known as caliche, were present at various elevations within the profile. Borings in the vicinity of the micropile installation indicate cumulative thickness of caliche varying from 0.3 to 4 meters. The layers were concentrated near 6, 15.5, and 55 meters in the 61 meter depth.

Undrained shear strengths (s_u) interpreted from Cone Penetration Test (CPT) values in non-cemented or weakly cemented soils ranged from less than 20 kPa to greater than 1000 kPa. Unconfined compressive strengths for the Las Vegas Valley caliche deposits have been measured within the range from 4 MPa to 100 MPa. The unconfined compressive strength, at a depth of 10.7 meters below the top of the micropiles, ranged from 25.1 MPa to 47.6 MPa in one of the layers at this site.

Groundwater was located approximately 10 meters below the natural grade of 657 m MSL and near the bottom elevation of the mat.

MATERIALS

The micropiles consisted of 12.7 mm wall, 152 mm I. D., threaded, flush joint steel pipe in 3.05 meter lengths, and type V Portland cement grout batched at a 0.44:1 (water: cement) ratio by weight. A dispersing and set retarding agent was added at a dosage of 5090 mm³ per kilogram of cement to aid in efficient placement.

The air temperature ranged from 17.8° to 40.6° C with a mean temperature of 28.4° at the time of grout placement.

Yield strength test results by ASTM A370-97 were submitted by the pipe suppliers. The mean yield strength of the steel pipe was 769 MPa (n=122, S.D.=118 MPa).

Type V cement test results by ASTM C109, C151, C185, and C191 were submitted by the cement manufacturers. The mean compressive strengths at 3 and 28 days were 20.3 MPa (n=14, S.D.=1.0 MPa) and 43.3 MPa (n=14, S.D.=2.4 MPa), respectively.

Fluid grout properties were measured by ASTM D4380, C939 and C1064 at the time of placement. The mean properties were: specific gravity=1.90 (n=513, S.D.=0.03), flow time=64 seconds (n=504, S.D.=32 seconds), and temperature=29.3° C (n=513, S.D.=3.2°).

Test cubes were prepared from grout samples collected at the micropile during placement by ASTM C109. The mean compressive strengths at 2 to 3 days, 4 to 5 days, and 28 days were 28.6 MPa (n=475, S.D.=6.7 MPa), 36.8 MPa (n=498, S.D.=5.9 MPa), and 55.9 MPa (n=510, S.D.=7.7 MPa), respectively.

CONSTRUCTION PROCEDURE

A steel reinforced concrete mat, three meters thick, had been constructed throughout the area to be micropiled more than one year before micropile installation began.

A 1.2 meter installation grid was established throughout the area to be piled (Figure 1). The mat was cored to provide access for the production micropiles. The drill pipe became the micropile reinforcement. Drill cuttings were removed by flush water, aided by polymer drilling fluid, along the annulus of the drill pipe. The micropiles were drilled with a sacrificial tri-cone rotary bit 210 mm to 216 mm in diameter.

Each micropile was grouted by tremie from the sacrificial bit to the surface. The micropile was completed by pressure injection to flush grout return to the surface along the micropile annulus. The micropiles are therefore classified as Type 1B (Bruce and Juran, 1997).

Prior to production micropile installation, one micropile was drilled to a depth of 61 meters (592 m MSL) to evaluate constructibility. This micropile was drilled adjacent to the area to be piled and beyond the edge of the mat. The decision to install micropiles to a depth of 61 meters below the mat (588 m MSL) was based on practical limitations of constructibility.

One geotechnical exploration to elevation 592 meters MSL and limited laboratory test results were available when the test pile was drilled. Geotechnical information developed concurrent with micropile installation and the load test results were evaluated periodically as they became available.

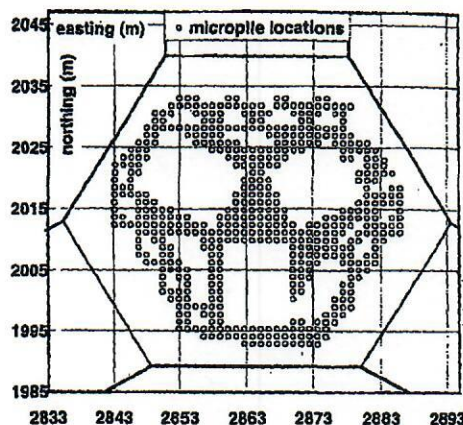


Figure 1 - Micropile locations

Following exploration to a depth of 94.5 meters (558 m MSL), the quality control data, the geotechnical data, and the load test results were reviewed. The review of these available data did not justify modification of the planned number of micropiles, their lengths, or locations.

Grout return to the surface was recorded in 509 of the 536 micropiles. The grout yield and the grout injection pressure were also monitored during placement. The mean grout yield (grout volume placed : theoretical micropile volume) was 1.16 : 1.00 (n=531, S.D.=0.10). The mean grouting pressure was 880 kPa (n=528, S.D.=440 kPa).

Each micropile was isolated from the mat by a 254 mm diameter PVC pipe sleeve. The test load reaction was transferred to the mat at each micropile through four 45 mm diameter bars with a specified tensile strength of 1030 MPa.

A calibrated hydraulic jack was mounted on each micropile. The jacks were installed to control the load and even, potentially, to lift the micropiled area, if it became necessary. However, removal of soil covering the mat, micropile installation, load testing, and the load applied to the micropiles at the end of construction stabilized the mat.

The contractor established an installation sequence to facilitate mobility within a congested work area. Micropiles were advanced by rotary drilling methods to the intended depth of 61 meters. Only the equipment and the penetration resistance of the soil profile restricted the drilling rate. The typical installation equipment and conditions are shown in Figure 2.

The photograph was taken during the installation of micropile number 394. The addition of a 3.05 meter length of drill pipe was in progress at that time. The drilling rate was monitored continuously in three meter increments throughout the micropile installation.

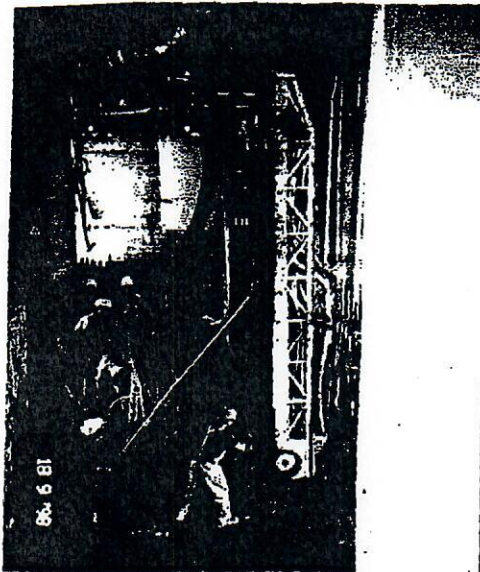


Figure 2 - Drilling equipment

Table 1 presents the mean drilling rate by depth. Table 2 presents the mean drilling rate by micropiled quadrant. Construction proceeded on a 24 hour per day, 6 day per week schedule.

LOAD TEST PROCEDURE

The load test procedure was developed from the ASTM D1143-81 and Post-Tensioning Institute (PTI, 1996) test methods. The procedure consisted of cyclic loading to the maximum test load of 2670 kN in seven load increments. Each intermediate load increment was held for one minute. The maximum test load was held for 10 minutes or up to one hour to observe the creep rate between 1 and 10 minutes or 6 and 60 minutes after the load was applied. The 2670 kN test load was sustained on one micropile (#298) for 5 hours.

The load was applied by a hydraulic jack positioned on the top of the micropile and reacting through a steel saddle beam and four steel bars anchored in the concrete mat.

A hydraulic pressure gage and a vibrating wire (VW) pressure transducer monitored the test load. Two dial indicators and two VW extensometers monitored the displacement. A typical load test assembly is shown in Figure 3.

Two VW strain gages were mounted on opposing sides of the micropile head. The VW strain gages were calibrated during the load test to provide for longer-term monitoring following completion of the micropile installation.

A total of 667 load tests were attempted on the 536 micropiles. Fifty-six of the tests were aborted for a variety of reasons. The reasons included reaction bar anchor bond failure, jack or frame misalignment or distortion, hydraulic leaks, and instrumentation failures. Seventy-four micropiles were retested. One micropile was retested twice. The retests were performed on micropiles that failed to achieve the creep acceptance criteria during the first test.

Table 1 - Drilling rate by depth

Depth (m)	Mean drilling rate (m/min.)
1.5	1.50
4.6	1.00
7.6	0.96
10.7	1.19
13.7	1.25
16.8	1.23
19.8	1.62
22.9	1.43
25.9	1.82
29.0	1.61
32.0	1.71
35.1	1.61
38.1	1.58
41.1	1.82
44.2	1.55
47.2	1.68
50.3	1.64
53.3	1.18
56.4	1.08
59.4	1.42

Table 2 - Drilling rate by quadrant

	Total tests	South half	North half	Southwest quadrant	Southeast quadrant	Northwest quadrant	Northeast quadrant
No.	536	283	253	135	148	123	130
Mean	1.45	1.51	1.38	1.41	1.61	1.29	1.46
S. D.	0.50	0.53	0.45	0.50	0.55	0.43	0.45

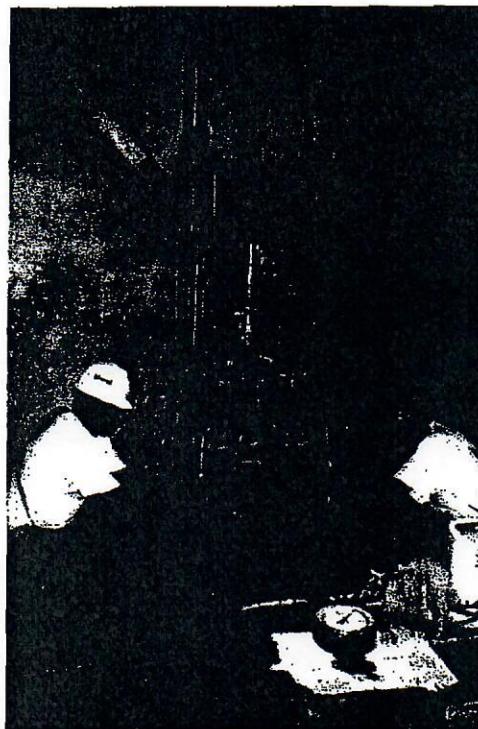


Figure 3 - Load test

The micropiles were attached to the mat after all of the load tests and retests had been completed. The load testing was completed 117 days after installation began.

OBSERVATIONS AND ANALYSES

Although the valley stratigraphy was relatively constant and micropile spacing was six diameters center to center, displacements varied at all load increments. The variation in the micropile displacement was greater than 50 percent of the mean displacement, as measured by the standard deviation, at all test load increments. Variation in the permanent displacement was greater than 100 percent of the mean permanent displacement (Table 3).

Four micropiles could not sustain the 2670 kN test load without continuing movement. These micropiles were installed #1 (SW), #2 (SW), #168 (NE), and #481 (SE) in the construction sequence and (quadrants).

In 58 cases, the micropiles were unable to sustain the 2670 kN test load at the acceptance creep rate criterion. The creep rate for these micropiles ranged from 2.3 mm to 8.6 mm after 6 to 60 minutes at the sustained 2670 kN load. The creep rate after 30 to 300 minutes was 2.5 mm at micropile # 298.

Replacement or remediation of these micropiles was not an option. The structure loads, the load test results, the

quality control records, and the distribution of these micropiles were reviewed.

Table 3 - Load test results

Load (kN)	No.	Mean disp. (mm)	S. D. (mm)	Mean + 1 S. D. (mm)	Mean - 1 S. D. (mm)
89	536	0.0	0.0	0.0	0.0
222	536	0.3	0.3	0.6	0.0
445	536	1.1	0.8	1.9	0.2
890	535	3.2	2.1	5.3	1.0
1330	535	6.1	3.8	9.9	2.3
1779	535	10.3	6.2	16.5	4.1
2220	529	16.5	9.6	26.1	6.9
2670	528	26.5	14.5	41.0	11.9
89	533	3.8	4.1	7.9	-0.3

Construction was allowed to continue to completion with the recommendation that the VW strain gages located at the tops of the micropiles and the displacement of the mat should be monitored.

Eight quality control variables were evaluated as part of this case study. The variables included:

- Installation sequence
- Micropile location
- Drilling rate by location and depth
- Fluid grout properties
- Grout compressive strength
- Grout return to the surface and volume placed
- Grout injection pressure
- Micropile rotation during pressure injection

There was a very weak correspondence between drilling rate and micropile displacement at each test load. The correspondence can be seen in Figure 4. The variation in the drilling rate was less than one-half the variation in the displacement at all test loads.

Difficulties were encountered during installation of the early micropiles. The problems appeared as low grout yield and no grout return to the surface. Polymer mud was added to the flush water near the completion of the drilling and the drilling technique was modified. A dispersing agent and set retarder was added to the grout and the grouting technique was modified. These changes in the construction methods reduced the problem of large displacements during load tests and the failure to meet the load acceptance criterion.

There was a correspondence between installation sequence and micropile displacement at each test load (Figure 5).

The same general pattern of large displacements early in the construction sequence and a general increase in displacements near the end of construction also was present in the permanent micropile displacement at the completion of each load test (Figure 6). The permanent (non-elastic) displacement at the end of all load tests was more variable than the displacement at any one of the load increments during testing.

Five of the first seven micropiles installed were tested at four to six days after grout placement. Two of these micropiles were among the four that did not meet the load criterion for acceptance. Five of these micropiles met the load and creep acceptance criterion.

The micropile installation sequence generally progressed from north to south and the last micropiles installed were in the southeast quadrant. There are many possible causes or explanations for the observed trends in Figures 5 and 6.

The age of the micropile at the first load test was not a significant factor in the variability of the displacements during load test (Figure 7).

Micropiles in the north one-half of the site displaced similarly at each of the test loads (Figure 8). The micropiles in the south one-half and particularly in the southeast quadrant of the site displaced more at each load increment than micropiles in other quadrants. The mean permanent displacement (non-elastic) was 37 percent greater in the south one-half than in the north one-half of the site (4.45 mm vs 3.06 mm).

The casing was rotated during grout pressure injection at 44 percent (238) of the micropiles. The load test displacements indicate the practice may have been the cause of increased displacements compared to the mean displacement of all micropiles that achieved grout return to the surface (Figure 9).

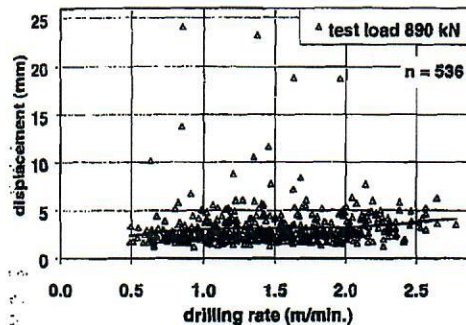


Figure 4 - Displacement at 890 kN vs drilling rate

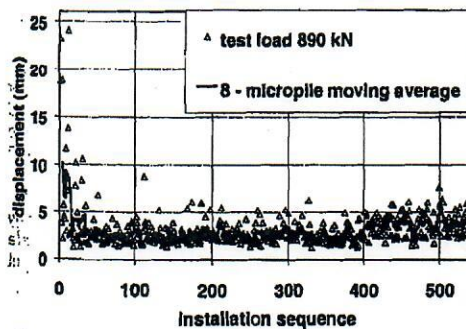


Figure 5 - Displacement at 890 kN vs installation sequence

The mean displacement at the end of the first test for all micropiles was 3.8 mm. The mean displacement from the first load test on micropiles retested was 6.3 mm. The mean additional micropile displacement following the retest was 1.8 mm. The mean displacements at the first test and retest load of 890 kN are shown on Figure 10.

The data from the first test and retests show the effect of cyclic loading on the displacement at intermediate loads. The reload data are nearly linear up to the maximum test load.

The load-displacement properties of the retested micropiles were well-developed within four to seven days after grout placement (Figure 11).

The strongest correspondence between load test displacement and a quality control parameter recorded was the relationship between grout yield (ratio of grout volume placed to theoretical micropile volume) and displacement. The mean grout yield was 1.16 : 1.00.

There were 27 micropiles where grout return to the surface was not recorded. There were 14 of 531 micropiles where the yield was less than 1.0. Grout yields greater than approximately 1.4 may be related to wash-out between pile locations beneath the mat by flush water. Grout yield greater than 1.4 had little effect on the displacement during load test (Figure 12).

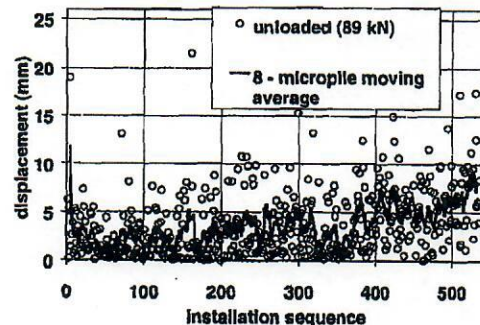


Figure 6 - Permanent displacement vs installation sequence

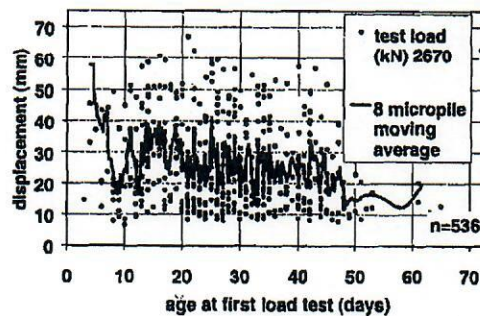


Figure 7 - Displacement vs age at first load test

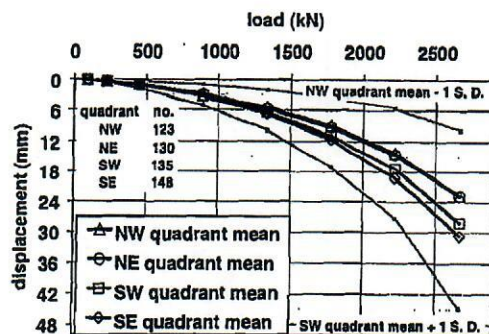


Figure 8 - Displacement by quadrant

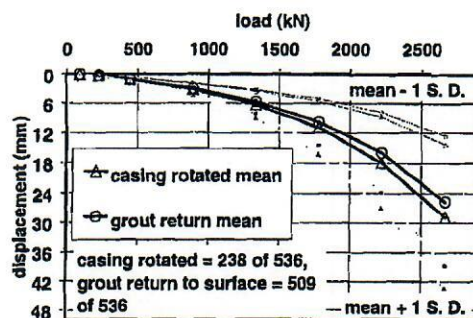


Figure 9 - Displacement with casing rotation and with grout return to the surface

An interpretation of the micropile capacity based on the first load test ($n=536$) is presented on Figure 13. The slope of the "initial linear region" was determined by a least squares best fit for the load interval from 222 to 890 kN (L_1).

The Davisson (1972) criterion was applied to obtain the off-set tangent intercept with the mean load-displacement data. The intercept occurred within the range of observations from the load tests. The beginning of the interpreted linear failure region was established by the relationship for micropiles, $L_2=1.18L_{st}$. (Jeon and Kulhawy, 2001).

A hyperbolic curve defined by the equation presented on Figure 13 also was fitted to the data set. The symbols are: P = applied test load (kN), D = micropile head displacement (mm), and the fitting coefficients a and b .

The analyses show the off-set tangent line and the load-displacement data curve intersect at a test load of 2100 kN for data obtained from the first load test. The "beginning of the linear failure region" (L_2) by the interpretation method proposed by Jeon and Kulhawy (2001) was 2350 kN for the mean of the first load tests in the data set.

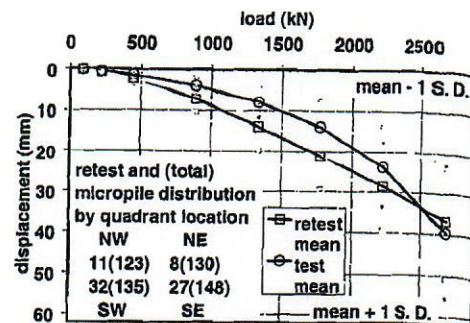


Figure 10 - Displacement at 890 kN load (first test & retest)

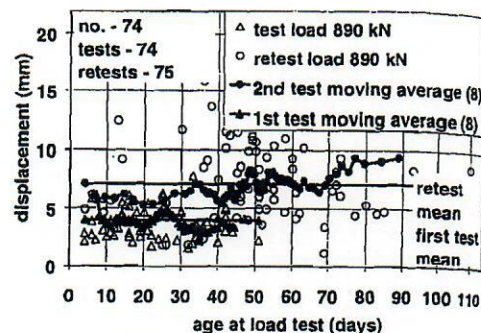


Figure 11 - Displacement vs age at load test and retest

The estimated elastic limit load by Hirany and Kulhawy (2002) at 50 percent of Q_{L2} closely matches the data set of this study. The displacement at the beginning of the "interpreted linear failure region" L_2 was 8 percent of the micropile diameter (B).

The "beginning of the linear failure region" has been shown to occur near 4 percent of B for straight sided drilled shafts (Hirany and Kulhawy, 2002). The "beginning of the linear failure region" for micropiles was found to be 5.5 to 6.5 percent of B for non-cohesive and cohesive soil profiles, respectively (Jeon and Kulhawy, 2001).

CONCLUSIONS

The standard deviation of the displacement was greater than 50 percent of the mean displacement at every load increment from 222 kN to 2670 kN. The standard deviation of the final displacement was greater than 100 percent of the mean final set (Table 3 and Figure 7).

This study strongly supports the observation by Hirany and Kulhawy (2002) that all drilled foundation types are subject to large variations in displacement at equivalent test loads.

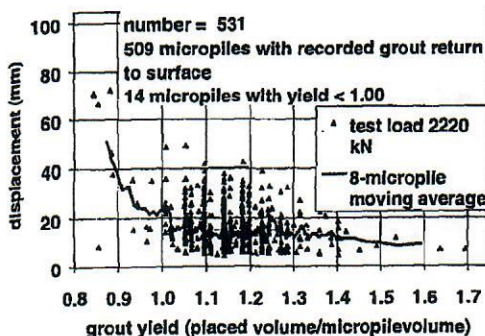


Figure 12 - Displacement at 2220 kN load vs grout yield

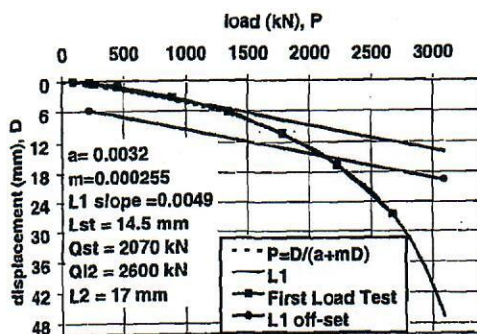


Figure 13 - Load test interpretation

Five trends appear from the analyses. The strongest correspondence between load test displacement and a quality control variable evaluated in this study was the relationship between grout yield (ratio of grout volume placed to theoretical micropile volume) and displacement. The mean grout yield was 1.16 : 1.00.

A minimum grout yield criterion of 1.15:1 is recommended as a quality control parameter for micropile construction.

Micropiles installed in the north one-half of the site displaced less than those in the south one-half of the site at all equivalent test loads and in the permanent set.

REFERENCES

ASTM, 1997, Sections 1 and 4, Volumes 1.01, 1.02, 1.03, 1.04, 4.01, 4.02, and 4.08, Test Methods A370-97, C109/c 109/m-95, C151-93a, C185-95, C191-92, C939-97, C1084-86 (Reapproved 1993), D1143-81 (Reapproved 1994), D4380-84 (Reapproved 1993).

BRUCE, D. A., and JURAN, I., 1997, Drilled and Grouted Micropiles: State of Practice Review, Volumes I, II, III, and IV FHWA-RD-96-016, -017, -018, and -019, Federal Highway Administration.

The largest displacements occurred at micropiles installed in the southeast quadrant. The drilling rates also indicate slightly faster drilling rates in the soils of this quadrant (Table 2).

There was an installation technique learning process during construction of the early micropiles. Adjustments in the application of the polymer drilling fluid and the drilling technique improved flush removal of the drill cuttings. Refinements in the grout placement technique improved reliable grout return to the surface (Figures 4, 5, and 6).

Micropile construction specifications should recognize the possible need to develop variations in the construction procedure to address variations in ground conditions and other possible conditions.

A trend of increasing micropile displacements at equivalent test loads during the construction sequence began to re-appear in the last one-third of the micropiles installed. This trend may be related to the micropile locations within the site, a relaxation in the construction technique, or a failure to maintain quality control feedback (Table 2 and Figures 4, 5, and 6).

The quality control program should include frequent progress review and feed-back to detect changes in ground conditions and changes in construction technique and to provide corrective action and process improvements, when necessary.

The practice of rotating the drill pipe during the pressure injection of grout to complete the micropile did not improve micropile performance. Statistically the practice produced slightly more elastic, softer, poorer performing piles (Figure 8).

Although 532 of the 536 micropiles (99%) supported the maximum test load for one hour, 58 of the micropiles (11%) were continuing to move by creep at a rate of 2.3 to 8.6 mm per hour after one hour.

The large variation in micropile displacements during load tests and the retests was not attributable to any one of the properties monitored during construction. The variation in micropile performance indicated by this study appears to be a fundamental characteristic of the Type-1B micropile. The variability must be addressed in the design model and in the evaluation of design or verification load tests.

BRUCE, D. A., BRUCE, M. E., and TAYLOR, R. P., 1999, High Capacity Micropiles - Principals and Case Histories, GeoEngineering for Underground Facilities (GSP 90), ASCE, Reston, pp 188-199.

DAVISSON, M. T., 1972, High Capacity Piles, Proc., Lecture Series on Innovations in Foundation Construction, ASCE Illinois Section, Chicago, 52 p.

HIRANY, A., and KULHAWY, F. H., 2002, On the Interpretation of Drilled Foundation Load Test Results, ASCE International Deep Foundations Congress, Orlando, February 14-16, 2002, in press.



JEON, S. S. and KULHAWY, F. H. 2001, Evaluation of Axial Compression Behavior of Micropiles, Foundations and Ground Improvement (GSP 113), Ed. T. L. Brandon, ASCE, Reston, pp. 460-471.

MATTI, J. C., and BACHHUBER, 1985, Las Vegas SW Quadrangle Geologic Map 3B9, Nevada Bureau of Mines and Geology, Reno.

PEARLMAN, S. L., 2000, Pin Piles for Structural Underpinning Proceedings, 25th Annual Meeting, Deep Foundations Institute, New York, pp. 389-400.

POST-TENSIONING INSTITUTE, 1996, Recommendations for Prestressed Rock and Soil Anchors, Phoenix, pp. 48-63.