INTRODUCTION

The interior of four structures that were annexed to form the original headquarters of the Fidelity Investment Company at 82 Devonshire Street in Boston, MA will be selectively demolished to allow for the internal addition of a 14 story structure. The foundations of the new office building will be supported on high capacity drilled micropiles. There will be 51 micropiles to support the new super columns, 67 micropiles to support the core of the new structure plus 19 micropiles to support the tower crane foundation. In addition, 8 micropiles were required to underpin two columns immediately next to a support of excavation (SOE) system that will allow access to the core mat foundation. The critical component of the SOE is 15 micropiles that will act as soldier piles.

Two pile load tests were performed, one in compression to a maximum test load of 1010 tons (8.9 MN) and one in tension to a maximum test load of 458 tons (4.1 MN). The test results were excellent. Based on these results, the geotechnical engineers agreed to reduce the rock socket length by 50% from the original design.

Hub was contracted jointly to the Consigli Construction Company (project construction manager) and RelatedBeal (the project developer). The structural engineer was MacNamara Salvia. Haley & Aldrich was the geotechnical engineer.

FOCUS OF THIS PAPER

This paper will focus solely on the core piles and the results of the pile load tests. Also, the results of these tests along with others that hub has performed over the past five years has led us to recommend that the industry revisit the design of high capacity rock socketed micropiles.

The main points of emphasis for future discussions will be the following:

- reevaluate strain compatibility within a confined section such that higher allowable stresses in high strength steel can be utilized;

- utilize a higher allowable stress in grout;

- evaluate the load distribution at and below the casing tip to develop an alternative approach to determine the rock socket length that is now solely based on friction and 100% of the design load; and
- evaluate the steel design within the rock socket using the load distribution data from these tests.

**SITE CONDITIONS AND SCHEDULE**

Two pile load tests were performed from March 28 to April 6, 2016 prior to the completion of the overall structural design. The tests were performed in a very tight open area adjacent to the project structures.

The core piles were installed from January 6 to February 11, 2017. The piles were installed from a level about 25 ft. (7.62 m) below street level. The piles were installed with two different drill rigs. Eight of the piles were installed with the Davey-Kent DK-525 drill rig at an area with only 10 ft. (3.1 m) of headroom. This drill rig was lowered to the drill locations via a hoist hatch next to an alley. Photo 1 shows the DK (without the mast) being lowered. After significant demolition of the existing structures was completed up to the roof level, a Casagrande C-8 drill rig was utilized for the remaining 59 piles. This drill rig entered the building from one of the main streets that surround the buildings. The rig walked inside a portion of the building and down a ramp (cut at the SOE) to access the drill level for the remaining piles. Photo 2 shows the C-8 crawling thru the opening and down the ramp down to the work level.

Photo 1: The DK-525 low headroom drill rig hoisted into the building though a small hole in the street.
The majority of the micropiles were installed while heavy demolition of the existing structures was in full swing. The work areas resembled a "war zone" during this process. Photo 3 shows the “scars” of the demolition process as the core area is redeveloped up to roof level.

Photo 2: The Casagrande C-8 entering the building off Quaker Lane. A hole was created in the outer wall just big enough for the C-8 to enter. The length of the mast had to be factored into the required clearance when walking down the ramp to 25 feet (7.3 meters) below street level.

Photo 3: Drilling support of excavation piles in the demolished center of the structure which will be the future location of the core.
ORIGINAl DESIGN

The original design loads for the micropiles was 325 tons (2.9 MN) in compression, 150 tons (1.3 MN) in tension and 11 tons (97.9 KN) lateral. The micropile design consists of the following:

- a 10 ¾ in. (273 mm) OD; 0.545 in. (13.8 mm) wall casing drilled down into stable rock;
- a core pipe [ 8.625 in. (219 mm) OD; 0.719 in.(19.8 mm) wall] to reinforce the rock socket plus 15 ft. (4.6 m) above the casing tip;
- a core pipe [ 7.0 in. (179 mm) OD; 0.50 in. (12.7 mm) wall] in the upper 25 ft. (7.6 m) of the micropile to sustain the bending moment due to the lateral load at the top of the pile;
- a 2.5 in. (63.5 mm ) Grade 150 bar to support the tension load; and
- 5000 psi (34.4 MPa) grout.

The length of the rock socket was specified by the Engineers to be 34 ft. (10.2 m) long. Refer to Figure 1 which shows the above details. Refer to the section of the coring log that describes the coring of the rock. In Figure 2

PILE LOAD TESTS

Two pile load tests were performed - one in compression to 1010 tons (8.9 MN) and one in tension to 458 tons (4.1 MN). The pile load tests were performed about three months before a wind tunnel study was performed which reduced all the design load components. The final micropile design is provided in a separate section below.

Compression Test

Figure 3 shows a cross-section of the test pile arrangement which consisted of four (4) reaction piles and the test pile in the middle. Photos 4 thru 9 show drilling of the test pile and the reaction piles, setting up the test beams and the final test pile setup. Note that the test pile was drilled with a low headroom drill rig (DK-525) to provide drill rates for the casing to simulate the low headroom conditions anticipated for the production piles inside the buildings. The reaction piles were drilled with a Casagrande C-8 drill rig.
Figure 1: Original core pile design which was the basis for the pile load tests.
Figure 2: Coring log taken describing the coring of the rock
Figure 3: Section of the compression test layout

Photo 4: DK-525 drilling the compression test pile
Photo 5: Casagrande C-8 drilling reaction piles.

Photo 6: Setting up the test frame
Photo 7: Test frame setup complete

Photo 8: Recording pile movement during the pile load test
Photo 9: Test frame covered during the test to protect the instrumentation from the elements during the test. A data logger and laptops were utilized to monitor and record the instrumentation installed within the test pile.

Figure 4 shows the plot of pile settlement versus load for the test pile. The “curve” is essentially linear for the entire loading schedule. The total settlement of the pile was 1.471 in. (34.36 mm) at 650 tons (5.8 MN) and 2.656 in. (67.46 mm) at 1010 tons (8.9 MN). The net settlement after unloading from 1010 tons (8.9 MN) was only 0.372” (9.5 mm). The total creep movement at 1010 tons (8.9 MN) from 1 minute to 90 minutes was only 0.032 in. (0.813 mm).

Figure 4: Compression test settlement versus load graph
Figure 5 shows the distribution of the test load throughout the full depth of the test pile. A total of 13 strain gauges were attached to the 2.5 in. (63.50 mm) Grade 150 bar. Geokon 4911 “sister bars” were utilized. Nine of the strain gauges were placed within the 25 ft. (7.6 m) rock socket.

![Figure 5: Compression test load distribution graph](image)

The large number of strain gauges within the socket were utilized to confirm that a significant amount of the test load would be “shed” at or just below the casing tip. Note that 145 tons (4.8 MN) at the casing tip (54% of the test load) and only 303 tons (2.7 MN) at a depth of 1.0 ft. (0.3 m) below the casing. This is only 30% of the applied test load. Hence, 263 tons (2.3 MN) was “shed” in one foot below the casing tip and 562 ton from one foot above the casing to one foot below the casing. At 10 ft. (3.04 m) below the casing tip, 234 tons (2.1 MN) remained, or 23% of the max. test load.

**Tension Test**

One of the four reaction piles for the compression test was utilized to perform a pile load test in tension. Figure 6 shows the test pile consisting of:

- a 3 in. (76.2 mm) Grade 150 bar;
- a rock socket length of 25 ft. (7.6 m);
- location of three strain gauges and two tell-tales within the rock socket.

Note that the bar was debonded with a PVC pipe close to the top of the rock socket. The depth to stable rock was 79 ft. (24 m).
Figure 7 presents the plot of test load vs elongation. The “curve” is essentially linear up to about 400 tons (3.6 MN) which is close to the minimum yield strength of the bar (~420 tons (3.7 MN)). The bar begins to show plastic deformation beyond this point which accelerates between 450 tons (4.01 MN) and 458 tons (4.08 MN). The net “settlement” after unloading is 1.006 in. (25.55 mm) which is very close to the actual total movement at 458 tons (4.07 MN) less the theoretical elongation of the bar, if it were fully elastic at that load. The recorded elongation during the test was strictly due to the elongation of the bar, as evidenced by the recorded small movement only 0.026 in. (0.661 mm) at the tip of the pile.

Figure 8 presents the test load distribution thru the test pile. Note that all the drill casing had been extracted after the bar was installed and grouted. Also, the PVC sheathing that was utilized to debond the bar was installed to a depth of about 5 ft. (1.5 m) above the top of rock (we wanted to be 100% certain that the sheathing did not extend into the rock socket). The combination of both items explains why some of the test load was transferred above the top of rock. This resulted in 344 tons (3.1 MN) or 75% of the max. test load being transferred to the top of rock which is still significant. Approximately 115 tons (1 MN) were transferred one foot into the rock socket. Only 10 tons (89 KN) was recorded at a depth of 1.5 M (5 ft.) into the rock socket. Hence, a total of 334 tons (3.1 MN) was transferred in the upper 5 ft. (1.5 m) of the rock socket. This is
73% of the maximum test load. Note that the creep at the maximum test load was only 0.022 in. (0.558 mm) from 1 to 10 minutes.

Figure 7: Tension test elongation versus load graph

Figure 8: Tension test load distribution graph
**Evaluation of Test Results**

The significance of the test results is as follows:

1) The plot of settlement v test load and the creep results at maximum test load for the compression test clearly indicate that this test pile could have been subjected to a significantly higher load. If the reaction piles and the test beams had a higher overall capacity, this pile could have been tested to 2000 tons (17 MN) without any significant increase in total movement and creep.

2) The compression test pile and many others that Hub Foundation has performed to high loads show that a significant portion of the test load is transferred to the tip of the casing and is supported in compression by the combination of the rock and grout column directly below the casing. In addition, the majority of the balance of the test load is transferred within the upper 5 ft. (1.5 m) of the rock socket.

3) The plot of elongation v test load of the tension test was essentially elastic within 267% of the design load or 400 tons (3.6 MN). The recorded elongation was solely that of the bar with only a minor movement within the rock socket.

The above evaluation and test results plus other similar tests can be utilized to develop the basis for changes to the overall design of high-capacity rock-socketed micropiles.

**FINAL DESIGN**

After the pile load test was completed, the project performed wind tunnel modeling of the structure to attempt to reduce the lateral loadings on the core of the new structure. The results of these model studies were extremely beneficial, thereby, reducing the loadings on the core piles, such that, a smaller diameter pile could be utilized.

The final design capacity of the core piles was 280 tons (2.5 MN) in compression, 120 tons (1.1 MN) in tension and 8.1 tons (72 KN) lateral. Internal core pipes were utilized to provide the structural requirements within the rock sockets and at the upper portion of the piles to take care of the bending moments from the lateral loads. Refer to Figure 9 which provides a sketch of the micropile design.

The micropiles for the core consist of the following:

- 9.625 in. (244 mm) OD; 0.545 in. (13.8 mm) wall casing drilled down into stable rock;

- a core pipe [ 6.302 in. (160 mm) OD; 0.85 in. (21.6 mm) wall] to reinforce the rock socket plus 15 ft. (4.6 m) above the casing tip;

- a core pipe [ 6.625 in. (168 mm) OD; 0.475 in. (12.1 mm) wall] in the upper 15 ft. (4.6 m) of the micropile to sustain the bending moment due to the lateral load at the top of the pile.
- a 1.75 in. (44 mm) Grade 150 bar to support the tension load; and
- 5000 psi (34.5 MPa) grout.

The length of the rock socket was evaluated from the pile load tests and was reduced to 17 ft. (5.2 m) long.

![Figure 9: Final micropile design for the core piles](image)

**PILE INSTALLATION**

The micropiles for the core of the structure were installed with two different drill rigs. Due to physical restraints and the omnipresent demolition process, Hub could only drill with one rig at a time.

Photos 10 thru 12 provide a view of the extreme difficulty of installing the low headroom micropiles within confined surroundings. Although about 25 ft. (7.6 m) below street level and in an “hostile” environment, the installation of the micropiles with unlimited headroom proved to be a much smaller challenge. Photos 13 thru 15 provide views of this process.
Photo 10: Casagrande C-8 drilling core piles within the central core while a snow storm hammered Boston outside.

Photo 11: Drilling in tight places with minimal access
Photo 12: Drilling in a narrow corridor

Photo 13: Installing pile reinforcing with a fork truck while piles are drilled elsewhere
Photo 14: Casagrande C-8 drilling core piles

Photo 15: Casagrande C-8 working its way out of the core
CONCLUSION

The structural components that are utilized in the current micropile market consist of relatively low cost but high strength pipe manufactured for the oil drilling industry. The yield stress of the pipe ranges from 80 ksi (551.8 MPa) to 140 ksi (965.3 MPa). This pipe is used for both the permanent drill casing seated into stable rock and, as in this project, for the core pipe to increase the overall capacity thru portions of the pile length. In addition, continuously threaded bars with yield stress of 75 ksi (517 MPa) to 150 ksi (1034 MPa) are included to provide additional compression and tension capacity. The current codes (IBC and/or local codes) only allow a fraction of the yield stress of the steel components, up to 32 ksi (220 MPa). Consideration should be given to form a committee to review this and make recommendations to increase this value to a higher allowable stress for the different steel components.

The above point was the emphasis of an excellent technical paper, “Design with High-Strength Steel Reinforcement in Micropiles - Clearing up the Misconception in Strain Compatibility”, by Terence P. Holman, John R. Wolosick and Thomas D. Richards. This paper was published in the ADSC Foundation Drilling magazine in August/September of 2016. The current strain compatibility models \( F_y = 0.003 \, E_{steel} \) is based upon unconfined sections in axial compression or combined compression and flexure. Studies reported in this paper cite results that indicate that ultimate strain levels can be up to 10 times higher in confined columns. Based upon these results and the recommendations made by Holman, et al, allowable stresses in steel can be safely increased from 0.4\( F_y \) to 0.5 \( F_y \), up to 60 ksi (413 MPa). Note that the industry designs tension elements with allowable stresses of 60% of ultimate stress. This is 90 ksi (621 MPa) for 150 ksi (1,035 MPa) bars and 162 ksi (1,035 MPa) for 270 ksi (1,863 MPa) strands. Tension elements face a much more challenging environment than steel in compression in a confined environment within grout and rock. Also, the allowable stress in the grout should be increased from 0.33\( f'_c \) [up to 1600 psi (11 MPa)] to 0.4\( f'_c \). Note that the grout mixes that are now utilized in micropile installations have 28 day compression strengths of at least 6000 psi (41.3 MPa) and up to 8000 ps (55.1 MPa). The maximum allowable stress for grout in a confined condition should be increased to a minimum of 2000 psi (13.8 MPa). Note that these recommendations will only apply to permanently cased micropiles into dense tills and rock.

The current evaluation of determining the length of rock sockets should be modified to account for the path of the load distribution that has been demonstrated by numerous pile load tests in rock. The concept that load transfer within the bond length of micropiles is solely in friction needs to be revisited. The test data that Hub Foundation has developed over the past 5 years for high capacity micropiles show that a significant percentage (55% to 90%) of the applied test load is supported by the steel casing (the stiffest member of the pile) and transferred to the tip of the casing. This load is supported in compression by the rock and the confined grout column below the casing. The majority of the balance of the test load is, typically, transferred within the upper 5 ft. (1.5 m) of the rock socket. The industry continues to ignore this critical factor and continues to design these high capacity micropiles with the only transfer being friction or bond between the grout and the rock socket. Depending upon the level of conservatism...
that is utilized in determining the bond value of the rock (based upon rock type and quality), this, typically, results in excessive rock socket lengths.

Another possible modification to high capacity rock-socketed micropiles is the structural design within the rock sockets. If only 10% to 45% of the maximum test load is transferred into the socket, why do we design these long rock sockets for 100% of the design load? This is a very critical question, especially for low headroom micropiles where the size of couplers and the acceptable level of grout cover have the greatest impact on the design capacity.

In conclusion, the biggest impediment to the growth of the micropile industry is the design of the rock sockets for high capacity micropiles. It is Hub’s goal to continue with the application of high and higher capacity micropiles to be utilized to solve critical constructability issues with deep foundations. Hub Foundation is planning for 400 tons (3.6 MN) to 500 tons (4.5 MN) micropiles on future projects. We strongly recommend that ISM along with DFI and ADSC review our recommendations and that of Holman, et al, to modify the design of high capacity rock-socketed micropiles.