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ABSTRACT

The foundation of two bridges was retrofitted using micropiles. The micropiles consisted of hollow core bars installed under limited headroom conditions. Of the total number of micropiles, 180 were installed in submerged sand and 80 were installed in stiff, silty clay. The micropiles were drilled using a lean cement grout which was re-circulated for de-sanding and re-use. Final grout was injected upon completion of drilling to the design tip elevation. The micropiles were subject to a rigorous quality control that included grout quality testing and proof-testing of each production micropile. All production micropiles were proof-tested up to 150 percent of the design load. In addition, four verification tests were performed on sacrificial micropiles to at least two and a half times the design load or to failure.

This paper presents a description of the procedure for installation and quality control of the micropiles, and the results of the verification and proof tests performed for this project. It also provides estimated of bond strength for hollow core bar micropiles in soils similar to those encountered at the project sites. This work shows that hollow core bar micropiles provide a significant unit bond capacity in both granular and fine soils, which may be greater than that typically expected in pressure-grouted (Type B) micropiles in granular soils.

INTRODUCTION

This paper discusses the use of hollow core micropiles to retrofit two existing bridges. Underpinning of the foundations was necessary due to deterioration of the exposed section of existing concrete pre-cast piles. A total of 180 hollow core bar micropiles were installed at Bridge 1 through granular soils, while 80 micropiles were installed through predominantly fine soils at Bridge 2. Bridge 1 is a four-lane, four-span structure supported on three piers, each consisting of thirty 18-inch (45.7 cm) precast concrete piles. Bridge 2 has three spans supported on two piers, each with twenty 18-inch (45.7 cm) precast concrete piles.

Ongoing deterioration of the precast piles required full retrofit of the bridge foundations. The contractor and its designer chose hollow core bar micropiles for several reasons. In traditional micropile installation, drilling is followed by grouting and installation of the reinforcement. Hollow core bar micropiles have faster installation rates in many soils than traditional micropiles because drilling, grouting, and placement of reinforcement are done simultaneously. Although material costs for hollow core bar micropiles may be higher than traditional micropile reinforcing such as threaded bars, faster installation rates often offset these additional costs, especially in cases of difficult access and limited headroom.

The micropiles were designed for an allowable capacity of 80 kip (355.9 kN). The micropiles consisted of hollow core bars bonded to the soil, and had varying bond lengths depending on their location. The upper portion of the micropiles included permanent steel casing to provide buckling and bending capacity along the exposed portion of the micropiles and the potential scour zone.

The micropiles were connected to the existing bridge through new cap beams. The cap beams were constructed in two stages in order to allow proof testing of all production micropiles.
During proof testing, careful measurements of the micropile deflection, as well as movements of the new and existing pile caps were performed. The proof testing schedule included a minimum 12-hour load hold period to verify creep, and several unload-reload cycles.

GEOLOGY

The project sites are located along the southern portion of the New Jersey Turnpike in the Coastal Plain Province of southwest New Jersey. These unconsolidated sediments consist of layers of sand, silt and clay deposited alternately in deltaic and marine environments as sea level fluctuated during cretaceous and Tertiary times (Geologic Map of New Jersey, Geological Survey, 1999). Test borings were performed at the verification test locations at each bridge. Details on the soils at each bridge location are presented below.

Bridge 1

Subsurface conditions generally consist of very soft organic silt to a depth of approximately 10 feet (3 m) below the bottom of the river, underlain by medium-dense sand to a depth of approximately 57 feet (17.4 m). The sand has a fines content ranging from 10 to 30 percent. The Standard Penetration Test (SPT) blow count ranges from 10 to 16 blows per foot in the sand layer. An interval of stiff clay exists below the sand layer. The 30-foot (9.1 m) bond length of the micropiles with a nominal grout body diameter of 6 inches (15 cm) was developed within the medium dense sand layer.

Bridge 2

A layer of very soft organic silt extends to a depth of approximately 14 feet (4.3 m) below the river bottom. The silt is underlain by stiff to very stiff silty clay with SPT values generally ranging between 11 and 16 blows per foot. The liquid limit and plasticity index values of the clay range from 56 to 78 percent and 38 to 58 percent, respectively. The natural water content ranges from 30 to 35 percent. The 40-foot (12.2 m) bond length of each micropile with a nominal grout body diameter of 9 inches (22.9 cm) was developed entirely within the stiff clay.

HOLLOW CORE BAR MICROPILES

The design load at both bridges was 80 kips (355.9 kN) per micropile. The reinforcement of each micropile consisted of one 52/26 IBO-Titan hollow core bar supplied by Con-Tech Systems, Ltd (see Fig. 1). This bar has a cross sectional area of 2.07 in^2 (13.4 cm^2). In the hollow core bar system, the grout is injected at the ground surface through the center hole of the bar. Upon exiting the drill bit at the tip of the bar, the exit grout velocity undercuts the soils and flushes the drill cuttings to the ground surface along the annular space around the bar.

Fig. 1. Hollow core bars.

The completed micropile consists of the hollow core bar as central reinforcement, surrounded by a grout body with a diameter larger than the diameter of the drill bit. The final diameter of the grout body depends on the injection pressure, exit velocity, type of soil, diameter of the drill bit, and other factors. At the perimeter of the grout body, there may be a layer of soil mixed with grout and there may be penetration of the grout as lenses within weaker parts of the formation.

Figure 2 depicts the characteristics of the hollow core bar micropiles used for both bridges. The micropiles also included a 9.625-inch (24.4 cm) external diameter steel casing with 0.5-inch (1.3 cm) wall thickness, which extended from the cap beam to the anticipated scour depth to prevent buckling of the micropiles. The casing was not intentionally bonded to the surrounding ground.

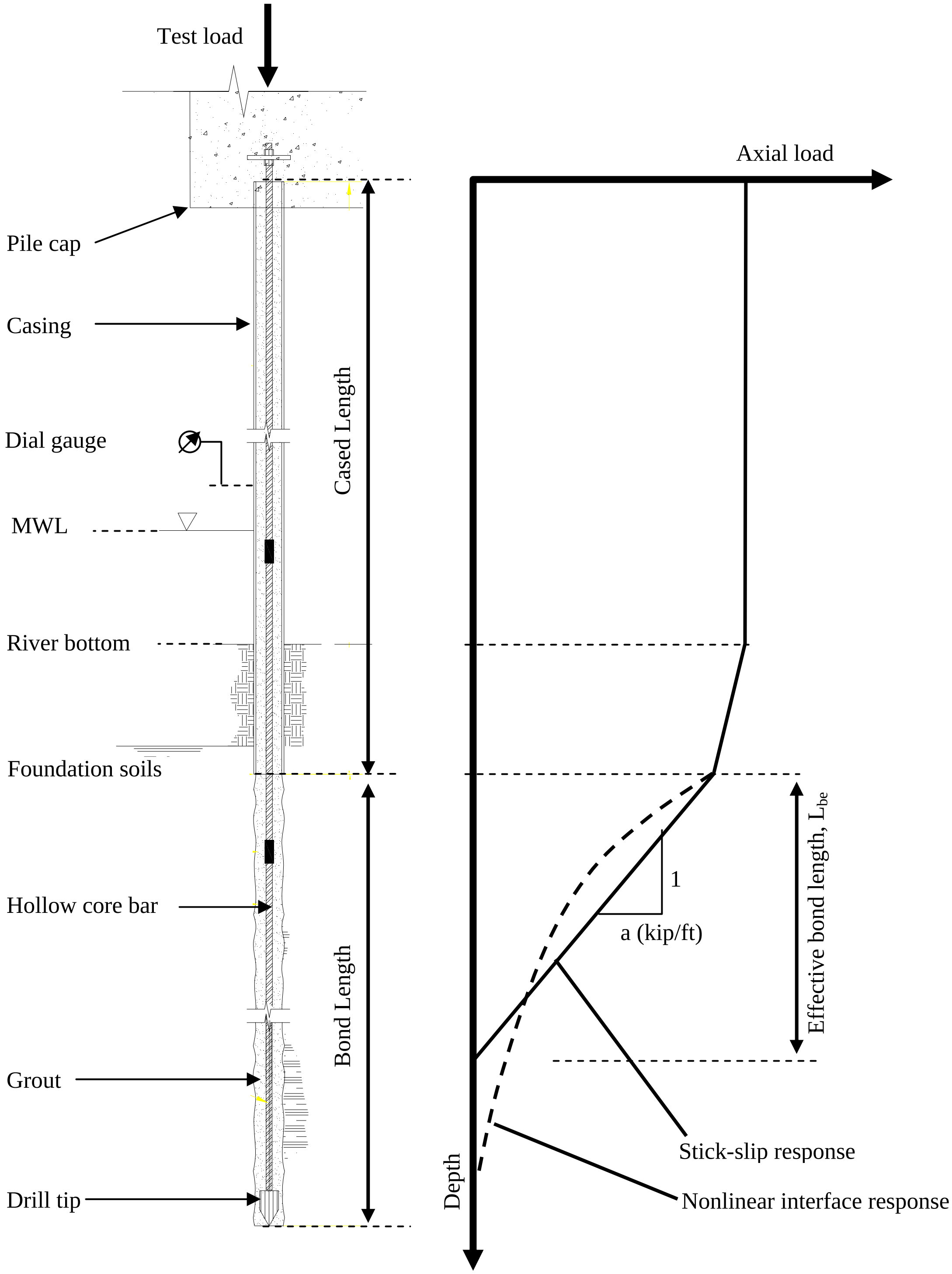


Fig. 2. Production micropile configuration in Bridges 1 and 2, and simplified micropile response model used for interpretation of proof test data.

DRILLING

Access to each structure was difficult, and all work had to take place under limited headroom and within temporary cofferdams. Figure 3 illustrates installation of the production micropiles under low headroom. A Davey DK-525 drill rig was used to install the micropiles. At each production micropile location, the 9 5/8 inch (24.45 cm) steel casing was first installed to the design depth using the external water flush method. During this stage, the water pressures inside the casing were limited to prevent movement of the nearby bridge foundations. The existing bridge was monitored to ensure that external water flush did not generate movement of the structure.

Fig. 3. Installation of production micropiles with limited headroom.

Once the steel casing was in place, the hollow core bar was inserted in 3-, 5-, and 10-foot (0.9-, 1.5-, and 3.0-m) segments until reaching the bottom of the casing (see Fig. 4). The bar was fitted with 6- and 7.5-inch (15.2- and 19.1-cm) clay bits at Bridges 1 and 2, respectively. J-teeth were welded to each sacrificial clay bit to obtain the design diameter and to improve grout flow around the bit (see Fig. 5).

During drilling, the hollow core bars were fed with a lean cement grout mix with a water/cement ratio of 0.89, which resulted in a specific gravity of 1.4 to 1.6 (1 bag of cement per 10 gallons (37.9 L) of water). The viscosity and density of this mix was suitable to flush the hole and carry sand or clay to the surface while advancing the hollow core bar to the desired depth. The bars were advanced with continuous grout flush to the design tip elevation.

Fig. 4. Drilling of test hollow core bar micropile through casing.

Fig. 5. Drill bit fitted to hollow core bar.

The grout was mixed in a colloidal grout mixer in 40 gallon (151.4 L) batches (see Fig. 6). Grout drilling pressures were continuously monitored at the drilling head and ranged from 10 to 50 psi (68.9 to 344.7 kPa). Higher grout pressures, of up to 120 psi (827.4 kPa) are often measured during hollow core bar micropile installation; however, the grout pressure is a function of several variables such as the drill bit diameter, grout viscosity, grout pump type, soil type, and number and diameter of port holes in the drill bit. Therefore, the grout pressure is a useful indicator of consistency and of potential installation problems within one project, and should not be extrapolated to other sites without due consideration of the variables involved.

Fig. 6. Grout plant.

The drilling grout was re-circulated and de-sanded. It was then agitated and re-used for drilling. Recycling of the grout allowed significant savings in cement quantities and limited the impact of the drilling operation on the environment.

FINAL GROUT MIX

Upon completion of drilling, a final grout mix was prepared for a target water-cement ratio of 0.45 (1 bag of cement per 5 gallons (18.9 L) of water). The specific gravity of the grout measured during installation ranged from 1.8 to 1.95, with sporadic values as high as 2.1. Compressive test results on grout cubes were generally higher than 4,000 psi (27.6 MPa) after 28 days. The final grout was pumped through the bar until achieving return at the ground surface through the annular space between the bar and the casing.

QUALITY CONTROL

The designer of the micropiles provided full-time observation during installation of the micropiles. The field personnel logged the drilling rates, grout return, cutting types, grout volume, etc. They also measured the specific gravity of the grout, and prepared grout specimens for compressive testing.

Drilling Rates

Drilling rates were measured during installation of each micropile. The time lag during addition of each bar segment was not included in the time measurements and, therefore, did not affect the computed drilling rates. Drilling rates typically ranged between 0.5 to 1 ft/min (0.15 to 0.3 m/min) in the granular soils of Bridge 1, and 0.3 to 0.7 ft/min (0.09 to 0.21 m/min) in the fine-grained soils of Bridge 2.

The authors found that measurement of the drilling rates was an invaluable tool to confirm the materials encountered and to have firm data for technical discussions with the project team. The drilling rates measured in one site using a specific set of equipment and tools may be used as a measurement of the consistency of micropile installation but must not be directly correlated with those measured at other sites.

Specific Gravity Measurement

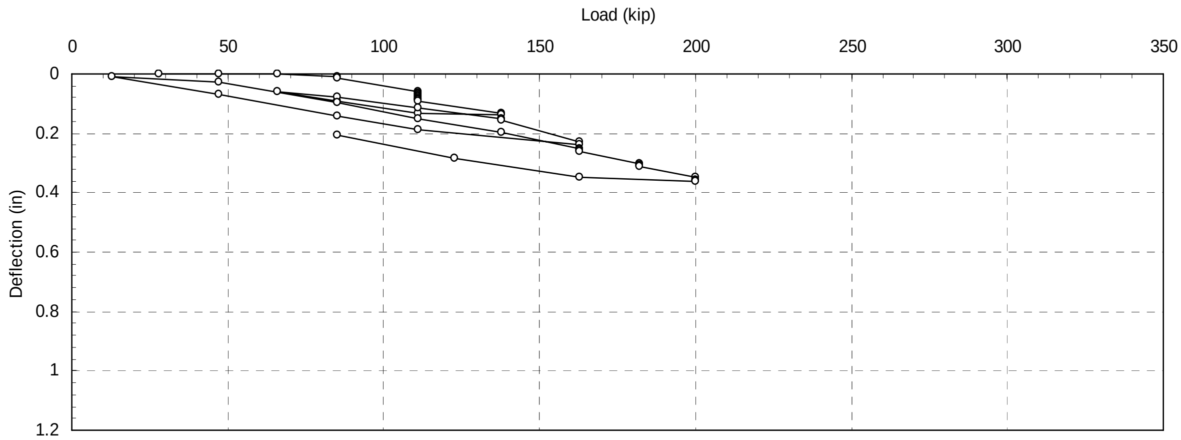
Specific gravity was the primary quality control of the grout. It was measured using a calibrated mud balance according to API RP 13B-1, “Recommended Practice Standard Procedure for Field Testing Water-Based Drilling Fluids”. The minimum specific gravity value was specified at 1.4 for the drilling grout and 1.8 for final grout.

Grout Cube Sample Testing

Grout cubes were tested following ASTM C109, “Standard Test Method for Compressive Strength of Hydraulic Cement Mortars.” Grout cubes were formed in 2-inch (5.1 cm) square polyethylene or brass molds. Cubes were molded from final grout batches at the grout hopper after mixing and confirming the specific gravity to be above 1.8. Grout cubes were also formed from samples of the final grout return at the top of the pile. The project specification called for a strength of 4000 psi (27.6 MPa) at 28 days, which was typically met throughout the project.

VERIFICATION TESTS

Two verification load tests were performed at each bridge location to a maximum test load of 200 kip (889.6 kN) (250 percent of the design load). One of the verification tests at each bridge site was loaded to geotechnical failure. For each load increment, micropile deflections were measured using dial gauges. Each load increment was held for 10 or 20 minutes. A 12-hour load hold was performed at 133 percent of the design load to verify the potential for creep. Figures 7 and 8 show the data obtained from these tests. Interpretation of the results of the verification tests consisted of calculating the average ultimate bond strength based on the results of the tests taken to failure, and on the approximate interpretation procedure discussed subsequently in this paper.



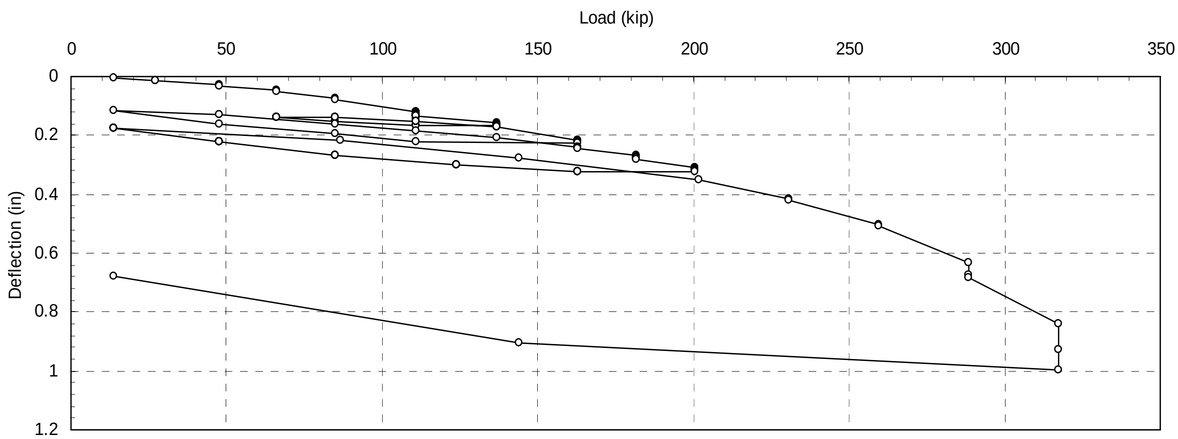


Fig. 7. Load deflection curves from Tests 1-1 and 1-2 at Bridge 1 performed on hollow core bar micropiles installed in sand.

PROOF LOAD TESTING

All 260 production micropiles were proof-tested to a maximum load ranging from 125 to 150 percent of the 80-kip (355.9 kN) design load. The production micropiles were tested in groups of six or eight micropiles. A new cap beam connected the micropiles in each test group (see Fig. 9). Each separate section of the cap beam, which encompassed a group of micropiles for proof testing, was isolated from adjacent portions of the beam, and from the existing bridge structure. Each cap beam section was poured in two stages. The first stage was completed after installation of the piles and before proof testing. Upon completion of the first stage pour, the new cap beam and the existing bridge cap beam were not yet connected. The second stage pour was performed after successful proof-testing of the micropiles and lock off of the jacks, and consisted of filling construction joints between cap beam sections and concrete pouring of the top portion of the cap beam to connect it to the existing bridge structure.

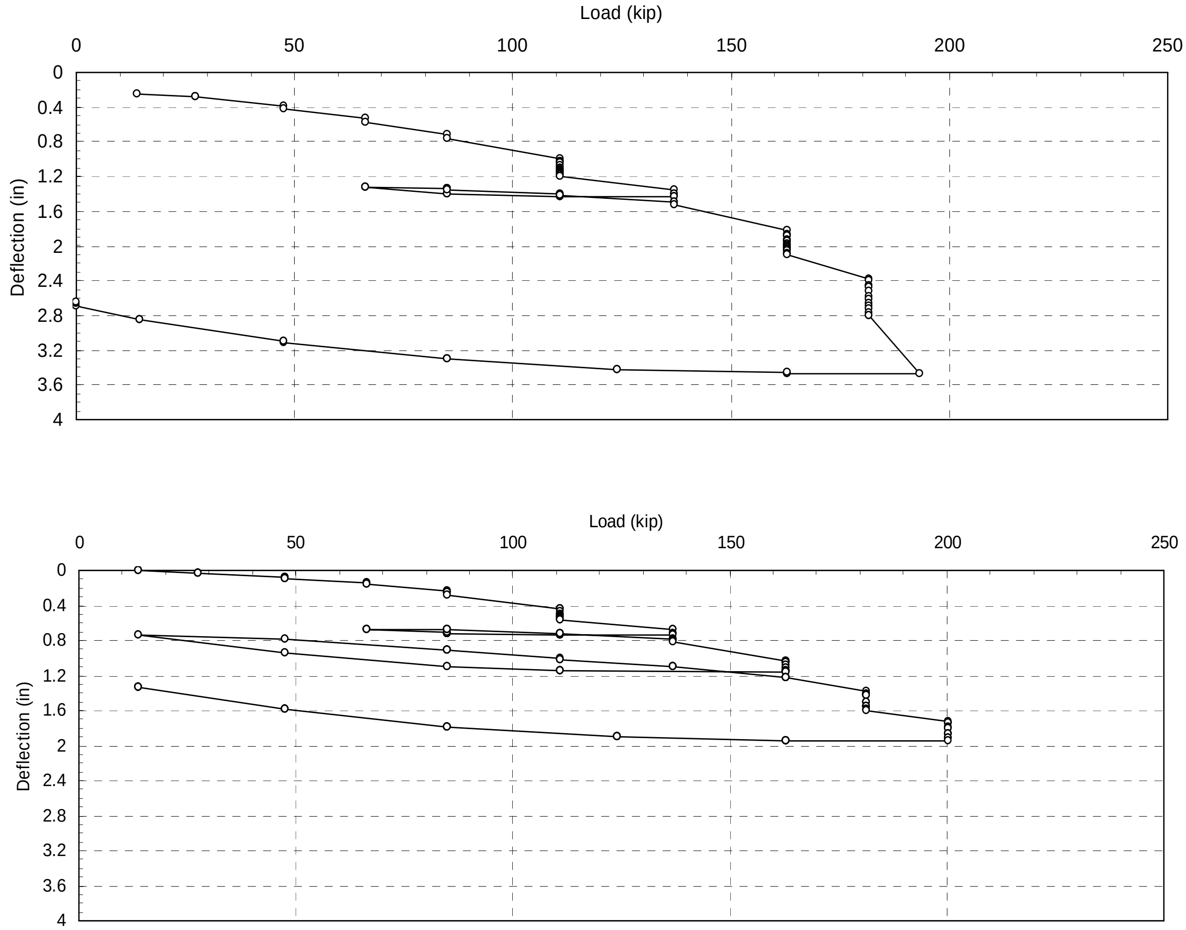


Fig. 8. Load deflection curves from Tests 2-1 and 2-2 in Bridge 2 performed on hollow core bar micropiles installed in stiff silty clay.

Five to six flat jacks were used to load each micropile group. The force on the jacks and the jack location was established so that all micropiles in the group were subject to about the same axial load. The flat jacks were installed within a temporarily open gap between the existing bridge and the top of the first pour of the new cap beam. The deflection of each micropile was monitored individually using dial gauges with resolution of 0.001 inch (0.025 mm) against a stiff reference beam. The dial gauges were set about 3 feet (0.9 m) from the river bottom as depicted in Fig. 2 and Fig. 10.

The existing bridge was used as reaction for the tests. Because the maximum proof load exceeded the dead weight of the bridge structure, the existing piles were subject to uplift. Therefore, movement of the existing cap beam was carefully monitored throughout the proof tests.

The authors developed the deflection curves for each of the production micropiles at Bridges 1 and 2. Figure 11 contains typical load-deflection data from one of the tests. The deflection values under the maximum test loads were generally within 0.1 to 0.2 inches (0.25 to 0.51 cm). The increase in deflection under constant load at 80 kip (355.9 kPa) corresponds to creep during the load hold period. Creep was not significant in most of the production piles, and did not exceed the specified maximum of 0.08 inches (0.20 cm) per log cycle of time in any of the production micropiles.

Figure 12 and Fig. 13 show the range of load-deflection responses for all the proof-tested micropiles. Micropiles installed in the granular soils at the Bridge 1 location were generally stiffer than micropiles installed in the fine-grained soils at Bridge 2. Also, the load-deflection data from micropiles installed in granular soils showed less scatter than those from micropiles in fine-grained soils.

Fig9. View of micropiles and completed cap beam.

Fig. 10. Typical proof load test set up performed to a group of six micropiles.

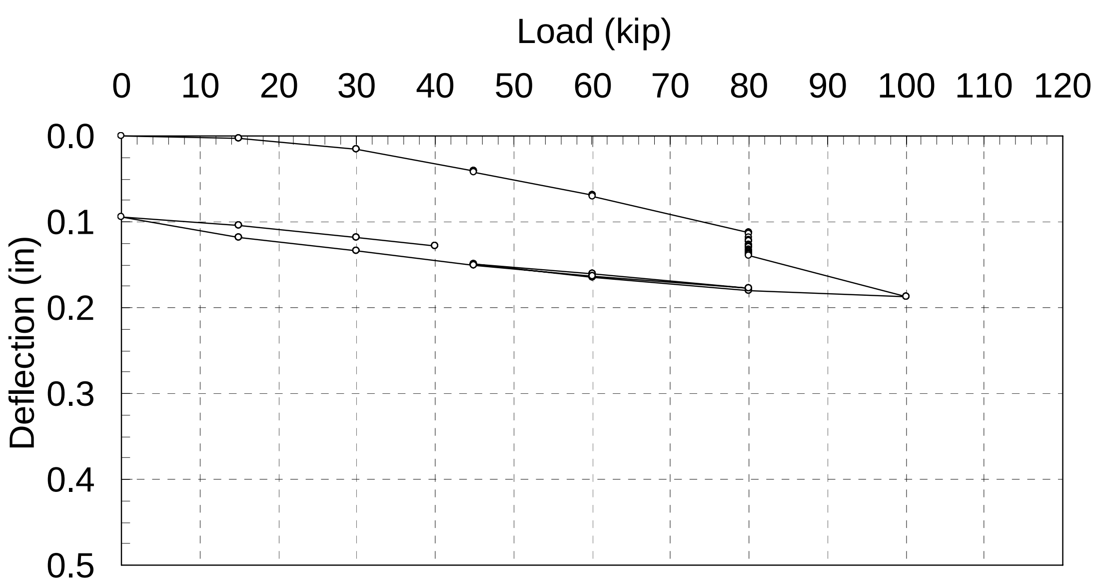


Fig. 11. Typical load-deflection data from one proof test on a hollow core bar micropile at Bridge 1.

INTERPRETATION OF TEST DATA

Apparent Elastic Length

The apparent elastic length, L_e, of a test pile can be calculated for each unloading cycle using the following equation (Gómez et al. 2003):

$$L_e = \frac{\delta_e \cdot \Sigma EA}{\Delta P} \quad (1)$$

where δ_e is the elastic rebound measured during unloading at each cycle, ΣEA is the modulus of the micropile section in compression, and ΔP is the magnitude of unloading calculated as the maximum applied load minus the final load after unloading.

The value of L_e is the apparent elastic length. It represents the length of a free-standing column with identical axial stiffness that undergoes a magnitude of elastic shortening equal to that of the pile under the same load. For micropiles that have a relatively long unbonded, cased portion, it is convenient to define the "effective bond length" depicted in Fig. 2. The effective bond length is the portion of the bond zone where load is transferred to the surrounding ground.

Figure 14 depicts a simplified stick-slip model for the interface between the micropile grout and the surrounding soil. In this model, the bond strength is fully mobilized at any level of relative displacement between the grout and the soil. The corresponding axial load distribution along the bond zone is depicted in Figure 2, assuming that the bond strength is uniform throughout the bond length. For this stick-slip response, the effective bond length is calculated using the following expression:

$$L_{be} = 2\frac{\partial_{be} \cdot \Sigma EA_b}{\Delta P_b} \qquad (2)$$

where δ_{be} is the elastic rebound calculated at the top of the bond zone during unloading at each cycle, ΣEA is the combined modulus of the bond zone in compression, and ΔP_b is the magnitude of unloading calculated as the maximum applied load at the top of the bond zone minus the final load after unloading. The value δ_{be} is not measured directly in a non-instrumented test pile, but can be estimated based on the properties of the cased section. The value ΔP_b is estimated based on suitable assumptions regarding the bond of the cased section to the ground, which is likely small.

The corresponding axial load distribution along the bond zone is depicted in Fig. 2, assuming that the bond strength is uniform throughout the bond length.

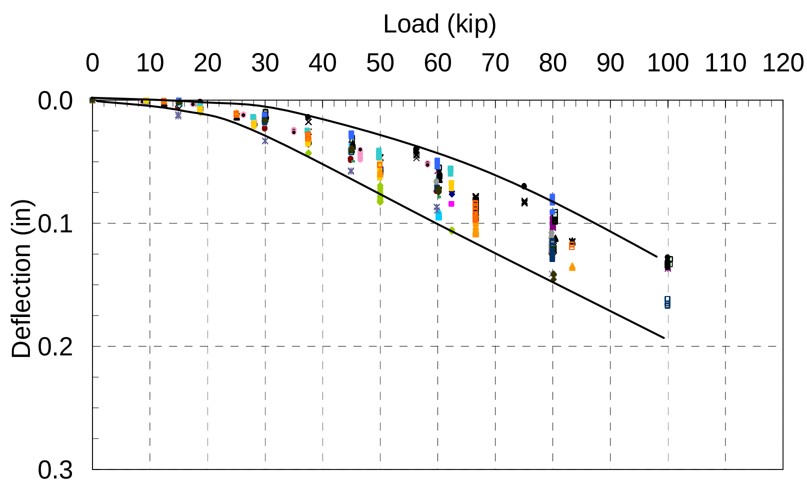


Fig. 12. Summary of all load-deflection test data from proof tests at Bridge 1.

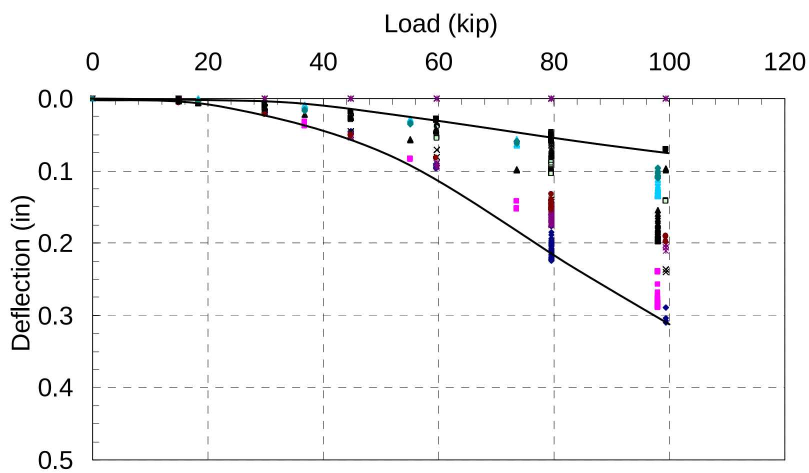


Fig. 13. Summary of all load-deflection test data from proof tests at Bridge 2.

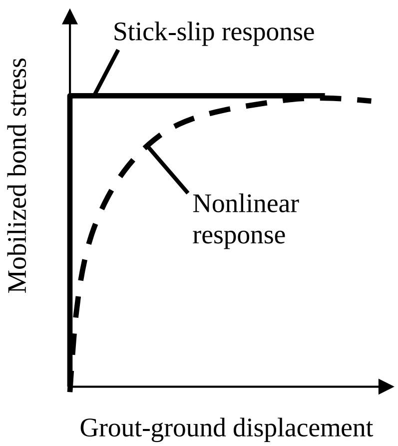


Fig. 14. Simplified grout-ground interface models.

In reality, upon unloading, the micropile still retains some level of elastic deformation caused by locked-in stresses (Gómez et al. 2003). Therefore, the portion of the bond zone where the micropile load is transferred to the ground is longer than the apparent elastic length calculated using equation (2).

Mobilized Bond Strength

For the stick-slip model discussed above, the average load transfer ratio can be calculated as follows:

$$a = \frac{P_b}{L_{be}} \tag{3}$$

where P_b is the estimated axial load at the top of the bond zone, equal to the load at the top of the pile minus the load transfer along the cased length.

As discussed in the previous section, the apparent elastic length given by equation (1) will typically be shorter than the actual load transfer length. Therefore, equation (3) overestimates the actual load transfer ratio. However, for piles that are not loaded close to failure, the average load transfer ratio may be smaller than the ultimate load transfer ratio. This is apparent in Fig. 2 for the axial load distribution corresponding to the nonlinear response. The ultimate bond strength is mobilized only in the upper portion of the bond zone, while only a fraction of the ultimate bond is mobilized along the rest of the bond zone. Consequently, for piles that are not loaded close to failure, equation (3) may be a good estimate of the ultimate load transfer ratio available along the bond zone.

Equation (3) may be used to predict the ultimate capacity of a micropile, or any pile in general, during the first stages of a load test with unloading-reloading cycles. It can also be used, as in this investigation, to obtain an approximate range of ultimate bond strength values based on a large number of proof tests.

MEASURED ULTIMATE BOND VALUES IN SAND

Both verification micropiles at Bridge 1 satisfactorily carried a load of 250 percent of the design load. Verification micropile 1-2 at Bridge 1 was loaded beyond the maximum specified test load until the pile reached geotechnical failure under 318 kip (1414.5 kN). Based on interpretation of the load-displacement curve, the load carried by the 26-foot (7.9 m)bond length of the pile at failure was approximately 260 kip (1156.5 kN), which considered that the upper casing carried approximately 58 kip (258.0 kN). The average load transfer ratio at failure was then 10 kip per linear foot (145.9 kilonewtons per linear meter) of bond zone, equivalent to an average ultimate bond strength of 44 psi (303.4 kPa) considering the nominal grout body diameter of 6 inches (15.2 cm).

Equations (2) and (3) were used to estimate the average load transfer ratio based on data from unload-reload cycles at a load of 137 kip (6049.4 kN). The estimated load transfer ratio was 12.4 kip/ft (181.0 kN/m), which is similar to the measured 10 kip/ft (145.9 kN/m).

MEASURED ULTIMATE BOND VALUES IN STIFF SILTY CLAY

The average ultimate bond strength along the clay layer on Bridge 2 was estimated from the results of Verification Load Test 2-3, which reached geotechnical failure under 182 kips (809.6 kN). Considering that the upper casing carried approximately 20 kip (89.0 kN) at failure, the load carried by the 40-foot (12.2 m) bond zone of the pile was 162 kip (720.6 kN). The corresponding ultimate bond strength was 4.1 kip per linear foot (59.8 kN per linear meter) of bond zone, or 18 psi (124.1 kPa) considering a nominal diameter of the grout body of 6 inches (15.24 cm).

Equations (2) and (3) were used to estimate the average load transfer ratio based on data from unload-reload cycles at a load of 137 kip (609.4 kN). The estimated load transfer ratio was 5.9 kip/ft (86.1 kN/m), which is similar to the measured 4.1 kip/ft (59.8 kN/m).

Based on the results of this verification test, the drill bit diameter was increased. Load test 2-3a was successfully completed on a sacrificial micropile with a nominal grout body diameter of 9 inches (22.9 cm).

ULTIMATE BOND VALUES INTERPRETED FROM PROOF TESTS

Figures 15 and 16 summarize the load transfer ratios estimated using equations (2) and (3) applied to each of the proof tests. The data obtained during the last unloading cycle of each test was used for these calculations.

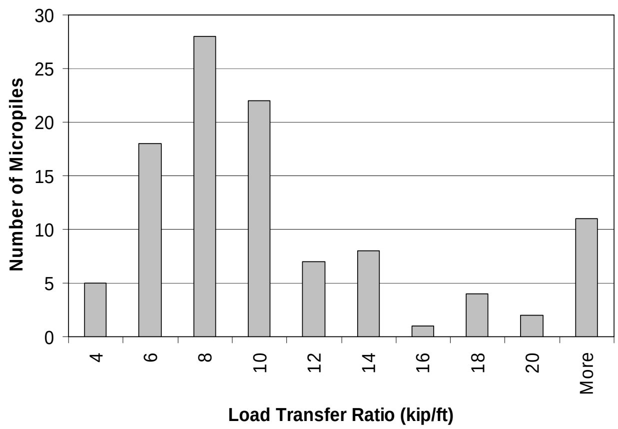


Fig. 15. Histogram showing load transfer ratio in hollow core bar micropiles installed in sand.

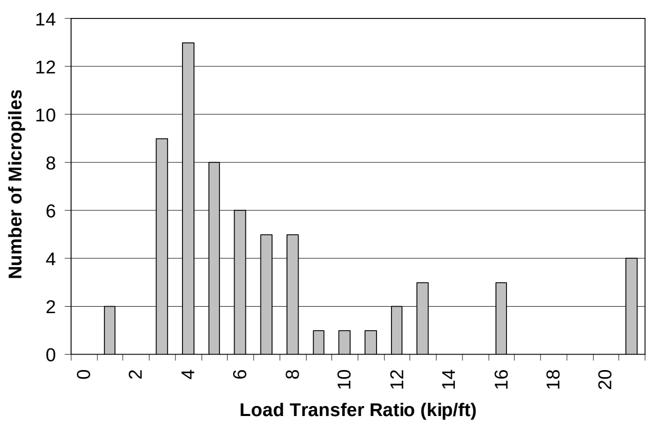


Fig. 16. Histogram showing load transfer ratio in hollow core bar micropiles installed in stiff silty clay.

A simple inspection of the histograms shows that the load transfer rate of the micropiles installed in sand ranged from 6 to 12 kip/ft (87.6 to 175.1 kN/m), which is equivalent to 27 to 53 psi (186.1 to 365.4 kPa) considering the nominal grout body diameter of 6 inches (15.24 cm). For micropiles installed in stiff silty clay, the load transfer ratio ranged between 3 to 9 kip/ft (43.8 to 131.3 kN/m), which is equivalent to 9 to 27 psi (62.1 to 186.2 kPa) considering a nominal grout body diameter of 9 inches (22.9 cm).

The reader must be aware that the nominal grout body diameter was estimated as 1.5 inches (3.8 cm) larger than the drill bit diameter. Therefore, the values given in Figs. 15 and 16 may need to be adjusted if using a different approach to estimate the grout body diameter.

COMPARISON TO PUBLISHED DATA

The bond strength values estimated from the results of verification load tests and proof load tests are summarized in Table 1, and compared to ultimate bond strength values for Type-B micropiles suggested by the Micropile Design and Construction Guidelines, Publication No. FHWA-SA-97-070, 2000. The bond strength estimated for the hollow core bars in the granular soils at the Bridge 1 site are larger than those values suggested by FHWA. This may be due to the beneficial effect of partial mixing of soil and grout in the periphery of the grout body, and to the penetration of the grout into the soil mass outside the micropile. The bond strength values estimated for the micropiles installed in the fine-grained soils of Bridge 2 are within the range of values proposed by the FHWA manual.

Table 1. Summary of Estimated Bond Strength Mobilized Along Hollow Core Bar Micropiles

Soil Type	Mobilized Bond Strength in Proof Tests	Bond Strength Suggested By FHWA Type B micropile
Sand (some silt) (fine, loose-medium dense)	27 – 53 psi (186.2 – 365.4 kPa)	10 – 28 psi (68.9 – 193.1 kPa)
Silt & Clay (some sand) (stiff, dense to very dense)	9-27 psi (62.1 – 186.2 kPa)	10 – 28 psi (68.9 – 193.1 kPa)

CONCLUSIONS

Interpretation of the verification and proof tests showed that the ultimate bond strength of the hollow core bar micropiles installed in sand was significantly larger than that typically used for design of Type B, pressure-grouted micropiles.

In stiff silty clay, the ultimate bond strength values obtained from the tests were very similar to those typically used for micropile design in this type of soils. Micropile design loads must always be verified through suitable load testing in each project.

The success of Bridges 1 and 2 has led to the opportunity for continued work on Bridges 3 and 4. Work at Bridges 3 and 4 will provide additional information pertaining to the installation and testing of 144 additional hollow core bar micropiles.

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REFERENCES

Geologic Map of New Jersey, Geological Survey, 1999.

Gómez, J.E., Cadden, A.W., Bruce, D.A. (2003), "Micropiles in Rock. Development and Evolution of Bond Stresses under Repeated Loading," 12th Pan-American Conference on Soil Mechanics and Geotechnical Engineering, Cambridge, MA, June 2003.