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Tension Tests On Drilled Micropiles In A Stiff Clay

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ABSTRACT

A series of 20 small diameter drilled and grouted micropiles were installed at three different depths in a stiff surficial clay crust at the National Geotechnical Experimentation Site in Amherst, Massachusetts. A detailed site characterization program was performed to evaluate the soil characteristics in the crust. Three different sizes of micropiles ranging in diameter from 76 mm to 152 mm and having lengths from 1.52 m to 4.57 m were installed vertically at the site using both continuous flight augers and hand auger techniques. Concrete was placed in the open holes using gravity free-fall. After allowing the concrete to cure for a period of 30 days, tension tests to failure were conducted on each of the micropiles. Following initial tests, some of the micropiles were retested after a resting period of one year to evaluate the recovery in tension capacity. This paper presents a description of the soil characteristics at the site including both laboratory and field test results and a description of the methods used to construct and test the micropiles. A comparison is made of the ultimate capacity obtained from the tests. The influence of drilling technique and the effect of reloading on the measured capacity are discussed.

KEYWORDS

Micropiles, Field Tests, Pullout Capacity, Tension Tests, Clay

INTRODUCTION

Small diameter drilled and grouted piles are becoming increasingly popular for resisting foundation loads. Such elements are often referred to as "pin piles", "minipiles", "micropiles", "root piles", etc., and are simply small diameter drilled holes filled with Portland cement based grout. Most engineers consider the primary application of micropiles to be for supporting compressive loads. There are a large number of reported cases in the literature (e.g., Singe and Heine 1984, Soliman and Munfakh 1988, Bruce 1989). However, as indicated in Figure 1, there are a number of design applications where micropiles may also be useful for resisting tensile forces. In this sense, micropiles are not altogether different than a grouted anchor or small diameter drilled shaft. Relatively few studies have been conducted to evaluate the influence of construction techniques on the performance of drilled piles (e.g., Clayton and Milititsky 1983; Van Weele 1988). The authors could find no examples where the influence of reloading was evaluated in clays. Tests were conducted on 20 micropiles installed in the surficial clay crust at the National Geotechnical Experimentation Site on the University of Massachusetts-Amherst campus in Amherst, Massachusetts. After allowing the grout to cure sufficiently, axial tension (uplift) tests were conducted to failure. Tests were conducted to evaluate the influence of drilling method on the uplift capacity as well as to determine what the ultimate capacity recovery would be in this clay crust by retesting the piles one year after the initial load test. The results of the tests are presented in this paper.

TEST PROGRAM

Site Characteristics

Tests were performed at the National Geotechnical Experimentation Site located at the University of Massachusetts at Amherst. The subsurface stratigraphy at the site generally consists of about 1 m of mixed cohesive and cohesionless random compacted fill overlying a thick deposit of late Pleistocene varved silt and clay. This deposit of silt and clay is identified as Connecticut Valley Varved Clay (CVVC) and is the result of lacustrine deposition into glacial Lake Hitchcock. The upper 5 to 6 m consists of an overconsolidated crust as a result of surface erosion, desiccation, seasonal fluctuations in the ground water level, and other physical and chemical processes. Below the crust, the deposit becomes more normally consolidated. The thickness of individual silt or clay varves is on the order of 2 to 8 mm and in general the varves occur in a horizontal position. The ground water table in the upper 3 m at the site typically shows variations of ±1.2 m throughout the year that coincide with changes in seasonal precipitation. Long term water levels have shown a maximum fluctuation of about 3 m. Geotechnical characteristics of the site throughout the upper 5 m are shown in Figure 2.

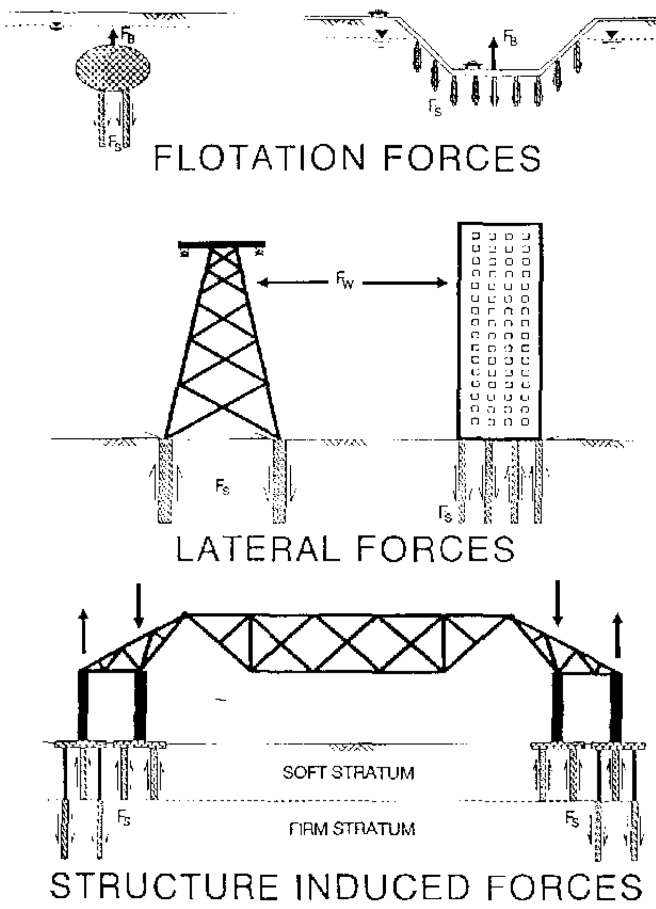

Figure 1. Tensile Forces in Geotechnical Construction.

Installation of Micropiles

Twenty drilled micropiles were installed at various locations around the site for this study. Micropiles were installed by two different methods. Open sided hand operated bucket augers were used to drill holes of three different sizes, i.e., 76 mm, 102 mm, and 152 mm. For each size of micropile a 1.52 m, 3.05 m, and 4.57 m length was constructed. In order to provide a comparison, a truck mounted drill rig equipped with different diameter continuous flight augers was also used to construct micropiles of the same diameter and length. As the drilling proceeded, a water content sample was obtained at the center of each 0.3 m depth for each boring. A single No. 6 reinforcing bar was installed down the center of the test shaft and was attached to a 12 mm thick circular base plate approximately equal to the hole diameter. After placing the reinforcing bar, concrete was placed by gravity free-fall. All of the holes were dry at the time of concrete placement. Table 1 gives a summary of the characteristics of the micropiles investigated in this paper.

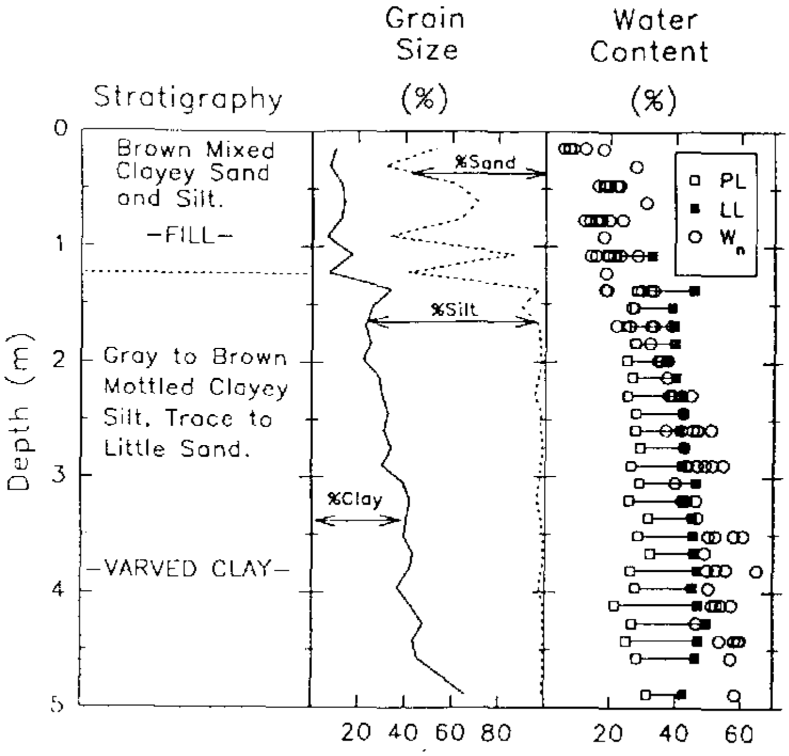

Figure 2. NGES Test Site Characteristics (upper 5 m).

Uplift Tests

Vertical tension (uplift or axial pullout) tests were performed on each micropile after a minimum of 30 days had elapsed from the time of installation. The load tests were carried out in general accordance with ASTM Standard D3689. Load was applied by a single acting, 250-kN hydraulic jack that rested on an I-beam supported by wood cribbing. Load was transferred to the tip of the shaft using a yoke system that was secured to the shaft with threaded rods welded to the steel reinforcement. The load test arrangement is shown in Figure 3. Load was applied to each shaft in increments in the range of approximately 5 to 10% of the predicted ultimate capacity. Each load increment was maintained for 20 min. and deflections were recorded immediately after and at 2, 5, 10, and 20 min. following application of the load. The load was measured using a Geokon 3000-300-2 load cell connection to a Measurements Group P-3500 strain indicator. Deformation measurements were made using two dial gauges capable of resolving 0.025 mm placed on opposite sides of the shaft. The dial gauges rested on a steel plate that was bolted securely to the threaded rod welded to the shaft steel reinforcement.

Table 1. Micropile Characteristics.

Micropile No.	Site I.D.	Drilling Method	Diameter (mm)	Length (m)	L/D
1	U-1	H	76	1.52	20
2	U-2	H	76	3.05	40
3	U-3	H	76	4.57	60
4	C-4a	H	102	1.52	15
5	C-4b	H	102	3.05	30
6	U-4	H	152	1.52	10
7	U-5	H	152	3.05	20
8	U-6	H	152	4.57	30
9	U-17	F	76	1.52	20
10	U-18	F	76	3.05	40
11	U-19	F	76	4.57	60
12	U-11	F	102	1.52	15
13	U-12	F	102	3.05	30
14	U-20	F	152	1.52	10
15	U-21	F	152	3.05	20
16	U-22	F	152	4.57	30

H = Hand Auger F = Continuous Flight Auger

RESULTS

A summary of the tension test results is presented in Table 2. Typical load vs. displacement curves are shown in Figure 4. The ultimate capacity in uplift determined from the interpreted failure load may be given as:

$$Q_{uplift} = Q_{shaft} + W$$

where:
Q_{shaft} = total shaft resistance in uplift
W = mass of the micropile

The unit skin friction is simply obtained as Q_{shaft} divided by the total surface area, A, as:

$$f_s = Q_{shaft} / A$$

Most tests were loaded to produce a displacement of at least 35 mm but in some cases a sharp plunging type failure did not occur. The ultimate capacity in each case was obtained using a simple tangent intersection method of interpreting the load-displacement curve. This method uses the intersection of tangents drawn to the initial and final portions of the load curve to define the ultimate capacity. The failure displacement was then obtained as the displacement on the load-displacement curve at the interpreted failure load. Results from the first 6 tests and a discussion of the interpretation of the load curve have previously been presented by Lutenegger and Miller (1994).

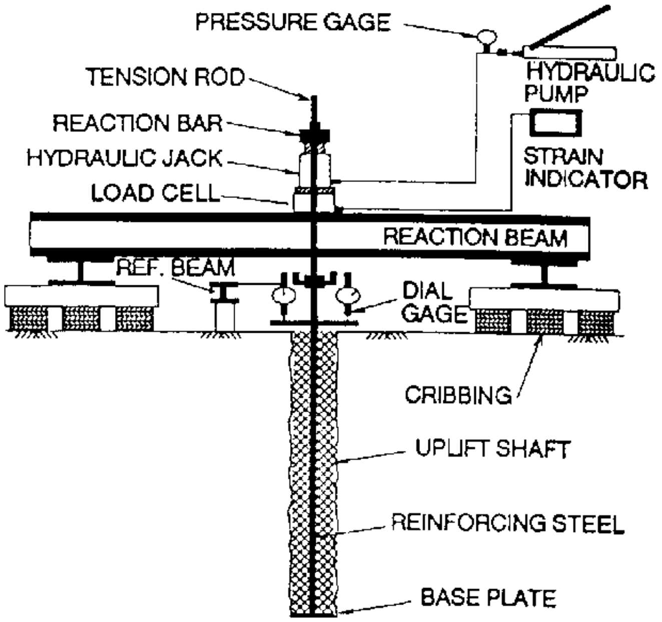

Figure 3. Load Test Arrangement.

Influence of Drilling Method

The influence of drilling method on the measured uplift capacity was investigated by comparing the ultimate uplift capacity obtained from the hand augered holes and the holes drilled with continuous flight augers. This comparison is presented in Table 3. It can be seen that in the smallest diameter holes, the hand auger technique gave smaller capacity, on the order of 75% as compared to holes produced by a flight auger. However, in the larger diameter holes, i.e., 102 and 152 mm, the hand auger holes gave considerably higher capacities than similar size and length holes produced with the flight augers. On the average, hand augered holes gave almost 2 times the capacity of flight augered holes. This is likely related to the build up of a remolded soil zone along the walls of the borehole from the flight augers. For the tests reported in this paper, micropiles drilled using continuous flight augers also showed a higher normalized displacement (s/D) to reach ultimate capacity; 9.1% compared with 6.9% for hand augered holes.

Table 2. Summary of Micropile Tension Test Results.

Micropile No.	Site I.D.	Failure Load (kN)	Unit Skin Friction (kPa)
1	U-1	16.6	45.5
2	U-2	34.9	47.8
3	U-3	52.7	48.1
4	C-4a	20.0	41.1
5	C-4b	73.0	75.0
6	U-4	46.0	63.0
7	U-5	88.4	60.5
8	U-6	113.6	51.9
9	U-17	20.7	56.7
10	U-18	62.0	85.0
11	U-19	66.0	60.3
12	U-11	10.7	22.0
13	U-12	23.0	23.6
14	U-20	18.0	24.7
15	U-21	62.5	42.8
16	U-22	83.0	37.9

Table 3. Ratio of Ultimate Capacity from Hand Augered Holes to Flight Augered Holes.

Diameter (mm)	Length (m)	Capacity Ratio H/F	Mean Ratio
76	1.52	0.80	0.72
76	3.05	0.56	
76	4.57	0.80	
102	1.52	1.87	2.52
102	3.05	3.17	
152	1.52	2.56	1.78
152	3.05	1.41	
152	4.57	1.37	

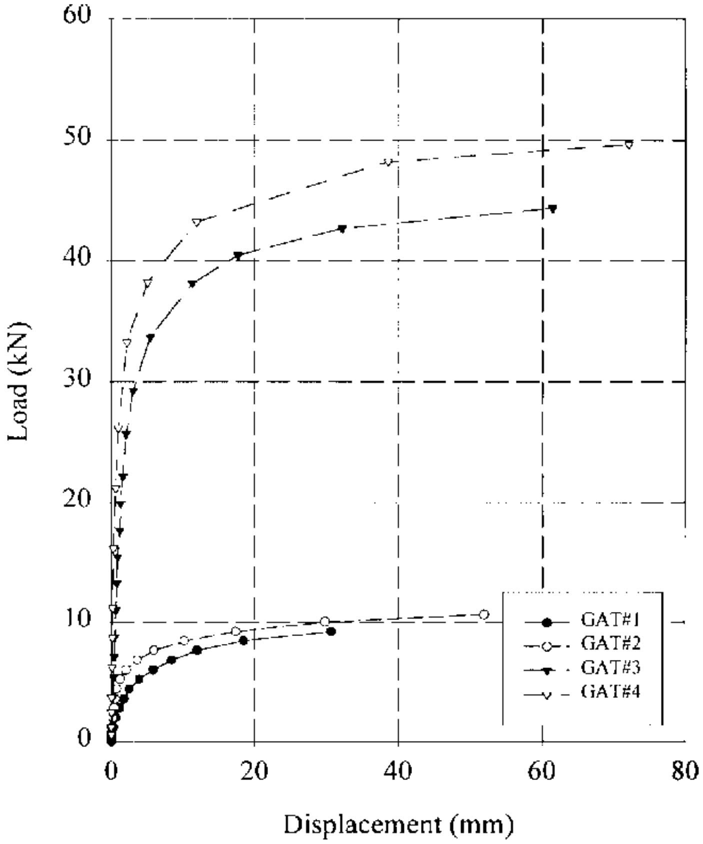

Figure 4. Typical Load-Displacement Curves.

Influence of Reloading

Two series of reloading tests were performed. After completing the initial uplift test, the pile was allowed to rest for a period of approximately one year. At this time a reloading test was conducted using the same procedure and equipment as used in the initial test. The results of these tests are presented in Table 4. The ratio of reload to initial ultimate capacity shows a wide range from about 0.5 to 2.0. The overall average of all the tests indicates that the ratio of reload to initial load capacity is about 1.1, i.e., no significant difference for this soil. Test Nos. 19 and 20 may represent anomalous data since it appears that the ground water table was considerably lower at the time of the reload tests. This may have produced higher negative pore water pressures in the upper part of the profile, leading to higher effective stresses and therefore higher unit skin friction. Reload tests did show a much higher normalized displacement at failure than did initial loading tests; 13.9 % compared with 4.7% from initial loading tests.

NORMALIZED LOAD-DISPLACEMENT BEHAVIOR

A simple technique of expressing the behavior obtained in load tests is to normalize the load by the ultimate load and to normalize the displacement by the diameter of the micropile. This transforms the load-displacement behavior into nondimensional terms and has been used previously by other investigators (e.g., Tucker 1987; Rollins et al. 1994). An example of this transformation is shown in Figure 5 which gives the normalized load displacement curves for the reload tests for micropiles 17 through 20. It can be seen that even though these four tests gave considerably different values of ultimate capacity, the normalized behavior may be represented by a single family of curves and show nearly identical results up to about 80% of the failure load. Similar results have been obtained by the senior author for small diameter grouted anchor pullout tests in compacted sand. The divergence after this level of loading may be related to the use of the tangent intersection method to define the failure load. This technique is somewhat subjective and is dependent on the maximum displacement obtained during the actual loading test.

An alternative method for defining the failure load is to use a simple hyperbolic model given as:

$$s/Q = a + bs \qquad (1)$$

where: s = displacement, Q = load, and a and b are regression constants. The ultimate capacity is obtained from the inverse slope of this linear relationship as 1/b which is really the load at infinite displacement. This model has also been used extensively to describe the load-displacement behavior of deep foundations and to predict the ultimate load capacity (e.g., Chin 1972, Promboon and Brenner 1981, Neely 1990). It appears that it may also be desirable to describe the normalized behavior using the hyperbolic model. This is still under investigation.

Table 4. Summary of Initial and Reloading Tests.

No.	Site I.D.	Dia. (mm)	Length (m)	Initial Failure Load (kN)	Reload Failure Load (kN)	Ratio R/I
1	U-1	76	1.52	16.6	9.0	0.54
2	U-2	76	3.05	34.9	26.5	0.76
3	U-3	76	4.57	52.7	62.5	1.19
4	U-4	152	1.52	46.0	46.0	1.00
5	U-5	152	3.05	88.4	85.5	0.97
6	U-6	152	4.57	113.6	115.0	1.01
17	GAT-1	76	3.05	13.3	7.1	0.53
18	GAT-2	76	3.05	13.8	9.1	0.66
19	GAT-3	76	3.05	21.1	41.4	1.96
20	GAT-4	76	3.05	22.2	46.7	2.10

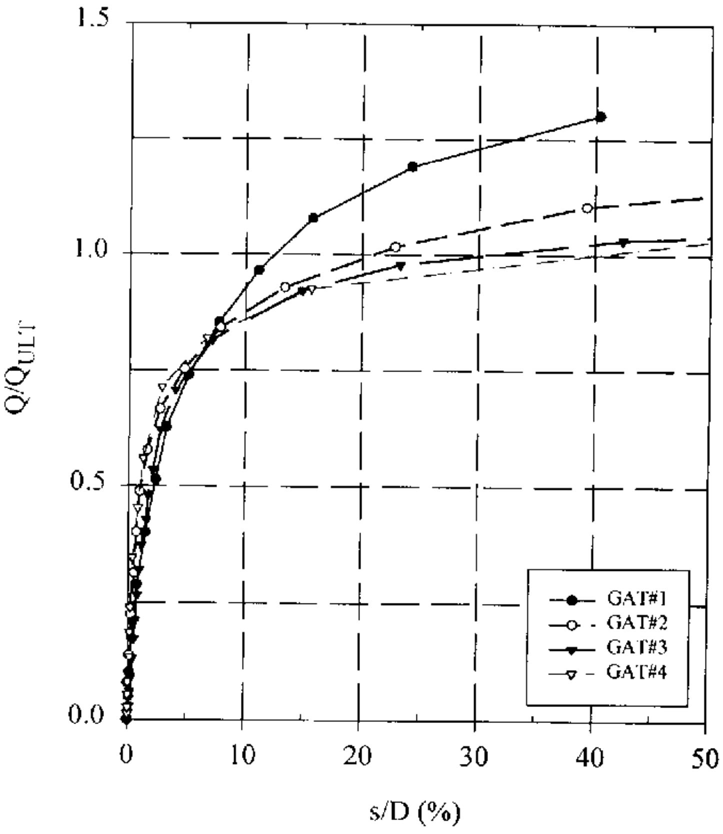

Figure 5. Normalized Load-Displacement Curves.

SUMMARY AND CONCLUSIONS

Tension tests on micropiles installed in a stiff clay indicate that the ultimate capacity can be significantly affected by the drilling technique. While the use of continuous flight augers did not adversely affect the results of the smallest diameter holes tested (76 mm), for 102 and 152 mm diameter holes flight auger holes gave about 50% of the capacity as obtained from hand augered holes. Reload tests performed one year after the initial load tests to failure on average gave almost the same capacity as the initial load tests. There was considerable variation in these tests results which may be related to differences in ground water levels at the different test times. Reload tests showed a much higher normalized displacement at failure which is probably related to the stiffness degradation. In a natural setting this is difficult to control, however it may be that a detailed examination of ground water records may help better explain these results. In the future, it may be essential to have tensiometer measurements within the zone of the foundations in order to evaluate negative pore pressures.

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