Design and Construction of a Telecommunications Tower Foundation Using Micropiles

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Abstract ***—*** *The design and construction of a self-supported tower (three-legged tower) foundation was performed. An existing tower affected structurally by hurricane Maria is located in a very remote location in Yabucoa, making the replacement of the tower a very challenging project. The replacement of this tower is vital for the reconstruction and re-establishment of the cellular communications in Puerto Rico. The use of micropiles was chosen because a relatively small equipment is needed to construct the micropiles. Also, the amount of material needed to complete the micropiles is small as well. This makes it a very suitable alternative for this very difficult vehicular access project. Cost effectiveness and constructability was also a very important aspect when choosing the micropiles. The geotechnical investigation, structural design and construction of the foundation supported with micropiles was successfully completed. A load test was also performed without geotechnical or structural failure. It was established that the Granodiorite Rock, geology of site and a vast area in the eastern part of Puerto Rico, is capable of developing relatively high grout-to-ground bond capacities.*

Important Terms *— Axial Capacity, Compression Load, Geotechnical Capacity, Granodiorite Rock, Grout to Ground Bond Value, Micropiles, Pile Cap, Shear Load, Structural Capacity, Tension Load, Tension Load Test.*

INTRODUCTION

On September 20, 2017 Category 4 Hurricane María made landfall in the town of Yabucoa, Puerto Rico. María caused enormous damages to Puerto Rico's infrastructure. Even though the electrical power grid was the most affected, the telecommunications industry with its high rise towers was affected as well. The existing self-supported tower located at La Pandura Sector in Yabucoa suffered some structural damages and needed to be replaced for a new tower capable of resisting the high magnitude wind loads according to prescriptions in ANSI/TIA-222-G [1].

The scope of work for this project consisted in providing a foundation alternative suitable for the very difficult access site. Typical foundation alternatives for this type of project are mat foundations, drilled shafts (caissons) and micropiles with corresponding pile cap. Large trucks such as concrete mixer trucks can access the site but with great difficulty and safety concerns. Contrary to the micropiles, the mat foundation and drilled shafts alternatives required relative large amounts of concrete. As a result, the micropiles was selected to minimize the concrete volume and concrete mixer trucks trips.

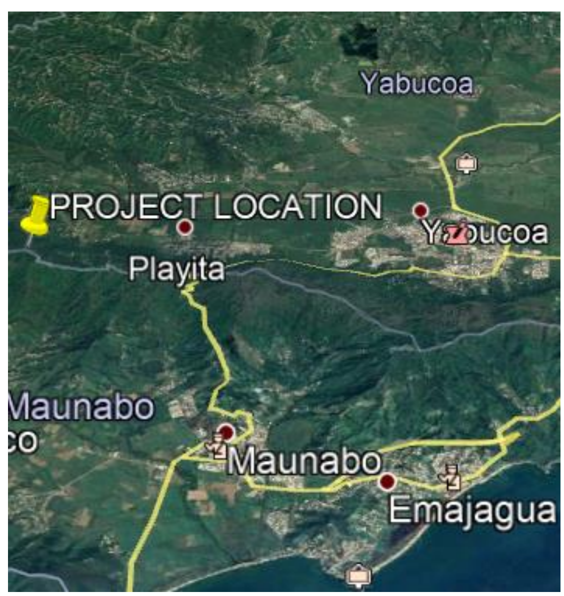

Figure 1
Project Location

OBJECTIVE

The objective of this project was to design and build a pile cap supported by micropiles. The geotechnical investigation, structural design and construction of the foundation needed to be provided. A value engineering analysis shall be performed to justify the alternative selected.

PROJECT DESCRIPTION

The proposed project consists in the design and construction of a foundation system to support a telecommunication self-supported tower, with a height of about 200-feet as measured from ground surface. See Figure 2.

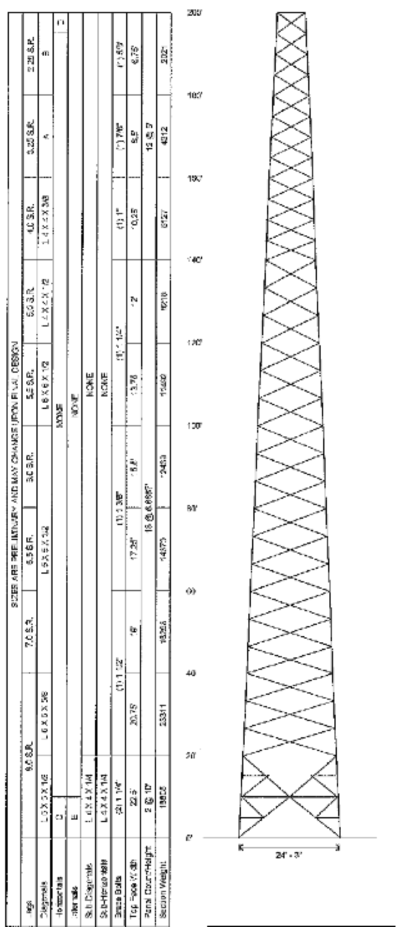

Figure 2
Proposed Tower Profile

Maximum factored tower reactions according to tower manufacturer[1] are as follows:

- Maximum Shear: 223.62 kips per tower leg.
- Maximum Compression: 1,995.0 kips per tower leg.
- Maximum Uplift: 1,797 kips per tower leg.

The design and construction shall be done to suffice these tower reaction loads. The geotechnical investigation, foundation structural design and construction of the selected foundation system shall be provided. A value engineering analysis shall be provided to justify the foundation system selected.

SITE GEOLOGY AND SUBSOIL CONDITIONS

The site has been mapped as part of the Geologic Map of the Yabucoa and Punta Tuna Quadrangles, prepared for the U.S. Geological Survey by C.L. Rogers, M.H. Pease, Jr. and M.S. Tischler (1979) [2]. According to this map, the project site is underlain by the Upper Cretaceous-aged Granodiorite of San Lorenzo (Kl).

Detailed description of this geological formation is as follows:

"*The Granodiorite of San Lorenzo (Kl) consists of Medium-dark-gray predominantly medium grained with hypiodomorphic-granular texture. Largely unfoliated but locally has simple planar arrangement of hornblende and biotite grains.*"

One (1) geotechnical test boring was drilled as part of this investigation on May 2, 2018. Boring was drilled to a maximum depth of thirty-three (33) feet below existing ground elevation. Boring had to be stopped when augers could not penetrate through relative hard weathered rock encountered.

Table 1 below shows a generalized soils stratigraphy.

[1] Sabre Industries designed and fabricated the self-supported tower.

Table 1
Summary of Subsoil Conditions

Depth	Nspt Values	Soil/Rock Type
0-4 ft.	22-23 bpf	Medium Dense Sand
4-33 ft.	SPT Sampling Refusal	Highly to Moderately Weathered Granodiorite. Very Dense Sand or Soft Rock.
33+ ft.	Auger Refusal	Moderately to Slightly Weathered Granodiorite. Soft to Medium Hard Rock.

Basically, subsoil conditions detected consists of a two-layered soils stratigraphy. These are; residual soil, underlain by weathered granodiorite rock, which are described next.

Residual Soil – This material was described as silty sand. Thickness of this layer is about four (4) feet. The Standard Penetration Test (SPT) showed “N” values ranging from 22 to 23 blows per foot (bpf).

Weathered Granodiorite Rock - This layer was described as sand with silt or silty sand with occasional gravels. The SPT showed “N” values of refusal, more than 100 bpf.

Very large and numerous boulders, as large as ten (10) feet in diameter or even more, are very common in this geologic scenario. These boulders are extremely hard to drill or excavate.

The groundwater table (GWT) at the site was not encountered during the subsoil exploration.

MICROPILE RECOMMENDATIONS

Even though the micropiles have been a foundation alternative for many types of projects for many decades, The Federal Highway Administration (FHWA) developed a reference manual in the early nineties. The FHWA NHI-05-039 Micropile Design and Construction Reference Manual [3] is the most widely used manual for the design and construction of micropiles.

A micropile is a small-diameter (typically less than 300 mm), drilled and grouted replacement pile that is typically reinforced. A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can withstand axial and/or lateral loads, and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed. Micropiles are installed by methods that cause minimal disturbance and/or vibration to adjacent structures, soil, and the environment. They can be installed in access-restrictive environments and in all soil types and ground conditions. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects.

Most of the applied load on conventional cast-in-place replacement piles is structurally resisted by the reinforced concrete; increased structural capacity is achieved by increased cross-sectional and surface areas. Micropile structural capacities, by comparison, rely on high-capacity steel elements to resist most or all of the applied load. These steel elements have been reported to occupy as much as one-half of the hole volume [3]. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout/ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter, any end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e., pressure grouting or gravity feed.

Micropile Axial Capacity

The geotechnical bond length tension and compression allowable axial load may be computed as follows:

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

Where:

α_{bond}: grout to ground nominal bond strength

FS: factor of safety

D_b: diameter of the drilled hole

L_b: bonding zone length

The geotechnical Investigation for this project [4] recommended a factor of safety (FS) of 2.5 non-seismic load groups.

Values for the grout-to-ground nominal bond strength ($\alpha_{\text{bond nominal strength}}$) are commonly based on the experience of the local Contractors or Geotechnical Engineers. The grout to ground nominal strength, as presented in the Geotechnical Investigation Report [4], are presented in Table 2.

Table 2
Nominal Grout to Ground Bond Values

Depth	(αbond nominal strength)
Bottom of Pile Cap (8 ft.) to 20 ft.	25 psi
20-30 ft.	40 psi
30+ ft.	75 psi

Many times geotechnical engineers are very conservative when providing these soil parameters. It could be due to the fact of lack of studies and comparative data in similar geologic scenarios. These values of grout to ground bond values may considered high, especially for depths below thirty (30) feet. These values correspond to the lower range of sandstone, as suggested by the FHWA NHI-05-039 [3].

The grout shall have a minimum compressive strength of 5,000 psi.

Group reduction effects do not need to be considered if pile spacing center to center is more than 3.75 times the micropiles diameter [3].

Micropile capacities may be approximated to 20 to 40 percent from the actual pile capacity estimated for a given embedment. Because of uncertainties in estimates of grout-to-ground nominal bond strength, load tests are usually recommended to verify the pile capacity and length estimate. Considering the relative small amount of micropiles to be constructed (18 total), we recommended one (1) proof test be performed on one production, "non sacrificial" micropile. The micropile shall be loaded to at least the design load (ultimate load). Tests must be performed in accordance to ASTM D3689 *"Standard Test Method for Piles Under Static Axial Tensile Load" [5]*.

Micropile Lateral Capacity

The behavior of a laterally loaded micropile depends of many factors such as; the micropile diameter, depth, bending stiffness, fixity condition of the pile in the footing, and on the properties of the surrounding soils. The effects to the surrounding soil from pile installation should also be considered. These effects can include loosening of the soil due to pile drilling and densification of the soil due to grout placement.

Methods available to increase the lateral capacity provided by micropiles include:

- Installing the pile at an inclined angle or batter.
- Installation of an oversized upper casing which increases the effective diameter of the pile, the lateral support provided by the soil, and the bending strength of the pile.

Micropiles lateral capacity is not of concern since the relative small lateral load of the self-supported tower may be supported by friction at the bottom of the foundation and passive forces on the sides of the foundation.

MICROPILES DESIGN

Like any other pile, micropiles shall be designed for two different aspects, these are the geotechnical design and the structural design.

Geotechnical Capacity

The relatively small lateral loads of each tower leg can be supported by friction at bottom of foundation and passive forces on the sides of the foundation. In addition, the relatively large bearing capacity of the weathered Granodiorite can support part of the compression reaction load. This allows for a very simplistic design. The micropiles are design to carry the governing tension load only.

For this project, the total tension load per leg was 1,797.0 kips. A total of six (6) micropiles were selected per each tower leg. Hence, each micropile shall resist an axial load of at least 300 kips, factored or ultimate load (Pu).

A load factor (LF) of 1.6 is normally used for wind loads for the design of communication towers.

$$P = Pu/LF \qquad (2)$$

$$P = 300 \text{ kips}/1.6 = 187.5 \text{ kips service load}$$

Using Equation 1 and the nominal bond values (α_{bond}) presented in Table 2, a six (6) inches in diameter (D_b) micropile with total length of fifty (50) feet (L_b), the safety factor against bond failure may be calculated. The nominal bond capacity is 497,698 kips. Hence, a safety factor of 2.65 was obtained.

Structural Capacity Summary

Micropiles Data:

- Micropile Diameter: 6 inch.
- Grout Comp. Strength: 5,000 psi
- Reinforcing Bar Dia.: 2.25 inch.
- Bar Type: Williams 150 ksi all-thread-bar
- Bar Area: 4.08 sq. in.
- Min. Ultimate Strength: 613 kips
- Tension Per Pile: Ultimate 300 kips
- Service Load: 187.5 kips
- LRFD % Capac. (300/613): 49% < 75% o.k.
- ASD Pall (0.55Fy*Abar): 336.6 kips
- ASD Tension Check: o.k
    - 187.5 kips < 336.6 kips

PILE CAP STRUCTURAL DESIGN

The pile cap was designed assuming that it is perfectly rigid. The following figures show a summary of the structural design.

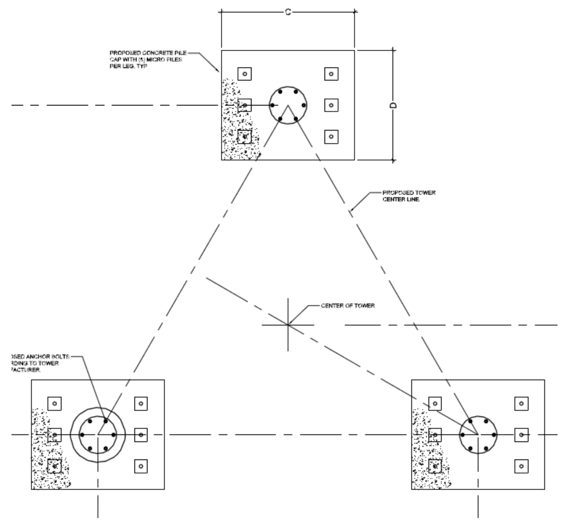

Figure 3
Pile Caps Plan View

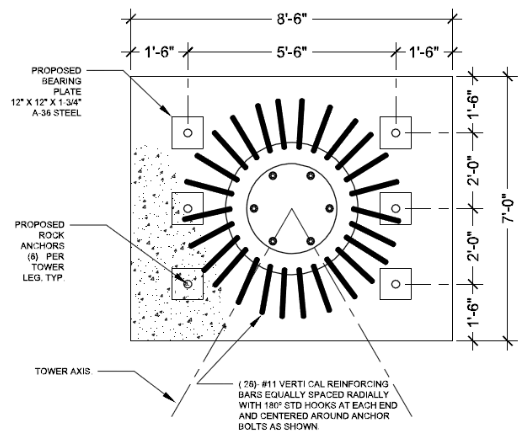

Figure 4
Pile Cap Detail

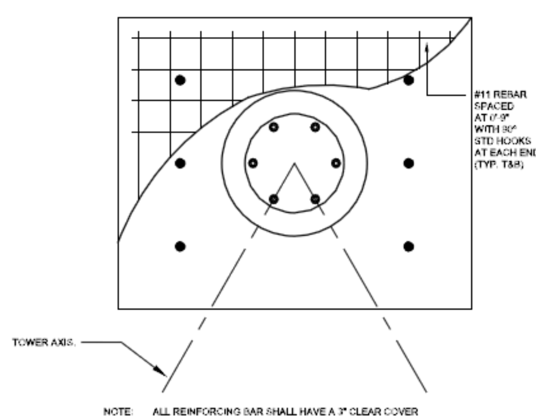

Figure 5
Pile Cap Reinforcement Detail

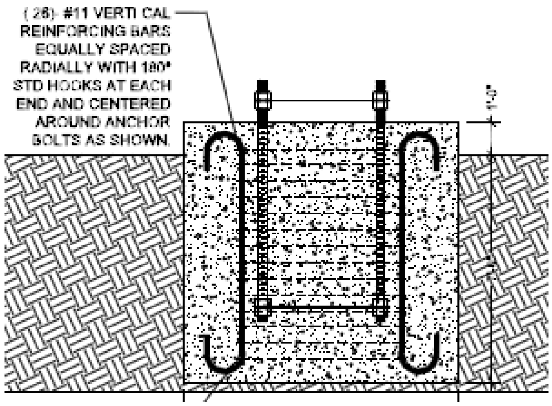

Figure 6
Pile Cap Reinforcement Detail

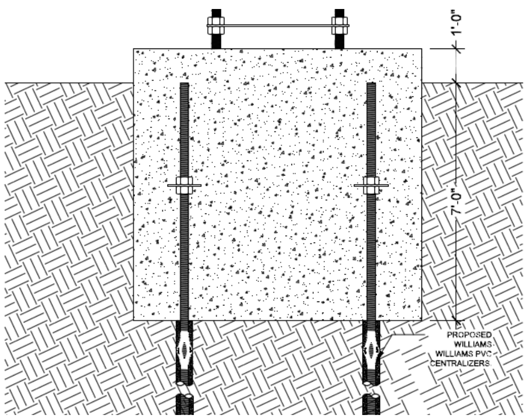

Figure 7
Pile Cap and Micropiles Detail

A summary of the design computations is presented next.

Table 3
Two Way Shear Design Summary

Two Way Shear Computations	
Φ	0.75
Df (ft)	4.0
D (in)	12
b0 (in)	48.0
Ac (sq. in.)	2304
Vu (psi)	130.0
ΦVn (psi)	164
DESIGN OK	

Table 4
Moment Design Summary

Long Direction		Short Direction	
hf (in)	96	hf (in)	96
b (in)	84	b (in)	102
d (in)	90	d (in)	90
f'c (psi)	3000	f'c (psi)	3000
Fy (psi)	60000	Fy (psi)	60000
Dp (in)	10	Dp (in)	10
Mu (K-ft)	899	Mu (K-ft)	150
df	90	df	90
Rho-req	0.0003	Rho-req	0.0000
Areq (sq in)	2.23	Areq (sq in)	0.37
As min (sq in)	14.52	As min (sq in)	17.63
As/ft	2.07	As/ft	2.07
Spacing	9	Spacing	9
Use #11@9"		Use #11@9"	

VALUE ENGINEERING

In order to minimize the project cost, the very difficult access of the project shall be considered greatly. The main concern was how to get concrete to the site. Also, other materials such long steel rebar was a concern as well. Three foundation alternatives were considered. These are; a mat foundation, three drilled shafts (one per tower leg), and micropiles with pile caps.

The design of the mat foundation consisted of a 47'-6" square 3'-3" thick pad with 5'-6" in diameter piers. The minimum depth of foundation was 8'-6". The total volume of concrete was 289 cu. yds. Considering the difficulties for the project, an estimated price to complete this foundation was in the order of $200,000.00.

The drilled shafts design consisted of three (3) shafts with 7'-0" in diameter and 35'-0" of depth. The concrete volume for this alternative was about 150 cu. yds. The site geology contains large and very hard boulders. The possibility to encounter these boulders could not be neglected. These boulders are very difficult to drill which increases the cost of the drilled shaft construction. In addition, the mobilization for a big drilling rig to the remote and difficult to access site was very expensive as well. The cost of drilled shafts construction was in the order $185,000.00.

Most of the difficulties for this project could be minimize with the use of micropiles. The drilling rig for the micropiles is small, the amount of concrete for the pile caps was in the order of 56 cu. yds. In fact, drilling through large and hard boulders is not difficult for the percussion drilling used for the micropiles. These large boulders can actually be beneficial to the micropiles geotechnical bond capacity. The price for the construction of micropiles and pile cap was estimated as $135,000.

Table 5
Project Cost Reduction with Micropiles Alternative

Foundation Type	Foundation Cost	Micropiles Cost Reduction
Mat Foundation	$200,000.00	32.5%
Drilled Shafts	$185,000.00	27.0%

MICROPILES CONSTRUCTION

The most important aspect of this project was the construction of the micropiles. These will most of the controlling axial tension and compression loads of the proposed tower.

The construction of a micropiles in this project was a relatively simple process. The subsoil conditions allow for open hole drilling without the need of hole stabilization. No casing or slurries were required to maintain the hole open.

Percussion rotary drilling was used to drill though the Granodiorite Rock. The rotary-percussion drill is a type of rock drill that uses both rotary and percussive action in order to chip away rock and produce a hole. This is an old technique that has improved with the mechanical developments currently on the market, providing smaller equipment with more drilling capacity.

The combination of rotation and percussion helps the drill achieve a cutting and grinding (rotary) action at the same time as a chipping (percussive) action. Usually these motions are hydraulically or pneumatically driven. Types of rotary-percussion drills include the blasthole drill and the down-the-hole (DTH) hammer drill.

For this project, the DTH hammer drill was used. The DTH hammer drill is basically a mini jackhammer screwed on the bottom of a drill string. The DTH hammer is one of the fastest ways to drill hard rock. The fast hammer action breaks hard rock into small flakes and dust and is blown clear by the air exhaust from the DTH hammer. See Figure 8.

Figure 8
Rotary Percussion Drilling with DTH

Once the hole was drilled with the DTH, the reinforcing bar could be inserted into the hole. See Figure 9.

Figure 9
Micropile Reinforcing Bar Installation

After inserting the reinforcing bar, grout is mixed to design properties such as water to cement ratio (w/c) and fluidity. See Figure 10.

Figure 10
5,000 psi Neat Cement Grout Mixing and Pumping

Figure 11 below shows the cement grout after pump through tremie pipe from bottom elevation of pile until overflow. This practice ensures a proper placement of grout to avoid voids in the micropile.

Figure 11
Finished Micropile

MICROPILES LOAD TESTING

One proof test on a non-sacrificial micropile was tested to 100% of the design ultimate load of 300 kips. The test was performed following Procedure A1 of ASTM D3689 [5].

The load test was supervised by a third party, Eng. Elías Mangual. The results of the load test were presented in Quick Load Test Report, Pandura Site, Yabucoa, PR, E. Mangual (2018) [6].

A free length (Unbonded Lenth) of about ten (10) feet allowed to place reaction beams right beside the pile tested. The free length prevents load transfer to the surrounding soils which otherwise would negatively affect the results of the test. A concrete slab was constructed to support the reaction beams. The slab transfer the loads more evenly and to a larger area while reducing the bearing pressure. Figure 12 shows the set up for the load test performed.

Figure 12
Load Test Set-Up

Figure 13 shows the graphical results for the load test [6].

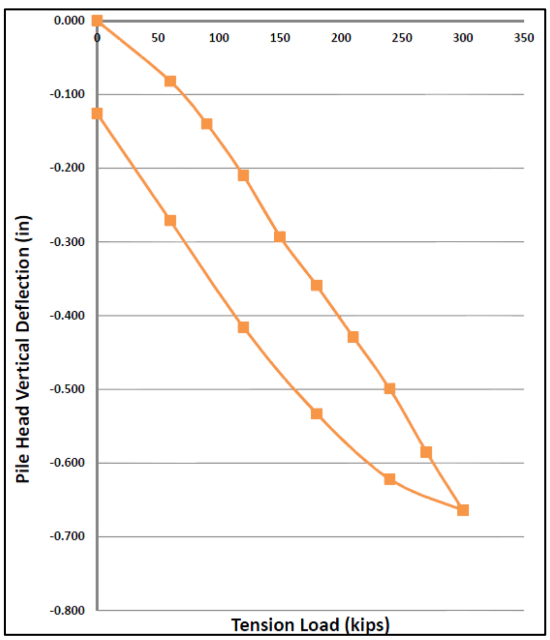

Figure 13
Graphical Results of Load Test Performed to Micropile

In general, the results of the load test showed an almost linear behavior. Failure of piles under tension test would be characterized with excessive deflections and a non-linear relationship of load to deflection, such was not the behavior observed for this test. Hence, no grout to ground bond failure is observed, nor structural failure of the reinforcing bar was observed. The pile has sufficient capacity from both geotechnical and structural point of view. Thus, it was approved to proceed with the pile cap construction without the need of any corrective measures.

PILE CAPS CONSTRUCTION

Once all micropiles construction have been completed and load test was performed, the construction of the pile caps was initiated. The pile will serve to transfer the load from the tower to micropiles. The steel reinforcement was partially pre-assembled to reduce construction time. See Figure 14 that shows the steel reinforcement partially completed.

Figure 14
Picture Showing Steel Reinforcement Partially Completed

Due to the difficulties to mobilize concrete mix trucks, all materials for concrete were transported separately to the site. Even the water was collected over time in a 2,000 gallons tank. Some five (5) hundred feet for pvc pipe was installed from the nearest house to the project. Water was pumped and collected for days previous to concrete casting.

The concrete mixing was done on site with the aid of a mobile concrete volumetric mixer (Mini-Master). This was calibrated on site before pouring. Concrete casting could be completed in two days. Please see Figure 15 for a depiction of the finished pile cap.

Figure 15
Picture Showing Completed Pile Cap

CONCLUSION

The design and construction of a foundation system to replace a telecommunications tower structurally affected by Hurricane María could be completed. Three foundation alternatives were considered for the new tower. Although, micropiles and corresponding pile cap was chosen over the mat foundation and drilled shafts alternative. The use of small equipment and small amount of materials for the construction of micropiles proved to be a great alternative for very difficult access projects.

It was shown that the use of the micropiles alternative resulted in a reduction of project cost. The estimated cost for the mat foundation and drilled shafts alternatives was $200,000.00 and $185, 000.00, respectively. The estimated cost of the micropiles alternative was $135,000.00. This represents a cost reduction of 32.5 percent and 27.0 percent for the mat foundation and drilled shafts, respectively.

The results of load test, performed on a non-sacrificial production micropile, showed that the micropiles can safely support the high magnitude 300 kips design ultimate load for each micropile. In addition, the load test showed that the weathered Granodiorite Rock, present at the project site and in a vast area in eastern part of Puerto Rico, is capable of developing relatively high values of grout to ground bond capacity. This last is of utmost importance, provided that general tendency of designers is to use lower values knowing a safe design will result. This 'tendency' results in cost increases that are transferred to owner.

Further investigation is expected to better define ranges of values for the ground to grout bonding in this kind of materials. To start performing this kind investigation, sacrificial piles would be needed. This would allow to solve for maximum values of the grout to soil bond constant for these kinds of materials.

REFERENCES

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