Micropiles Performance Based Design: Evaluation of Current Design Methods Applied to Puerto Rico Soils

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Abstract *— Micropiles have been used mainly as foundation support elements to resist static and seismic loads. The use of micropiles have been increased not only in locations with poor soil conditions, but also in congested areas and those requiring a less obtrusive presence than other installation methods may provide. This investigation reviewed results from tension tests performed on different soil scenarios and compared results of ultimate capacity with tension results. Comparison between actual capacity versus design values showed micropiles underestimation on micropiles installed in rock, opposed to an overestimation when installed in cohesive soils. Economic impact of overestimation has been approximated to 12% increment, based on one of the cases studied on this investigation. On the overestimation on rock anchors, further investigation is needed to determine the economic impact. This initial step into defining soil to grout bonding of micropiles for Puerto Rico soils sets place for more detailed investigations, targeted to provide better design values, hence reducing cost impacts on the construction phase of this elements.*

Key Terms *— Foundation, Geotechnical Capacity, Grout to Ground Bond, Micropile, Tension Tests.*

TECHNICAL BACKGROUND

Micropiles are a deep foundation element constructed using high-strength, small-diameter steel casing and/or threaded bar. In general, these are high-performance, high-capacity drilled and grouted piles with diameters typically less than 12". These micropiles can extend to depths of 200 feet and can take loads as small as 3 tons or as high as 200 tons. These elements are commonly used for foundation underpinning, reducing foundation settlement, or deep foundation installation in difficult soil makeup.

Micropiles have been used mainly as foundation support elements to resist static and seismic loads, and to a lesser extent, as in-situ reinforcements to provide stabilization of slopes and excavations. Micropiles are seeing increased use not only in locations with poor soil conditions, but also in congested areas and those requiring a less obtrusive presence than other installation methods may provide. Because of the size of the equipment used for the installation, this system can be used in tight spaces inaccessible for some machinery. The installation process is also relatively quiet as a drilling technique, rather than a hammering one. A micropile is constructed by drilling a borehole, placing steel reinforcement, and grouting the hole, see Figure 1.

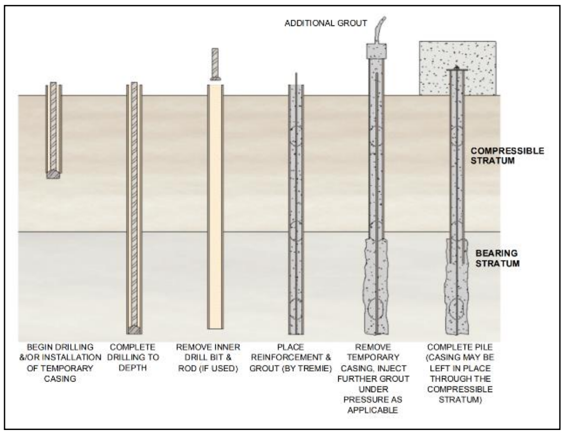

Figure 1
Examples of Micropiles Sections

Although introduced circa 1950, formal attention to the design methods began in the early nineties in an effort of Federal Highway Administration (FHWA) to review the "state of the practice", resulting in the technical Document:

"Drilled and Grouted Micropiles – State-of-the-Practice Review" [1]. The document provided a comprehensive international review and detailed analysis of available research and development results, laboratory and field testing data, design methods, construction methodologies, site observations, and monitored case studies. As part of this study, the limitations and uncertainties in the state-of-the-practice were evaluated, and further research needs were assessed. One of the highlighted needs was a manual of design and construction guidelines intended for use by practicing highway agency geotechnical and structural engineers. In response to this need, the FHWA sponsored the development of the manual "Micropile Design and Construction Guidelines, Implementation Manual" [2].

These documents are the general reference for designers when using micropiles applications. The design of these elements is controlled by the geotechnical capacity and the grout to soil bonding. Although the Design Manual provides with ranges of values for the soil to grout bond (α_{bond}), these are broad and leave the matter to the interpretation and experience of design engineer.

The maximum axial loads applied at the top of a micropile must be resisted by grout to ground bond over a specific length of the micropile. This is defined as the *geotechnical capacity* and is defined as:

$$P_{(G\text{-}allowable)} = (\alpha_{bond} * \pi * D_b * L_b)/FS \qquad (1)$$

Where:

α_{bond}: grout to ground ultimate bond strength

FS: factor of safety applied to the ultimate bond strength (typically from 1.5 to 2)

D_b: diameter of the drill hole

L_b: bonding length

The bonding length (L_b) is referred to as the bond/contact zone or bond/contact length. This length depends on the grout to ground bond strength that can be developed in a given earth material. A critical part of the design process is the determination of this bonding length. This is the most sensitive part of the design, since the grout to ground bond values are dependent of the type of soil and the density/consistency of the soil.

The Design Manual [2], in Table 5-3, provides ranges of grout to ground bond (α_{bond}) for different soils and consistency. Reader is directly advised on judicious use of the ranges of this parameter, suggesting the use of average values for those engineers that are familiarized with the behavior of the soils or to use the lower bound values where granular soils are loose or when the working with cohesive soils of medium to high plasticity. This is covered on page 5-19 of [2]. A simplified version of Table 5-3 of [2] is presented in Table 1, below.

Table 1
Summary of Table 5-3 [2], Showing the Recommended Grout to Ground Bond Values for Different Type Soils

Soil / Rock Description	Grout-to-Ground Bond Ultimate Strengths, kPa (psi)			
	Type A	Type B	Type C	Type D
Silt & Clay (some sand) (soft, medium plastic)	35-70 (5-10)	35-95 (5-14)	50-120 (5-17.5)	50-145 (5-21)
Silt & Clay (some sand) (stiff, dense to very dense)	50-120 (5-17.5)	70-190 (10-27.5)	95-190 (14-27.5)	95-190 (14-27.5)
Sand (some silt) (fine, loose-medium dense)	70-145 (10-21)	70-190 (10-27.5)	95-190 (14-27.5)	95- 240 (14-35)
Sand (some silt, gravel) (fine-coarse, med.-very dense)	95-215 (14-31)	120-360 (17.5-52)	145-360 (21-52)	145-385 (21-56)
Gravel (some sand) (medium-very dense)	95-265 (14-38.5)	120-360 (17.5-52)	145-360 (21-52)	145-385 (21-56)
Limestone (fresh-moderate fracturing, little to no weathering)	1,035-2,070 (150-300)	N/A	N/A	N/A
Sandstone (fresh-moderate fracturing, little to no weathering)	520-1,725 (75.5-250)	N/A	N/A	N/A
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	1,380-4,200 (200-609)	N/A	N/A	N/A

Table Notes:
Type A: Gravity grout only (All Cases on this Study)
Type B: Pressure grouted through the casing during casing withdrawal
Type C: Primary grout placed under gravity head, then one phase of secondary "global" pressure grouting
Type D: Primary grout placed under gravity head, then one or more phases of secondary "global" pressure grouting.

OBJECTIVE OF INVESTIGATION

The purpose of this project is to optimize the procedures established by the Federal Highway Administration for micropiles designed on Puerto Rico soils. This project is limited to study micropiles tensile axial capacities and does not intends to cover laterally loaded elements. The scope is limited in general to rocks, non-cohesive residual and cohesive residual soils. The main outcome expected is to identify if the parameters provided in the design manual by FHWA are suitable for the three types of soils under study. The project does not intend to set a standard for analysis, but to demonstrate that the use of the available parameters should be done properly and that there is a need to prepare a local database for the grout to ground bond values (α_{bond}) for the soils in Puerto Rico.

APPLIED METHODOLOGY

The tension test methodoly varies, depending on the application or structural engineer specific requirements. A summary of the Quick Load test procedure from [2] follows:

- Load increments are applied and measure by means of a calibrated hydraulic jack and pressure gauge. The pressure gauge shall be graduated to 72 psi increments or less. The jack and pressure gauge shall have a pressure range not exceeding twice the anticipated maximum test pressure. Jack ram travel shall be sufficient to allow the test to be done without resetting the equipment.
- Measure the pile top movement with a dial gauge capable of measuring to 0.001 in. The dial gauge shall have a travel sufficient to allow the test to be done without having to reset the gauge.
- Visually align the gauge to be parallel with the axis of the micropile and support the gauge independently from the jack, pile or reaction frame.
- Use a minimum of two dial gauges when the test setup requires reaction against the ground or single reaction piles on each side of the test pile.
- Once data has been collected and reduced, a scatter plot graph is used to define the load vs. displacement.
- The plotted data was evaluated, and analyzed in detail, to define the discussion and further conclusions.

Figure 2 provides a general set up configuration for a tension test.

For this project three different soil scenarios, with grouted anchors installed were evaluated:

- Residual Cohesive Material
- Residual Granular Material
- Weathered Rock

The field tests were provided by professional practicing firms, together with the corresponding geotechnical data. The method for analysis will implies the use of the stress vs. deformation curves generated from each case. Our intension is to perform a reverse engineering analysis to determine the α_{bond} values used on these designs and cross reference those values with Table 1. Geotechnical capacity model will be calibrated, by solving the soil to grout bond values based on the tension capacity results obtained from the tests using Equation (1). The interpretations of the results will be followed by a Performance Based Design of the micropiles, in other words, the results from the tests will be evaluated with grout to ground values that approximate to the field test results.

DATA COLLECTION

Data was collected from two local companies that perform independent testing to micropiles systems. The tests were performed on actual projects with design parameters for each of the systems. Three different scenarios were compiled, residual soils with, medium plasticity, residual soils with non-plastic behavior, and partially weathered rock materials. The data collection was designed to test the anchors for the ultimate capacity. Due to the nature of the soils and the design loads the target load in the tests vary from location to location, as shown in Table 2. However, this is not interpreted as bias, since each of the sites was evaluated individually and based on the design loads and the actual performance obtained from the load vs. elongation plots.

Data from four (4) different projects was used. For practical purposes the general location and geologic setting is discussed for each test site, as means to understand the soil genesis and type of materials.

Table 2
Test Location, Geologic Setting and Target Loads

Location	Soil Media	Target Load
Rio Grande	Silts and Low Plasticity Clays from Felsic Rocks	45 kips
Caimito	Volcanoclastic Rocks	133 kips
Carolina	Basaltic Residuals	293 kips
Hato Rey	Quaternary Deposits	150 kips

Figure 2
General Test Set Up

Geotechnical data was available for all the projects. However, for one of the sites (Hato Rey), the depth of the geotechnical investigation borings was less than the total length of the micropile design depth. The implications of this will be discussed to further details in the results discussion.

Calibrated loads cells with capacity of up to 200 tons in tension were used for the tests. The tests were performed following general guidance provided in [2], following the "Quick Load Procedure". The data was recorded manually, reading the displacement for the intended load. The loads were applied in increments until achieving target load, followed by the corresponding unloading of the system. The residual displacement was recorded to evaluate performance of the pile, given this could be a failure parameter if allowable values defined by the structural engineer are exceeded. Figure 2 depicts a general set up of the equipment in one of the actual tests performed, at Carolina Site.

Based on the geotechnical investigation data for each of the sites, estimated values of the ultimate tension capacity for the piles could be calculated using Equation (1). Given the length and hole diameter are known, the value missing to estimate a capacity is the α_{bond}. Reader will notice that from Table 1 the capacity values vary considerably for each soil. For practical purposes, we performed a capacity prediction for each of the scenarios based on the low, average and high value of α_{bond}. The results are presented in Table 3 below. Once the field data is evaluated, these baseline values will be used in the analysis of results.

Table 3
Estimated Capacity in kips for the assigned range of α_{bond}

Location	Low	Average	High
Rio Grande	24	48	72
Caimito	622	1659	2525
Carolina	664	1327	2020
Hato Rey	56	121	186

FIELD TEST RESULTS

Each individual data set was plotted using spreadsheet assistance. For each of the sites the graph shows a loading and unloading phase. The loading illustrates the behavior of the bar while load increments are applied. The unloading, which is observed as generally a single slope line, illustrates the recovery of the system in the elastic range. A residual deformation is recorded on each site. This residual deformation is used by engineers as means of acceptance criteria for each individual project. For this investigation the behavior and results of the residual deformations was not considered. Residual deformation is a parameter assigned by structural engineers in their design to define the allowable deformations of the system, and to predict creep behavior of the soils. None of this are at this moment within the scope of the investigation, focusing in the grout to ground bond. For practical purposes, reader is advised that for performing piles capable to withstand the target loads, this value shall be relatively small. Large residual values imply poor recovery of the system in the elastic range and in most cases is deemed a reason for failing tests.

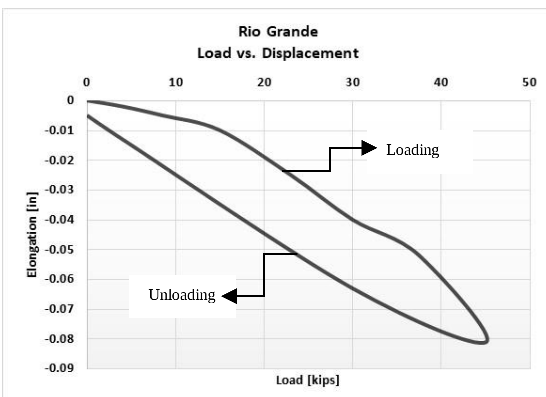

Figure 3
Rio Grande Load vs. Displacement Results

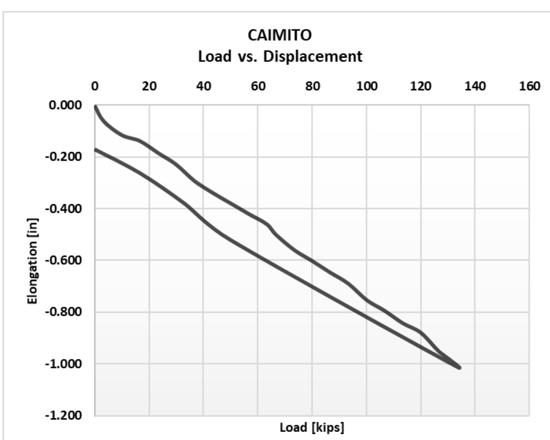

Figure 4
Caimito Load vs. Displacement Results

Figure 5
Colobos Load vs. Displacement Results

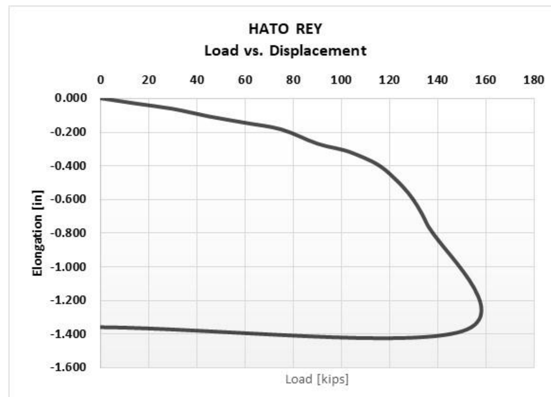

Figure 6
Hato Rey Load vs. Displacement Results

Graphical results depict that all but the Hato Rey site were able to manage the target loads defined in Table 2. Different behavior in of the cure patterns could be observed as well as the residual displacement recorded. The evaluation of results, together with the analysis are presented in the next section.

RESULTS ANALYSIS

Each of the scenarios was evaluated individually, comparing the field test results to the estimated capacity values. The following is a discussion and analysis of the results observed.

Rio Grande

The test was performed for a target load of 45 kips. This was the location were the lowest load was applied. The graphical results in Figure 3 reveal three different slope changes [3]. The first section could represent the alignment of the loads in the system with the deformation recorded being vertical displacement of the soil that is using for the reaction of the cell. The second interval could be associated the bar elongation as load is increments. This interval is likely to represent the actual deformation of the grout to ground interphase. The deformation recorded was on the elastic range, this is supported by the small residual elongation recorded at the end of the test. The tested pile on this location was a production pile, hence the element was not tested to failure.

Comparing the capacity values in Table 2, to those obtained from the field test, show that the design its likely to have followed the recommendations of FHWA to use average values when estimating capacity. If table 2.1 is followed using (1), pile capacity would be in the order of 56 kips. To properly define the limit value of ground to grout bond, an additional test, pulling a micropile to failure would be required further evaluate these soils behavior.

Caimito and Colobos

On both sites, the estimated ultimate capacity values defined by the designer for testing where considerably lower than the actual performed capacity during testing [4]. See Figures 4 and 5 for the load vs. displacement graphical representation. The results for this micropiles depict a linear behavior with a quasi linear rebound, implying the test was in the elastic range all the time and that the tested micropiles were not loaded to failure.

If Equation (1) is revisited with the values suggested in [2] for rock, the ultimate design capacity would be greater than 2,000 kip for both cases. It is evident that designer underestimated the materials assigning α_{bond} values of dense gravel to rocks. The capacity developed by the element in rock is high. To be able to define the actual α_{bond} for this earth materials, shorter elements with a smaller hole diameter would be needed. The results suggest underestimation and overdesigning of the micropiles for both scenarios tested in rock, with an approximate F.S. of 6 [5].

Hato Rey

The interpretation of the data on the Hato Rey site provides more valuable information than the previously discussed. The fact that the micropile failed provides an ultimate value to compare to the design. Per design criteria, the estimated ultimate capacity value for testing was 150 kips. When referring to Table 2, reader will notice that this value is in between the average and high range estimates, from 121-186 kips. Failure from the graph occurred borderline at 137 kips, hence failing close to the average estimated on table. This is of interest for this investigation, since it suggests that even if following the advice in [2] to use average values could result in an overestimation of the micropile capacity, at least on cohesive soils.

Per previous discussion, the geotechnical exploration for the site did not extended beyond 30-ft deep. Hence, the capacity analysis would relay on equal or improving capacity values of soil to grout bond (α_{bond}) from 30 to 55-ft. Reportedly [6], the soils conditions below 30 feet were less competent, based in the progress of the drill rig at that depth. The cuttings observed during the drilling suggest that the type of soil did not change, but that the consistency was of soft soils [6]. With this consideration the pile would have an ultimate capacity of 137 kips. When referring to the graphic behavior on Figure 6, the load vs. elongation curve, correlation between this value and the displacement-based failure could be defined. It is evident that the designer overestimated the capacity of the α_{bond} while using the average values.

CONCLUSION

The following conclusions could be presented after the data analysis:

- The grout to ground bond values in rock were underestimated. Values corresponding to gravel were used for rock. Both geotechnical and structural engineers without a solid background working with rocks, might tend to assign general properties to rock. Such is the case for instance when working with limestones, “weak rock" and sandstones. The literature inclines to oversimplify a broad group and assign and ample margin of values. Designers are likely to use lower values knowing a safe design will result. However, this result in an apparent need for longer micropiles ending up increasing the construction cost. Diameters for micropiles are in general no greater than two times the diameter. Such was the case of the Caimito and Colobos Sites. The underestimation of the ground to grout values resulted in longer micropiles. A test to failure would be required to quantify the actual a bond value for these materials. This could be part of further investigation.

- The α_{bond} values on residual cohesive soils were overestimated for silts and clays as demonstrated on the Hato Rey Site. Per Figure 6 the ultimate value was around 137 kips before failure at the Hato Rey Site. The overestimation could be based in the confidence of improving conditions with depth, a condition that was not observed during the installation. For this specific case, the overestimation resulted in the addition of 10% the original length of 55 feet, an additional load test, and the down time needed for the additional materials to arrive. This cost was directly transferred to client.
- The only site where an α-bond could be determined based on the testing was on the Hato Rey Site. The reverse engineering analysis suggests an α-bond of 9 psi, a value in between the low and average range suggested in Table 1. On the Hato Rey case this had an impact in the project foundation part in the order of 12%, in dollar amount it represented an additional cost of $12,000USD, plus downtime costs. This project consisted of six production micropiles, if projecting this to a bigger size project the impact could had been sizeable.
- Both previous conditions need attention from the value engineering point of view. In both cases the interpretation of results, over estimation, or underestimation of the soil to grout bond, resulted in direct costs to the project owner.

Micropiles are a solution to many engineering challenges but are a costly solution. Underestimating results in the installation of longer piles or bigger diameter borings, results in a cost charged to project cost due to overdesigning. In the other hand, failing results imply the need of additional test and redesigning, both costly and time-consuming process. Such a situation could result in significant changes to design, cost and calendar. None of these two falls within engineering best practices.

The definition of geotechnical parameters to structural design is a delicate task. Lack of an appropriate subsoil model could lead to failing results due to overestimation, such as the case of the Hato Rey Site. Additional work is needed in the Hato Rey site. The fact that the ultimate capacity was tested, a deeper soil boring with continuous sampling is recommended. Such condition would allow to define finite sections of changes in the soil and to create a multilayer model to estimate the capacity. This would be the last step into defining what are the closest α_{bond} values on this type of soil. A laboratory program on the soil samples could also assist to better define the relationships in (1).

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