DISPLACEMENT MICROPILE FOR EFFICIENT INSTALLATION OF TIEDOWN MICROPILES

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ABSTRACT

A 16 storey office tower with 2 levels of underground parking north of Toronto Canada was constructed in an area below the water table. The building basement had a footprint substantially larger than the tower above. Hence, tiedowns were required for hydrostatic uplift. Tension micropiles bonded to the sand formation were chosen to resist the uplift forces and a pre-production load testing program for conventionally drilled micropiles was carried out in advance of tender. The successful contractor proposed a displacement method of installation for the 363 piles. A subsequent load test program was carried out to re-establish design lengths. The installation methodology allowed for fast installation such that the contractor could proof test every pile and replace any deficient ones. A comparison of the methodologies and load testing results will be presented.

1. INTRODUCTION

A 16 storey office tower with 2 levels of underground parking was proposed in Vaughan Ontario – Immediately north of Toronto. The site is 73m x 135m with a 7.75m deep basement throughout. The tower occupies roughly 25% of the footprint.

Underground construction in the Greater Toronto Area is extending further north from the downtown core and Lake Ontario and as you head north the depth to bedrock becomes deeper. The municipal governments have understandably not allowed developments to dewater permanently, forcing them to either build above the water table or build water proof basements. The evolving market is driving the use of micropiles to resist hydrostatic uplift.

Due to the high water level the areas of underground parking without enough dead load from the above structure required tie down piles to resist the uplift. Micropiles were selected as the uplift members.

2. SOIL CONDITIONS

The site is relatively level with an existing grade of 203m above sea level and is covered with a layer of fill that ranges from 0.5 to 4m thick.

Below the fill is a clayey silt to silty clay till (Till No. 1) that is 4 to 7m thick with standard penetration test (SPT) values from 17 to greater than 100, generally falling between 35

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and 55. Interlayers of silt and sand were described in the boreholes, cobbles and boulders are expected to be encountered.

Starting at approximately 195m ASL there is a granular deposit consisting primarily of sand with varying levels of gravel and silt. The layer thickness reduces across the site from 15m to 3m. The SPT values range from 19 to over 100, generally falling between 30 and 40.

A second clayey silt to silty clay till layer (Till No. 2) underlies the granular deposit with a thickness increasing across the site from 8m to 23m. The SPT values range from 80 to over 100.

Below the second till is a glaciolacustrine clayey silt to silty clay extending to an unknown depth. Bedrock was not encountered in the geotechnical investigation but geologic maps from the area indicate that it is 50 to 100m below ground level and is part of the Georgian bay formation which is a shale with limestone interbeds.

Table 1 below summarizes the soil properties described in the geotechnical investigation.

Groundwater levels were measured at 199.5m or approximately 3m below existing ground surface.

Table 1. Soil Properties from Geotechnical Investigation

Soil Layer	Total Unit Weight kN/m³	Effective Angle of Friction °	Undrained Shear Strength kPa
Fill	19	26	n/a
Till No. 1	22	35	160
Granular	22	36	n/a
Till No. 2	21	35	250

3. PRE-PRODUCTION TEST PROGRAM

To assist the design of the foundation system of the permanent structure a pre-production test program was conducted to assess the bond capacity of both the granular layer and the second till layer.

Seven pile locations were selected by the project geotechnical engineer based on the varying levels of the granular deposit and the Till No. 2. Four locations were to be bonded in the granular deposit (G1 to G4) and three locations in the till (T1 to T3). The test piles were to be installed from existing surface elevation.

A contractor from a nearby infrastructure project was retained by the owner to install the test piles. They were installed using a double head rotary duplex drilling system using air and water as the flushing medium.

The piles were grouted using the tremie method and a single post grout tube with three valves was affixed to the bar (i.e. Type C single global post grout). Once the tremie grouting was completed, the tremie tube was raised to the top of the bond zone as the remaining grout was displaced with a bentonite mix to create the unbonded length.

The pile sections consisted of a 204mm diameter drill hole filled with a 35 MPa neat cement grout and a high grade 66mm diameter bar with a yield load of 2,754 kN and ultimate load of 3,442 kN. The arrangement of both unbonded and bond length are shown in figure 1 below.

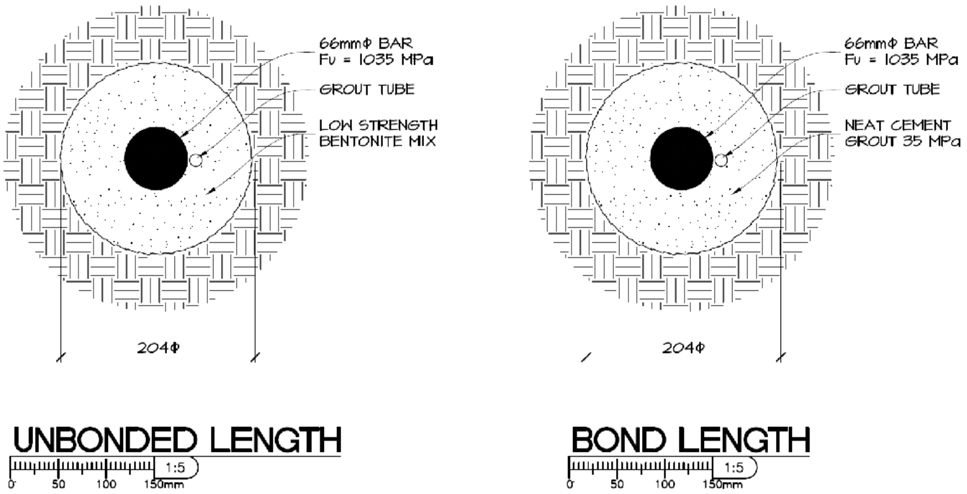

Figure 1. Test Micropile Sections.

Bond lengths from 4.5m to 7.5m were evaluated for the Granular deposit and 4.5 to 6m for Till No. 2. Details of the installation are given in Table 2 below.

Table 2. Pre-Production Test Installation Parameters

Pile ID	Unbonded Length (m)	Bond Length (m)	Post-Grout Volume (L)	Post-Grout Pressure (MPa)
G1	8	4.5	0	No Break
G2	8	6	144	4.1
G3	10	6	90	3.4
G4	10	7.5	72	4.1
T1	17.5	4.5	36	4.1
T2	18	6	72	4.1
T3	17	6	108	4.1

Test results for the pre-production tests in the granular deposit are shown in Figure 2, test G2 was omitted for clarity as it resembled G3 very closely. The maximum achieved adhesion was 489 kN/m.

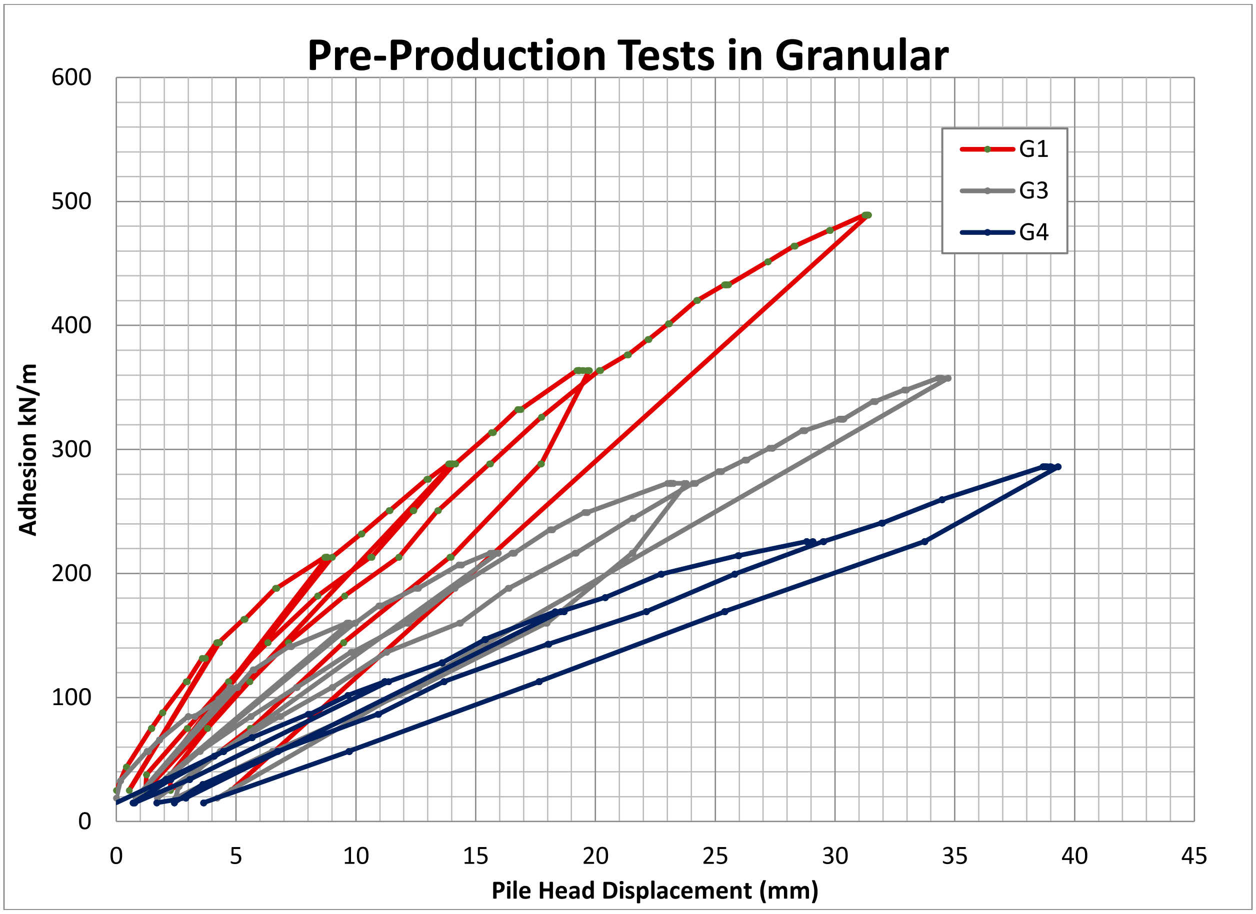

Figure 2. Pre-Production Test Results in Granular

Table 3 Pre-Production Test Results

Pile ID	Max Test Load (kN)	Max Adhesion (kN/m)	Apparent Free Length (m)
G1	2201	489	8.2
G2	2201	367	8.8
G3	2145	358	9.7
G4	2145	286	11.4
T1	2201	489	11.2
T2	1976	325	10.3
T3	1947	325	8.5

Test results for the pre-production tests in the Till are shown in Figure 3. The maximum achieved adhesion was 489 kN/m.

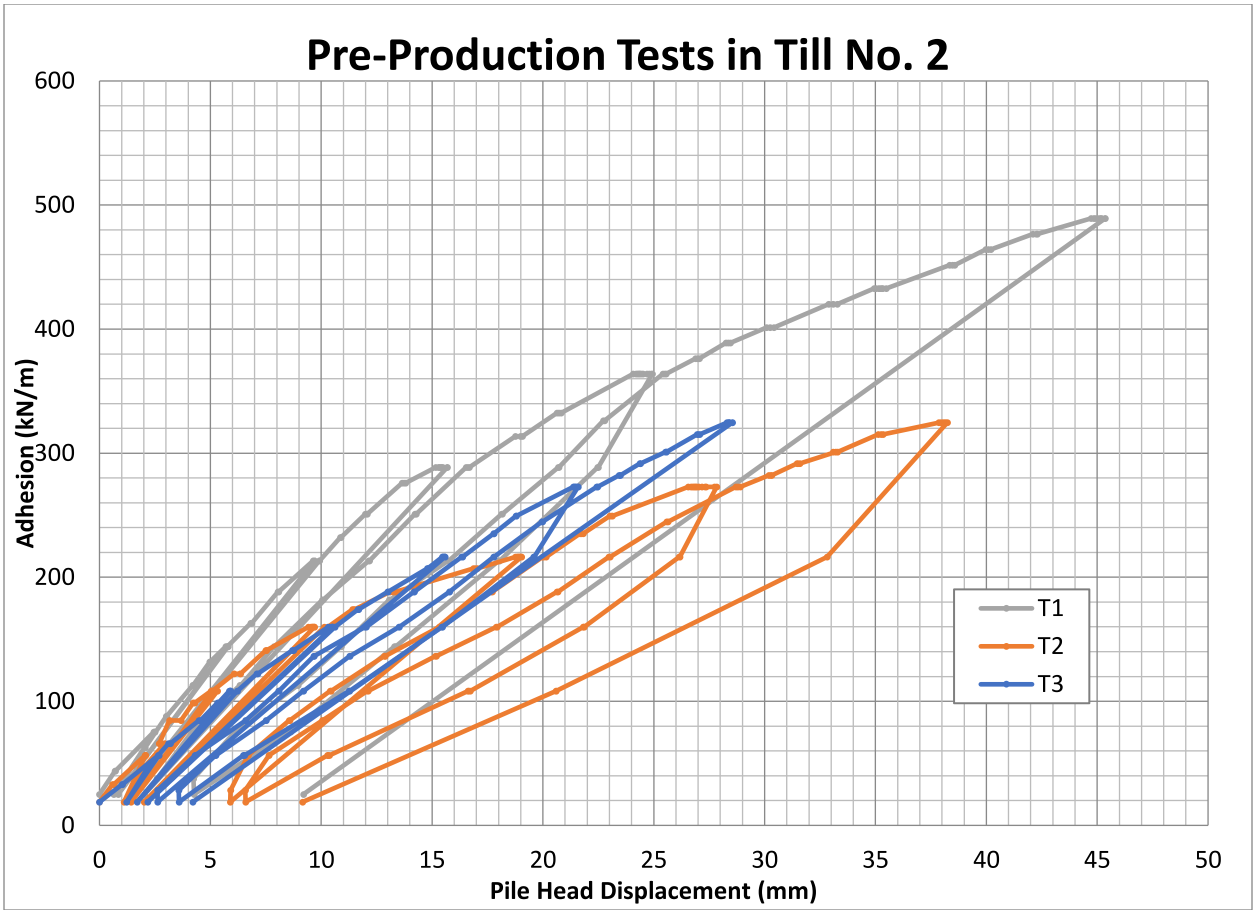

Figure 3. Pre-Production Test Results in Till No. 2

The results of the tests revealed a couple of important considerations to account for in the final design. First, the unbonded lengths were not fully achieved for the longer piles bonded to the till, the use of bentonite flushing was not recommended for the use of permanent anchors. The second was that the short 4.5m bond lengths were more efficient than the longer 6m and 7.5m bond lengths. These results led to the use of many piles with short bond and free lengths in the final design.

Project geotechnical engineer conducted an analysis on the tests to provide bidders with ultimate and service bond stresses. These were corrected for a change in overburden pressure.

4. CONTRACTOR ALTERNATIVE

Following the pre-production testing the design of the foundation was completed and the project was tendered. The general contractor selected a different contractor than the pre-production testing contractor.

The successful contractor had proposed an alternative installation methodology that involved driving a closed end pipe pile with a sacrificial pile toe cap. The bar would then be lowered into the pipe and then filled with grout. The pipe was then retracted before the grout sets up. A single post grout tube was also used in this methodology. The production micropile sections are shown in figures 4 and 5 and are very similar to the pre-production test piles.

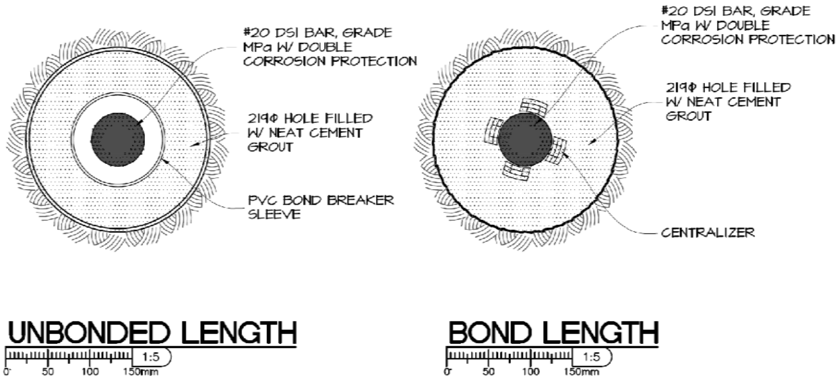

Figure 4. Production Micropile Sections

A verification test was performed on the first production pile to assess the capacity of the final micropile design and installation method. The test was conducted in the granular deposit from the final excavation level. The results of the test are shown in Figure 6 and are overlaid onto the pre-production tests in the granular deposit. The maximum achieved adhesion was 175kN/m. The results show that the unbonded length was achieved and that the pile was generally less stiff than the pre-production piles. The difference in behaviour may be down to a combination of factors that could include the cross sectional properties, the installation method not creating a rough borehole to enhance the bond and the reduction in overburden pressure.

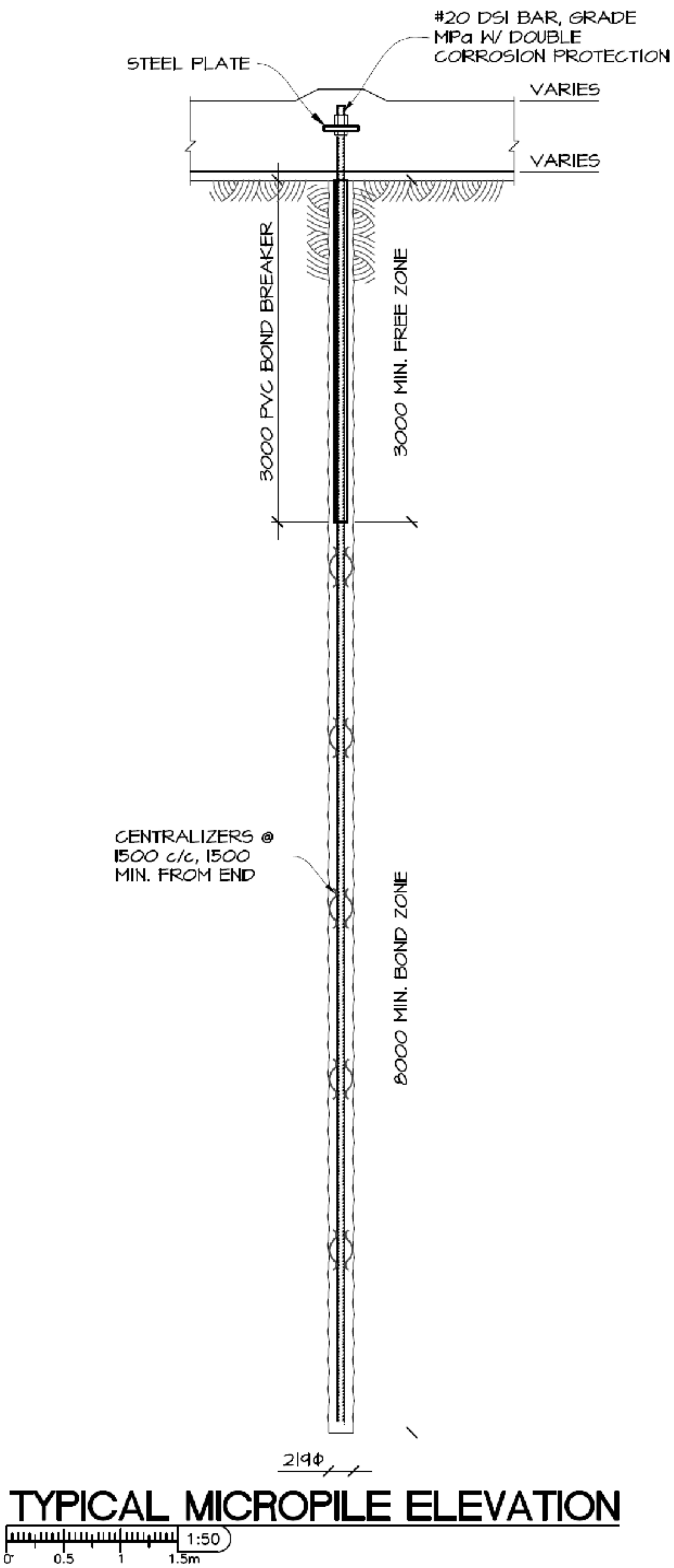

Figure 5. Production Micropile elevation

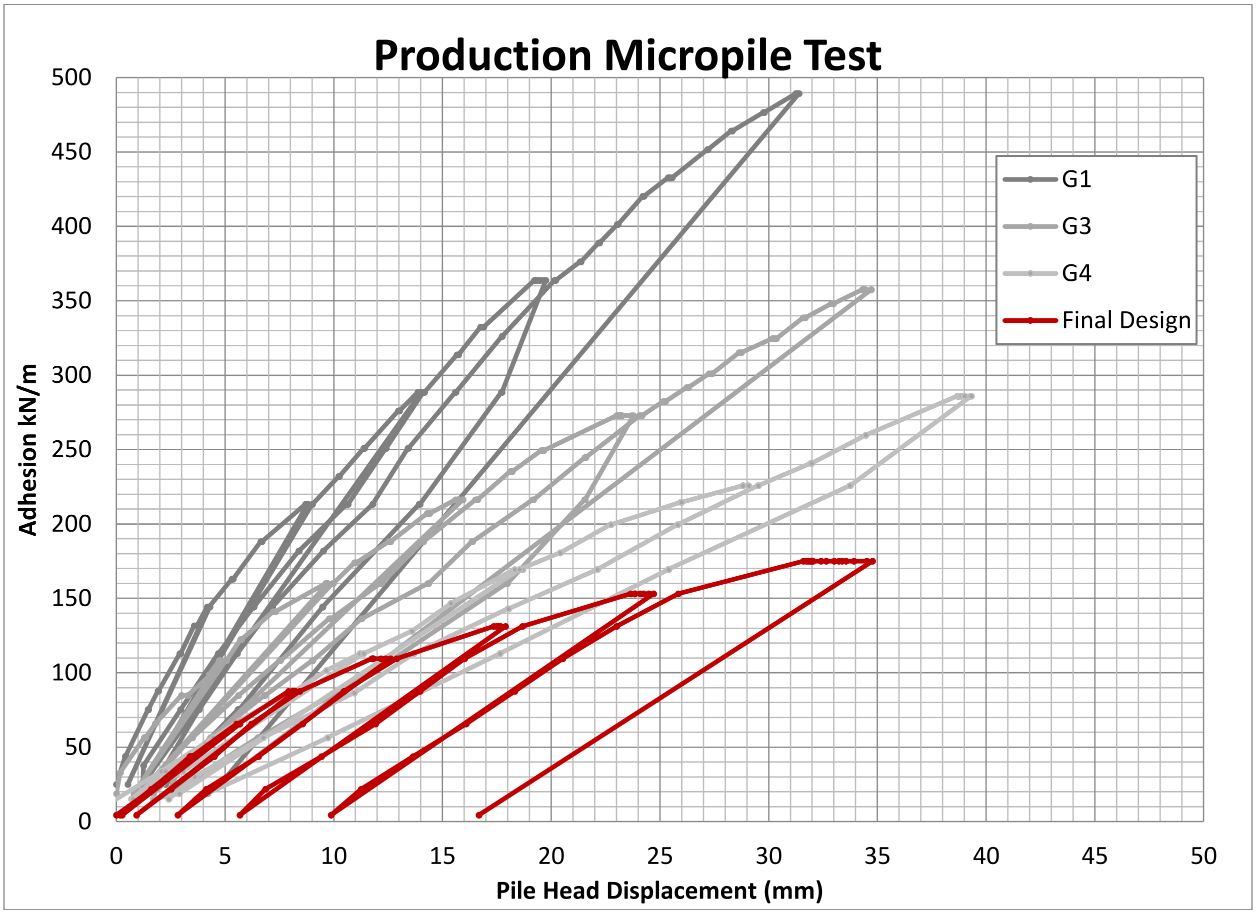

Figure 6. Production Micropile Test

5. DISCUSSION

The change in methodology between the conventional drilled replacement micropiles and the driven displacement micropile allowed for some advantages during construction. The speed of installation was a major advantage with 363 micropiles by significantly reducing the schedule. Due to displacement method there were no spoils to remove, keeping the site clean.

The contractor used a lead mounted diesel hammer suspended from a crane to drive the pipes since there were no overhead restrictions. This system allowed the contractor to reach over previously installed piles and other obstacles. This was particularly useful for adding additional piles where some piles did not perform during proof testing.

6. CONCLUSION

This project illustrated the importance of pre-production testing and how it can be used to influence the final design. The project also shows how a successful pre-production program may not be reflected in the final design but provides a baseline for the tender while leaving room for a contractor proposed alternative.

The innovative installation method used for the installation of tension micropiles to resist uplift shows that the conventional reasoning behind micropiles may not always apply and that new techniques used in specific situations may result in an overall better product.

7. REFERENCES

Golder Associates Geotechnical Investigation Report Project No. 11-1111-0061(3110)

Reference Manual. Publication No. FHWA-NHI-05-039, 2005. US Department of Transportation, Federal Highways Administration, 2005, “Micropile Design and Construction”