Foundation DRILLING

NOVEMBER/DECEMBER 2016

Hub Foundation and Great Faith in Micropile Foundations

Drilled Shafts: Small Scale, Large Benefit

IAF Funded Research: Connection Capacity Between Micropiles and Existing Footings

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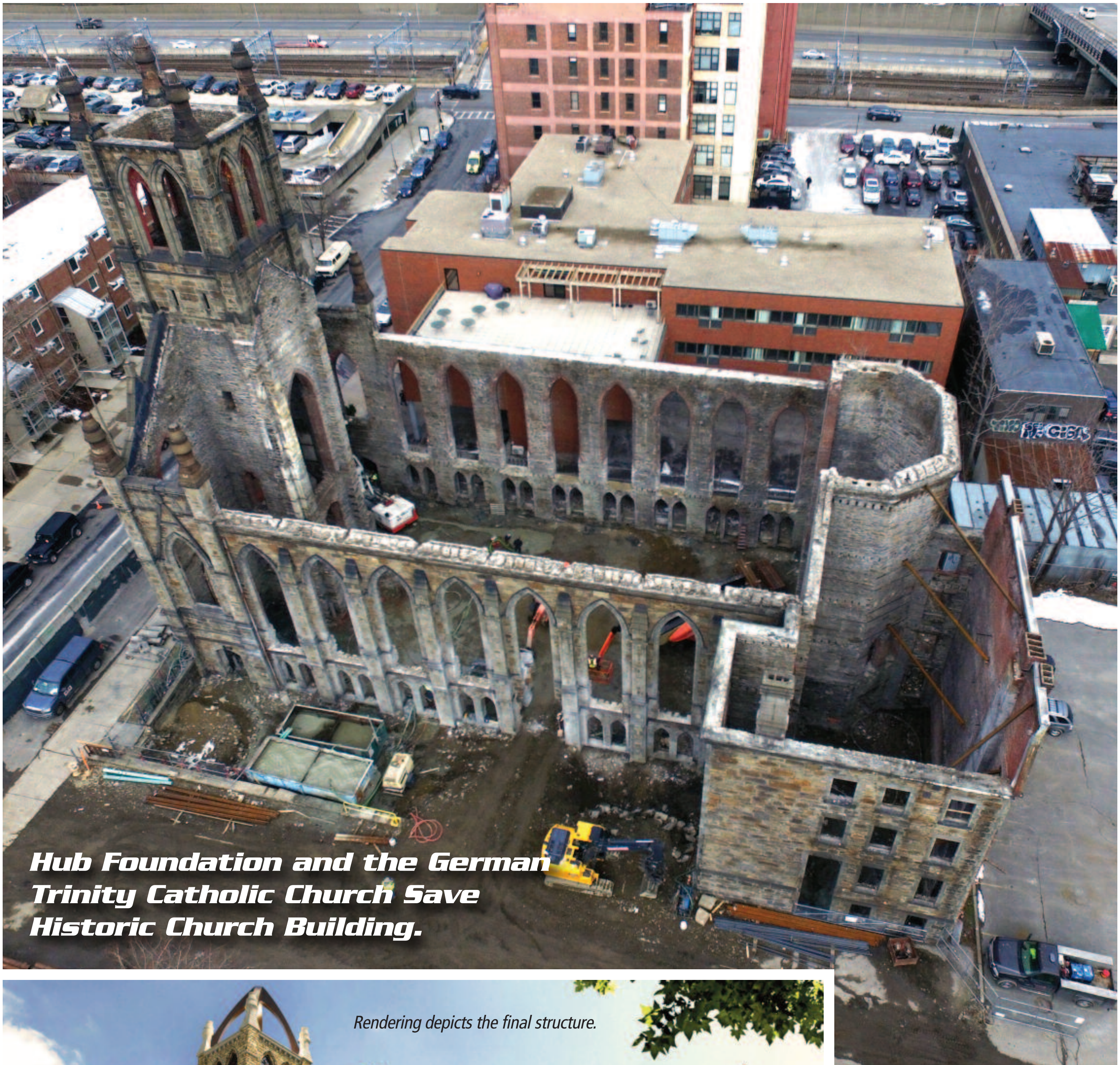
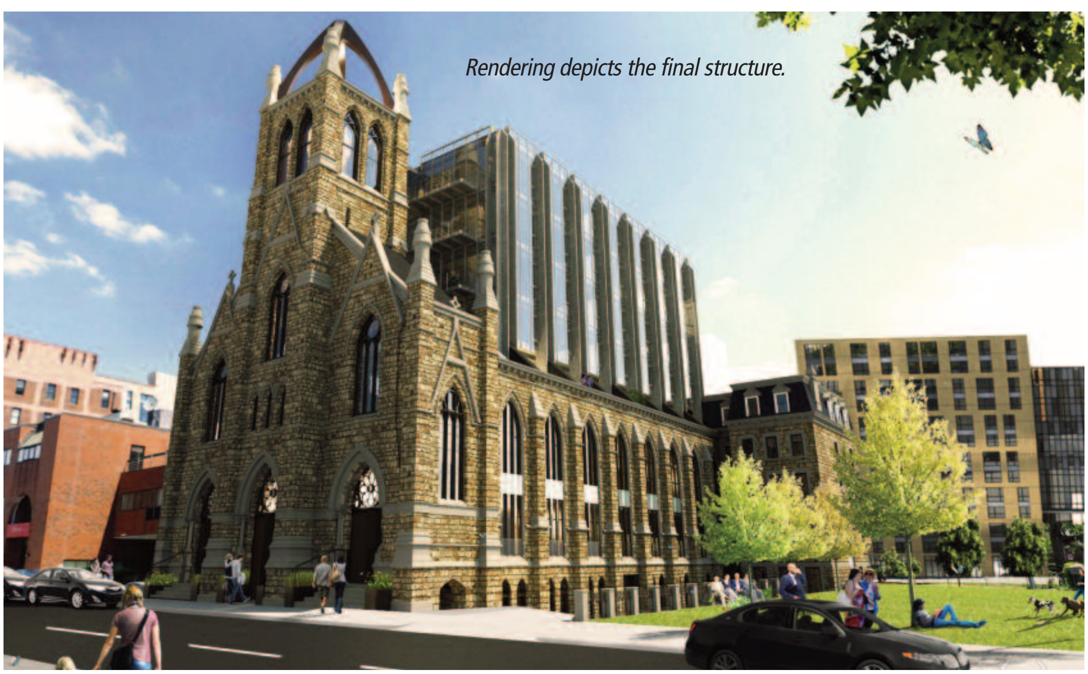
DEPARTMENTS



Hub Foundation and the German Trinity Catholic Church Save Historic Church Building.

Figure 1. Aerial shot of work in progress.



Rendering depicts the final structure.

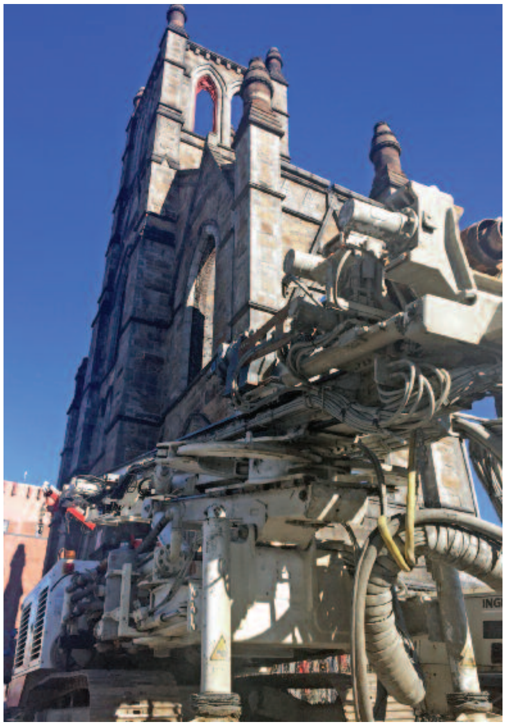
Background

The German Trinity Catholic Church and micropiles have "come together" to extend the life of a very beautiful Roxbury Puddingstone and Maine Granite structure that was originally completed in 1874. The Archdiocese of Boston sold the structure, located at 136 Shawmut Avenue in the South End section of Boston, Massachusetts to the New Boston Ventures Group. The exterior facade of the structure will be preserved while the interior and roof were completely demolished to allow

COVER FEATURE

Great Faith in Micropile Foundations

By Anthony C. Barila, P.E., Acting Project Manager and John R. McKinnon, General Superintendent, Micropile Division Hub Foundation



Casagrande C-8 prior to mobilizing into the church.

for an eight story glass structure to be constructed from within.

Architects Finegold Alexander of Boston, Massachusetts designed the new project named The Lucas. The plan inserts a new eight story building within the existing Roxbury Puddingstone walls and 100 foot tower. The design retains the architectural qualities of the original structure merging it with a striking steel and glass contemporary interior that rises out of the massive stone walls of the former church. The project will be 80,000 square feet consisting of 33 luxury condominiums and 30 underground parking spaces and was approved by the Boston Redevelopment Authority in 2015.

Figures 1 and 2 provide an overhead view of the stone facade structure. Note that except for some of the internal masonry walls within the structure, all internal components of the church were removed. A drone was used for all the overhead photos herein and it was carefully maneuvered to abide by its usage along the flight path to Logan Airport.

Construction

The project began in late 2015 with the extensive demolition phase. The micropile work began in early 2016 and was completed in mid-March. The overall project will be completed in 2017. A total of 54 micropiles have been installed to support the new structure. Sixteen of the piles are located along the perimeter of the new structure and have a design capacity of 75 tons in compression. Thirty-four micropiles with design capacities of 175 tons in compression, 75 tons in tension and 2 tons lateral are located throughout the interior.

Two different pile designs were submitted and approved by the project design team. Larry Johnsen with GZA worked closely with Hub to develop these designs and those for the test pile. All the piles were designed with permanent steel casing drilled into stable rock. We define stable rock being that which will not collapse while the rock socket is being drilled and will remain open until the pile is grouted. The rock sockets were designed with an allowable bond value of 125 psi. The perimeter piles were designed with a 7.0″ OD casing. The interior piles were designed with 9-5/8″ OD casing. Figure 3 shows a design sketch of the perimeter piles. The reinforcement for the perimeter piles was a No. 18 Grade 75 bar and was limited to the rock socket plus a 4.5 feet lap above the tip of the casing. Figure 4 shows the design of the interior piles. The reinforcement of the rock socket for the interior piles was a No. 28 Grade 75 bar extending 8.5 feet above the tip of the casing. Note that the tension load requirements for the interior piles was provided by a No. 18 Grade 75 bar in the upper 16 feet (below pile cutoff) of the piles. The tension requirements did not warrant a full height bar.

The early use of micropiles for foundations was for relatively low design loads (20 to 40 tons) which were installed with small diameter temporary

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Figure 2. Shows the close proximity of adjacent buildings.

Birds eye view of the drilling from atop the parapet.

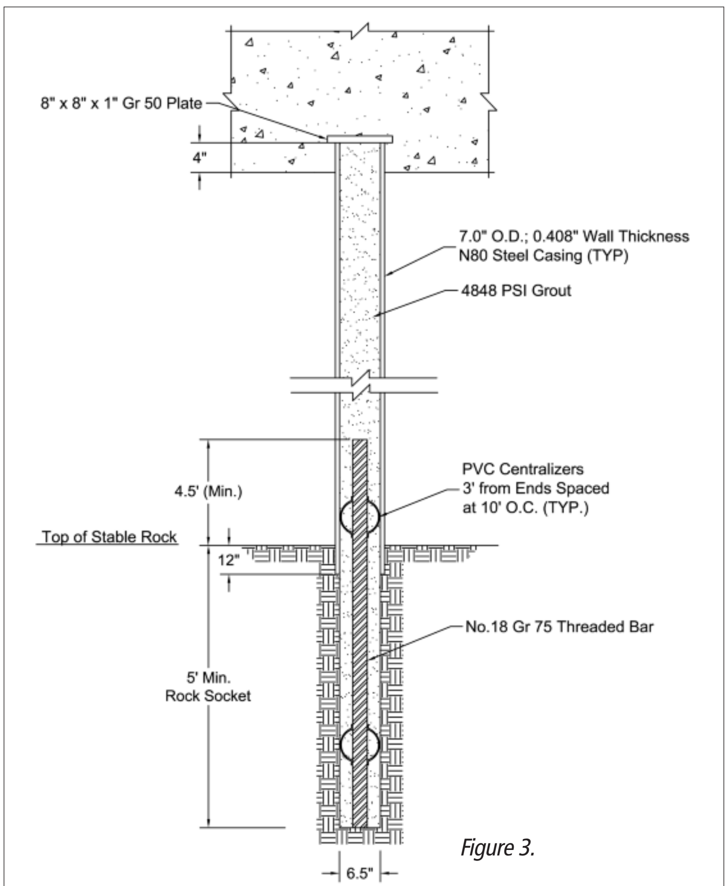

Figure 3.

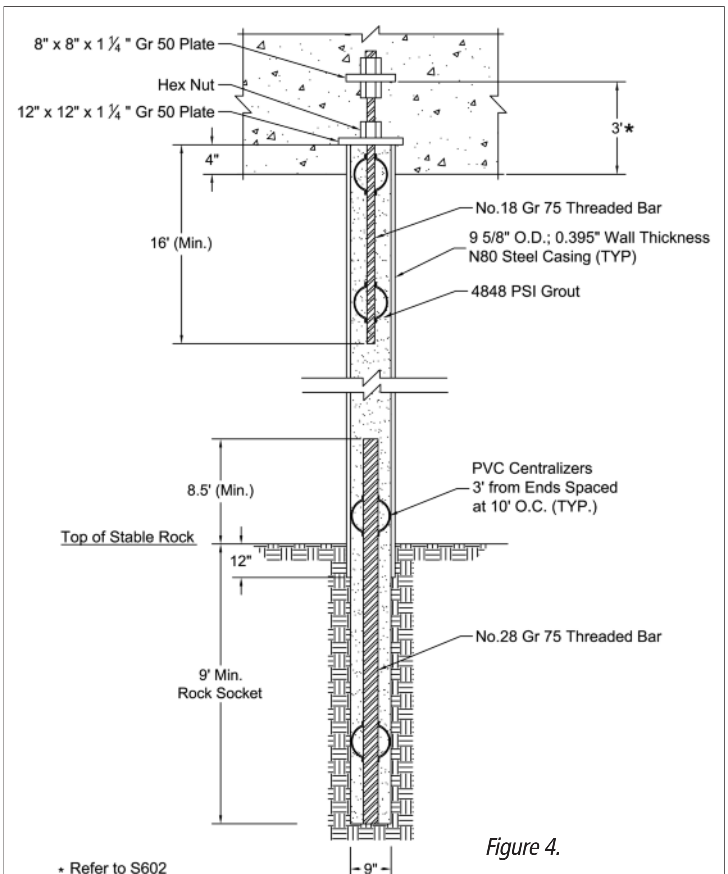

Figure 4.

casing (similar to tieback installations). These piles required full height core steel and, in some cases, a small diameter pipe in the upper 10 to 15 feet to provide for the seismic requirements. Over time, micropile design capacities have increased due to the availability of larger diameter mill secondary oil-country pipe. We are now installing 400 ton piles and planning to get to 500 tons soon. The use of this pipe, as permanent casing, has revolutionized the design of high capacity and deep rock-socketed micropiles. The relatively low cost of this pipe offsets the labor cost to extract it and provides significant structural design capacity within the cased portion of the piles. As a result, the emphasis of the core steel is to reinforce the rock sockets only. For very high capacity micropiles, additional core reinforcement will be required within the casing, but it is significantly less in combination with the permanent casing. The permanent casing also provides for both seismic reinforcement in the upper portion of the piles and some lateral capacity. As a minimum, both design considerations will require the permanent casing to have a depth of 10 to 20 feet.

The subsurface conditions at this site consisted of urban fill, Boston Blue Clay, glacial till and Cambridge Argillite. The depth to rock was approximately 115 feet. Hence, the piles ranged in total depth from 120 to 125 feet.

A pile load test was performed in tension on a sacrificial pile to confirm the design bond value for the rock and satisfy the local building code. Figure 5 shows a design sketch of the test pile with the locations of the strain gauges and tell tales. A 3.0" Grade 150 continuously threaded bar was utilized to provide the required 350 ton test load. We utilized this bar size to be able to reach a higher test load – 420 tons. A point to emphasize should be that the real and only value to a pile load test is to ensure that the proper socket length is determined to support the design requirements of the piles. We have great confidence (and factors of safety) in the capacity of the steel components and grout materials that we utilize in our micropile designs.

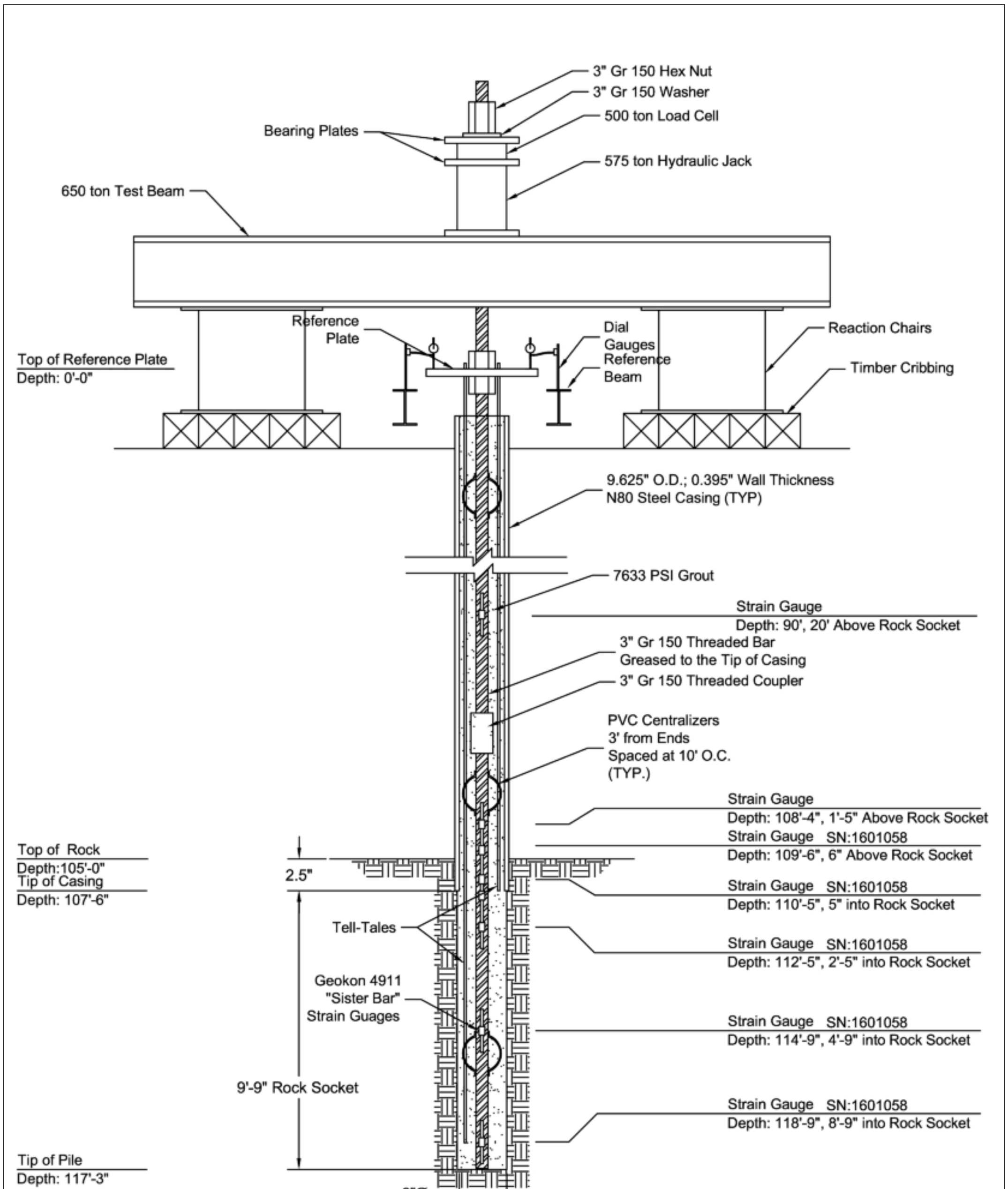

Figure 5.

Testing

Due to the lack of space onsite that would have supported the reactions to the test loads the pile load test was performed within the footprint of the existing structure. At this junction in time, the limited space within the project property line had been excavated for utility work and backfilled and/or filled with demolition debris. The access opening in the masonry wall was not large enough to allow for the test beam to be moved into the building area safely. The test beam required a large piece of equipment that would not fit through the opening. Therefore, the test beam had to be hoisted with a crane (Figures 6a, 6b and 6c) over the existing facade. A Grove 5275 crane, owned by Hub Foundation, was utilized to lift the test beam. The test was performed on January 19, 2015. Winter conditions in Boston can sometimes be very challenging, especially, with strong and cold wind gusts from the Northeast. The day of the test the high temperature was 15 degrees Fahrenheit with 20 mph winds. A temporary structure was erected over the pile load test frame and heated. Figure 7a and 7b shows the "hermetically" sealed test set-up. Figure 8 gives a view of the cozy interior with all the instrumentation in place during the load test. Throughout the day and into the night, during the long holding periods of the test, the crew enjoyed hot moose meat chili and barbecued specialties. These comforts made the crew and the inspectors very happy.

Figures 9 and 10 present the Deflection vs. Test Load plot and the Load Distribution plot within the test pile, respectively. The test pile was fully

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Figure 6a.

cased into stable rock and grouted full height. The test bar was greased down to the casing tip.

The test pile was initially loaded to twice the design load or 350 tons, at which point the creep rate at the tip of the pile was only 0.002″ over a 60 minute period. The elongation plot was essentially elastic over this test load range. The test load was increased to 2.4 times the design load or 420 tons. After 60 minutes the creep rate at the tip of the pile at this test load was still only 0.002". The bar experienced a sudden increase in elongation between 367.5 and 385 tons which remains somewhat of a mystery. We are surmising that the interaction of load transfer between the grout and the ID of the casing plus the effect of the couplers played a role in this behavior. The elongation stabilized in the subsequent three increments. The net settlement of the pile after the maximum test load of 420 tons was 0.540″. A projection of the net settlement had the test pile been unloaded at 350 tons would have been about 0.174″.

The load distribution plots in Figure 10 clearly show the benefit of both the permanent casing and the thorough greasing of the test bar. At all the


Figure 6b. Shows the precision it took to get the test bar over the church wall.


Figure 6c. Showing how close the test bar came to the church walls.

as, at the maximum test load of 420 tons. We utilized an allowable bond value of 125 psi with the original design. The test results indicate that this value could have been increased to at least 150 psi (possibly much higher). Nonetheless, all the production piles were installed with the original rock socket lengths since the rock sockets were relatively short in relation to the total length of the piles and did not have much impact on the overall installation schedule of the piles.

The original construction concept for installing the micropiles was to have been from the basement level with the existing wooden first floor intact above. The micropiles would have been installed while the demolition of the building occurred overhead with only the wooden floor providing protection. The access into the basement would have been through a five foot wide by seven foot high opening cut between two of the buttresses on the exterior wall. The safety concerns of pile installation with "heavy" demolition

Figure 7a and 7b showing hermetically sealed test site.

Figure 7b.

higher load increments, the full test load was transferred to the tip of the casing. However, over 80% of the maximum test load was "shed" within the upper 2.5′ of the rock socket. Only 8% of the maximum test load could be seen at the midpoint of the rock socket. Hub Foundation has performed several pile load tests in compression to maximum test loads of 700 to 1010 tons. A significant portion of the maximum test load was transferred at or just below the casing tip during these tests. This observation of the behavior of the load transfer mechanism in high capacity rock-socketed micropiles will require further study on how to advance future design requirements for these micropiles.

The pile load test was very successful at twice the design load, as well

activities directly above plus the higher installation costs of the piles in a low headroom scenario using a small drill rig convinced the general contractor to delay the pile installation until after all the demolition was completed. In conjunction with this, the project team performed an analysis of the masonry walls with the horizontal rafters and roof removed. The stability analysis demonstrated that the stone walls would remain stable during construction without the roof and rafters. Thus, those components were also removed during the initial demolition phase. This allowed for a much larger drill rig to be utilized. The speed of the larger rig enabled the recovery of a significant portion of time lost while waiting for the demolition to be completed.

Space within the foot print of the structure was limited and maneuverability of the drill rig and all the other equipment was very difficult. For this reason, the perimeter piles were installed first. This allowed for much

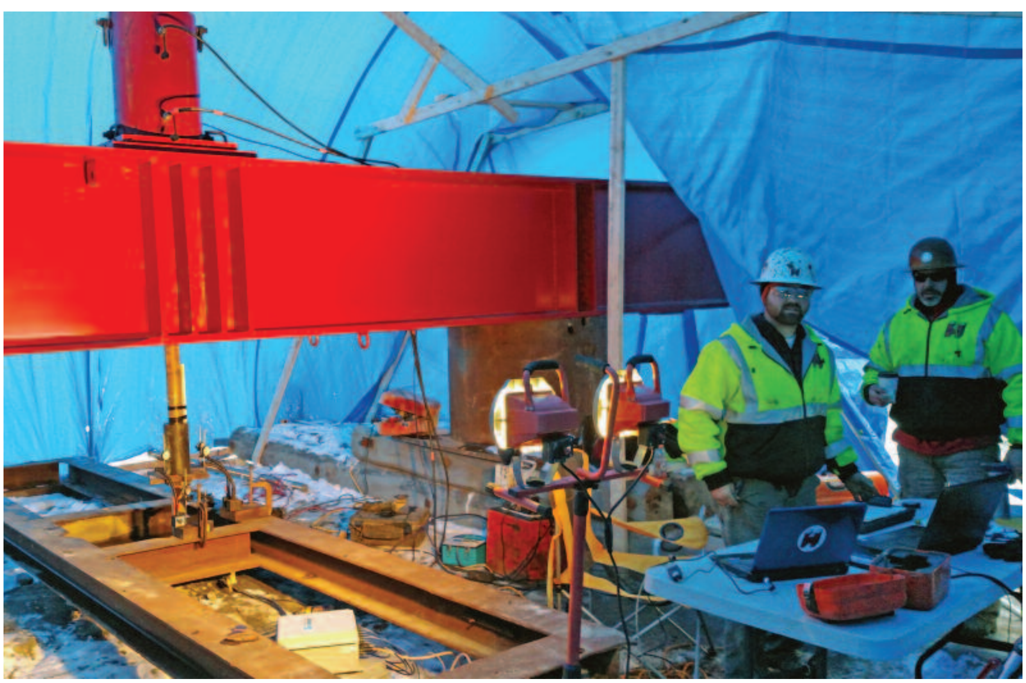

Figure 8. Left to right: John McKinnon and Charlie Silva, staying warm during load test.

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HUB Contd.

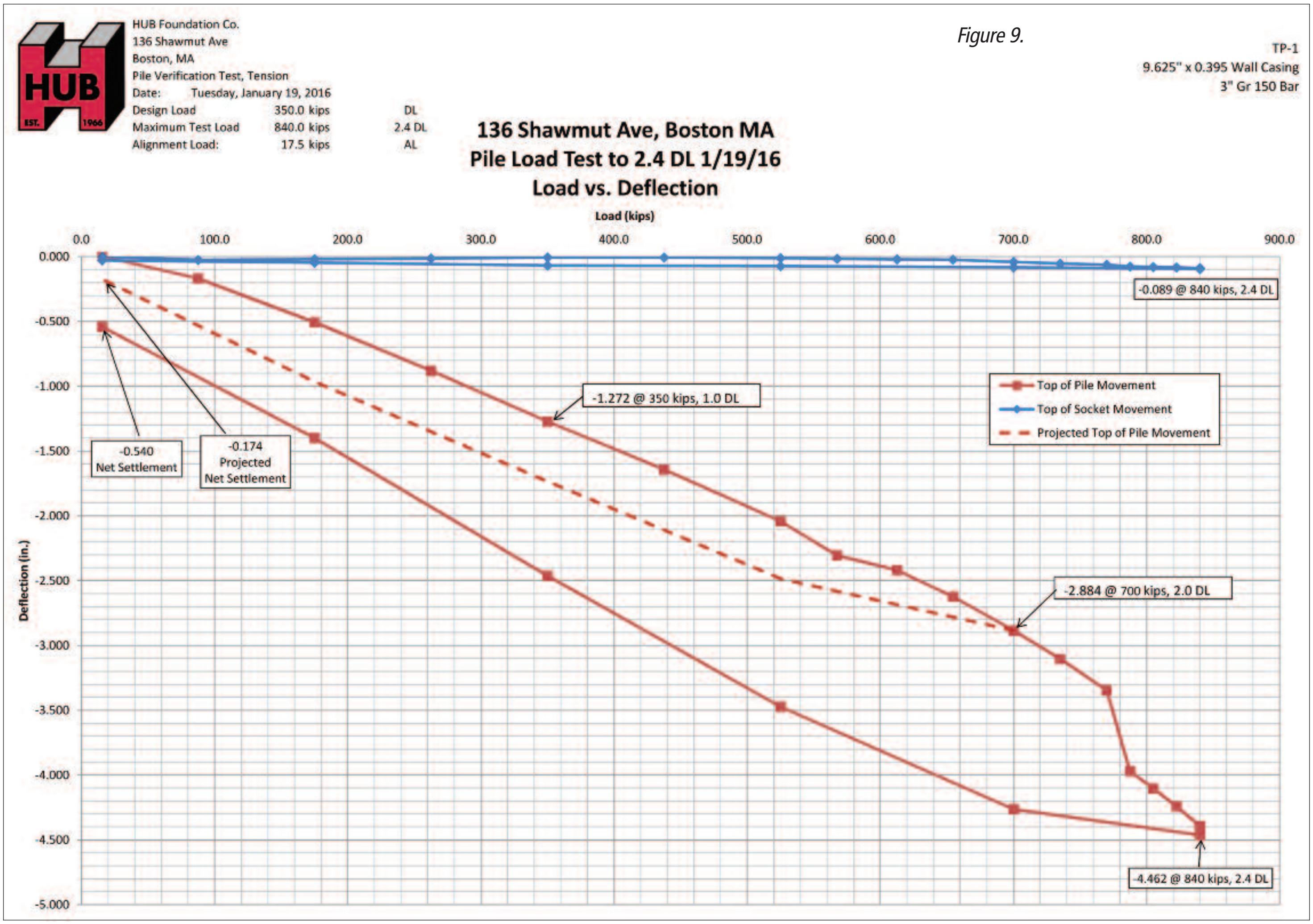

Figure 9.

better access along the masonry walls without the presence of the "stick up" of the interior piles had they been installed first. This sequence also allowed for a more controlled orientation of the system required to channel and collect the drill spoils while each pile was drilled.

The piles were drilled with a Casagrande* C-8 drill rig with a duplex drilling system. Wet-rotary internal-flush methods were utilized to remove the drill cuttings. Hub worked side-by-side with the site contractor to control the drill spoils within the site. The dense fill overlying the clay did not allow the drill water to filter through the soils. A recirculation system was employed to effectively control the spoils and reduce the volume of "wet" spoils that had to be disposed of offsite. Local trenches were excavated adjacent to the pile locations to initially channel the wet spoils to several pits. A centrifugal pump was utilized to pump the wet spoils into one watertight container. The heavy solids suspended within the "wet" spoils were allowed to settle out of the drill water. The "cleaner" fluids were directed into a second container from which a submersible pump was utilized to pump back to the drill rig.

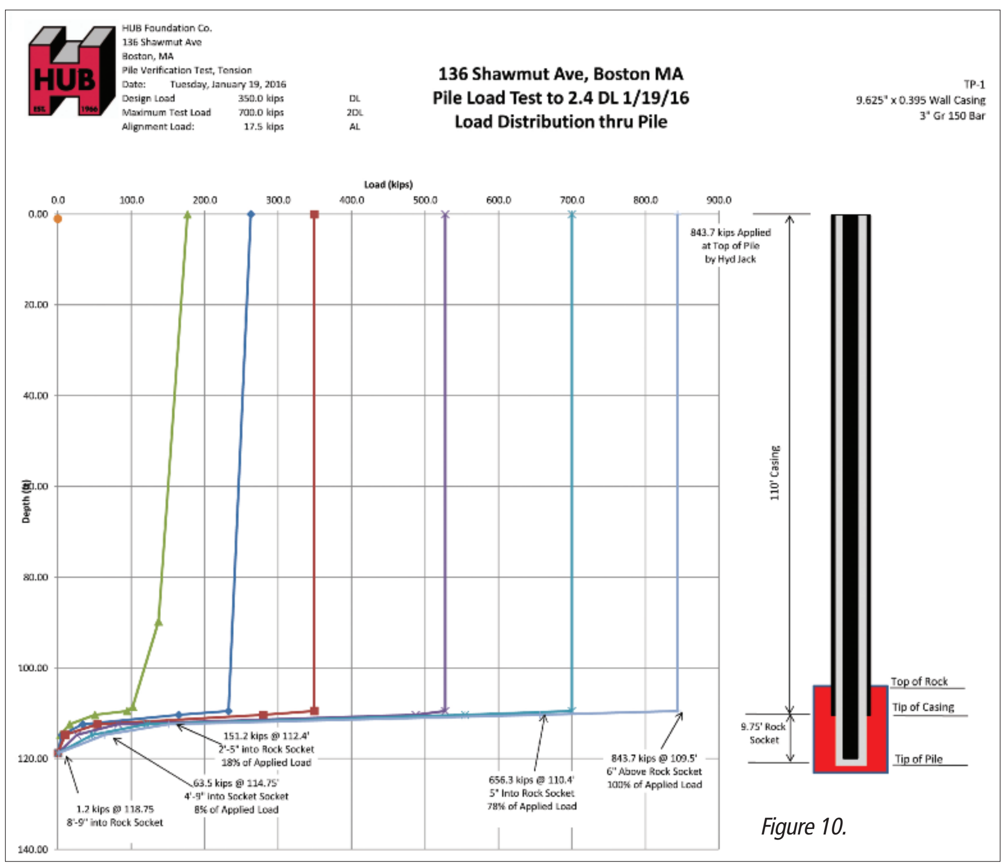

Figure 10.

A 6,000 pound Telehandler Lull was used to support the drilling operation and install the pile reinforcing prior to grouting the piles. A second crew was utilized to grout the piles once the drilling was complete. Since the piles were socketed into stable rock, several piles were drilled ahead of the grouting operation. This allowed the drilling operation to be completely independent from the grouting operation.

The perimeter piles were designed to support the new "exterior wall" of the structure. The existing stone facade would be tied to the exterior wall but would not provide structural support to the new structure. The perimeter piles were all located within 24 inches of the edge of the stone facade. Many challenges had to be overcome to successfully install these piles. Extreme care was required, especially, when the drill rig was moved to each pile location. There was concern that any significant impact of the upper portions of the stone walls by the drill mast could dislodge some of the smaller masonry materials. The crew maintained a safe distance from the walls at all times to ensure a safer environment. Figure 11 shows the drill rig exiting the tight opening at the completion of all the production piles.

The interior piles supported various elements within the new building. The core wall was the most prominent. The decision to drill the larger interior piles with a larger drill rig proved to be a blessing since these piles with a deeper rock socket were drilled in a relatively short amount of time. Also, the presence of approximately 15 feet of glacial till was not a significant factor drilling with the Casagrande C-8. This would not have been possible if the original low headroom drill rig had been utilized.

Figure 11. Tight quarters for drill rig.

Figure 12. Left to right: Jeffrey Crispo, Jacob Littlefield, John McKinnon, Maxx Martinez and driller Charlie Silva.

Summary

The micropiles were installed during winter conditions within tight access. All aspects of the installation were extremely successful. Figure 12 shows the key Hub employees that made this project a great success – they were all saints!

Project Team

Project Name:	136 Shawmut Avenue
Project Owner:	New Boston Ventures Group
General Contractor:	Metric Construction
Architect:	Finegold Alexander Architects
Structural Engineer:	McNamara/Salvia
Geotechnical Engineer:	McPhail Associates
Pile Designer:	GZA*, Larry Johnsen
Foundation Contractor:	Hub Foundation Co., Inc.*

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