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Rock Socketed Micropiles for Subway Station Improvement in Manhattan Schist

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Abstract. The improvement to the existing 63rd Street Station for the Second Avenue Subway in Manhattan, New York involved the installation of drilled and grouted micropiles socketed into schist bedrock. The micropiles were required as foundations to support two new ancillary structures and a new subway station entrance. The micropiles were installed within existing basements in adjacent buildings under confined space conditions. Load tests were conducted on sacrificial test piles to 3 times the design load to verify assumptions for grout-rock interface bond resistance. Additionally, proof load tests to 1.5 times the design load were conducted on selected production piles to confirm workmanship compliance. This paper presents the design concept for the foundations, interpretation of the load test results, and highlights the drilling issues encountered as well as the unconventional load testing procedures that were adopted due to constraints of the site.

Keywords. Micropiles, pile load test, compression, tension, lateral load.

1. Introduction

As part of the recent Metropolitan Transportation Authority's Second Avenue Subway expansion program in New York City, the existing 63rd Street Station at Lexington Avenue was retrofitted to accommodate an extension of the Q Line via a tunnel from 63rd Street to 96th Street beneath Second Avenue, with the addition of three new stations. The 200ft (60.9m) long, 60ft (18.3m) wide and 130ft (39.6m) deep station is located beneath 63rd Street stretching from Lexington Avenue to Third Avenue, with the upper egress passages located about 30ft (9.1m) from street level. The existing station is founded predominantly within the Manhattan Schist Formation.

The improvements to the station included four additional entrances to reduce egress time to meet the anticipated increase in passenger load. Two new ancillary structures were also constructed to accommodate additional fan plants and associated ventilation facilities. These additional structures were formed as extensions from the existing station platforms through breakouts or penetrations in the station structure, and connecting into existing adjacent basements, or passages and shafts that rise to the ground surface [1]. This paper presents the unique challenges for foundation design and construction at Ancillary 1 and 2, and Entrance 1, where the new spaces were created within existing building properties adjacent to the station and discusses the interpretation of the load test results.

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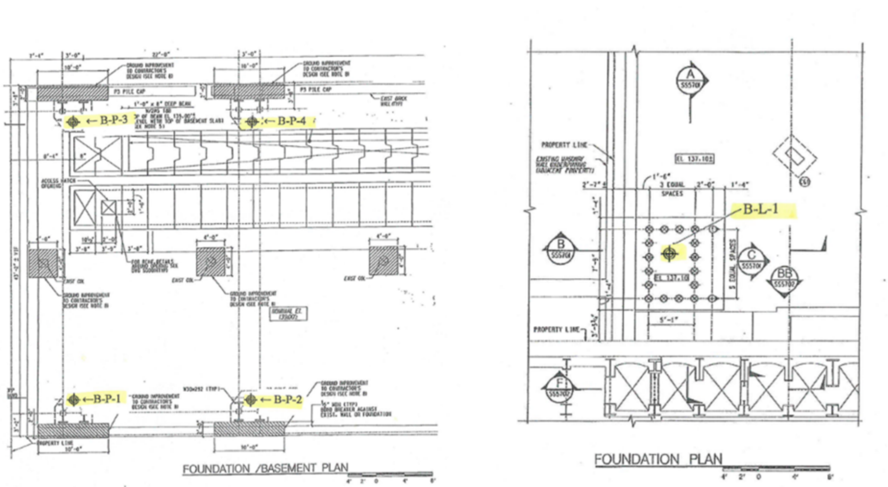
2. Foundation concept

2.1. Ancillary 1

Construction of the new facilities at Ancillary 1 included a cooling tower located within an easement taken at the far end of an existing multi-story garage. The garage was a four story brownstone building with a single level basement. The existing street level was at EL+150ft (+45.7m), with the basement slab and roof at EL+139ft (+42.4m) and EL+200.5ft (+61.1m) respectively. The tower structure comprised of a steel frame about 24ft by 18ft (7.3m by 5.5m) in plan and extended from the basement slab up to 28ft (8.5m) above the roof top. The tower legs had to be located as close to the existing basement wall as possible so as not to take up unnecessary parking space within the garage. The existing internal floors and roof slab were locally demolished to make way for the tower structure. Figure 1a shows the foundation layout for the cooling tower.



(a) Ancillary 1 (b) Ancillary 2

Figure 1. Foundation layout and geotechnical borings at Ancillary 1 and 2.

The new tower was designed to be structurally independent of the existing building. The foundations were therefore required to resist the full imposed loads, primarily due to wind loading as the self-weight of the structure was relatively small. A 3.5ft (1.07m) thick pile cap consisting of three vertical micropiles was designed to support each of the four legs of the steel frame. However, in order to fit within the confines of the easement, two micropiles had to be located along the basement wall directly beneath each column. A third pile was required to provide overturning stability of the foundation system to accommodate potential pile installation inaccuracies of up to 3ins (76.2mm) at the pile head. The micropiles were installed from the ground bearing slab within the existing basement and founded in the bedrock below.

2.2. Ancillary 2

The construction of Ancillary 2 included a new shaft located within very limited easement space inside an underground parking garage. The existing interior floor slabs had to be demolished to construct the new shaft. The shaft was located at the corner of the property immediately adjacent to a brownstone building with a single level rubble wall basement. The existing garage invert slab and the neighboring basement wall were founded on the bedrock at approximately the same elevation. The new shaft structure was required to be designed entirely independent of the existing buildings for stability. The new foundation comprised a 10.4ft by 9.8ft (3.17m by 2.98m) piled raft 2ft (0.61m) thick supported on eighteen vertical rock-socketed micropiles (Figure 1b). The top of the new raft slab was flush with the existing basement invert slab at EL+137.1ft (+41.8m). The bottom of the raft was located at the same level as the adjacent rubble wall foundations to avoid undermining the building. The micropiles were designed to resist vertical and lateral imposed dead and live loads.

2.3. Entrance 1

Entrance 1 was located across the street from Ancillary 2 at the intersection of 63rd Street and 3rd Avenue. The new facility was constructed within the basement of an existing seven-story tall residential building to provide an underground link to the 63rd Street Station. The building occupied a corner lot at the southeast sector of the street intersection. The basement comprised a single-level rubble wall with internal columns and walls supported by footings founded directly on dense native sandy soil. Retrofit of the first floor and basement structures was necessary to accommodate the new entrance facility. All temporary framing for the new structure was limited to below the second floor level and the basement. Construction was carried out while the building remained occupied by the tenants. The layout for the new foundations is shown in Figure 2.

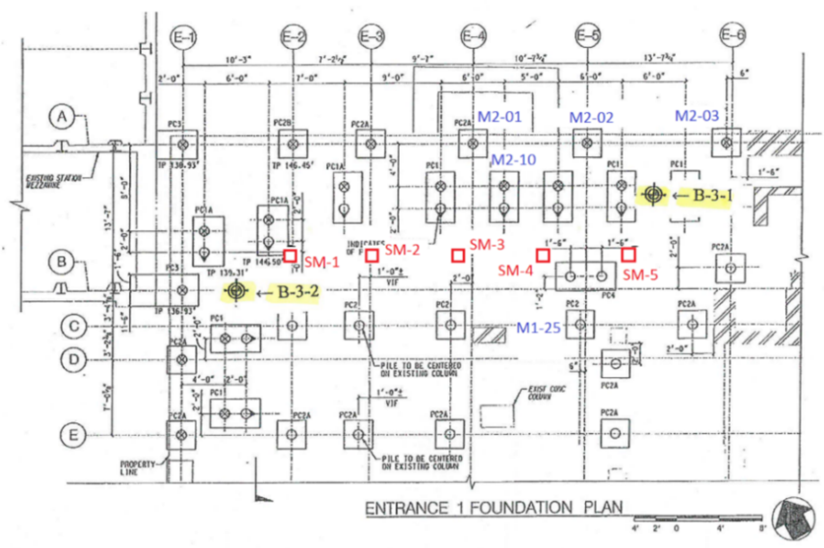


Figure 2. Foundation layout and geotechnical borings at Entrance 1.

The design called for the new construction to be structurally independent from the neighboring buildings, so that it will not rely on the adjacent buildings for lateral stability. Twenty single vertical micropiles were required to support vertical loads from columns and walls in direct compression. In addition, ten pairs of vertical and raked micropiles were required to provide lateral stability against sway of the building due to wind loads on the exposed northern and western faces, as well as lateral earth pressures acting on the basement walls. The micropiles were connected by pile caps and a 2ft (0.61m) thick base slab. The new top of base slab elevation was at EL+151.5ft (+46.2m). The micropiles were installed from the temporary excavation subgrade at EL+149.5ft (+45.6m) within the existing basement. The existing first floor slab was temporarily demolished to create more headroom for pile installation. Access for foundation work was from the street sidewalk at EL+156ft (+47.5m).

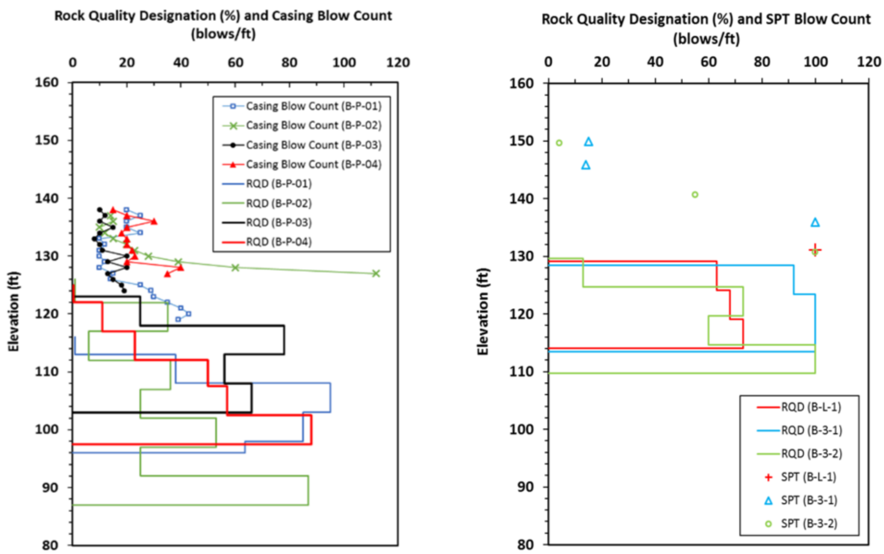
3. Geotechnical borings

Geotechnical borings were carried out to confirm rock conditions at each of the pile cap locations at Ancillary 1 (B-P-1 to B-P-4). Drilling commenced from the top of the existing invert slab at EL+139ft (+42.4m). As shown in Figure 3a, bedrock conditions were highly variable at Ancillary 1, with Rock Quality Designation (RQD) ranging from 0 to 100%. Top of rock elevations were between EL+116ft (+35.3m) and EL+126ft (+38.4m). As part of the investigation process, a 3ins (76.2mm) diameter steel casing was progressively driven using a 140lbs (63.5kg) DONUT hammer with 2ft (0.61m) drop height up to refusal on the rock. Driving blow counts suggested the fill and overburden soils were generally medium to dense. Additional borings were drilled at Ancillary 2 (B-L-1) and Entrance 1 (B-3-1 and B-3-2) from the top of existing slabs at EL+137.1ft (+41.8m), EL+151.9ft (+46.3m) and EL+151.6ft (+46.2m) respectively (Figure 3b). Rock conditions at Ancillary 2 and Entrance 1 were very similar, with RQD > 60% generally. Bedrock elevations were less variable, ranging from EL+128.4ft (+39.1m) to EL+129.6ft (+39.5m). Standard Penetration Test (SPT) measurements within the overburden soils ranged from N_{60} = 8 to 32 at Ancillary 1 (average N_{60} = 18) and N_{60} = 4 to 55 at Entrance 1 (average N_{60} = 22).

4. Micropile design and installation

Foundations on this project were evaluated based on the Allowable Stress Design (ASD) approach under serviceability limit states in accordance with the 2008 New York City Building Code (NYCBC) [2]. Three types of piles were specified: Type 1 [M1, vertical, with allowable axial capacity of 193 kips (858.5kN) in compression]; Type 2 [M2, vertical, with allowable axial capacity of 193 kips (858.5kN) in compression and 125 kips (556kN) in tension] and Type 3 [M3, raked at 1 (horizontal) to 2 (vertical) with allowable horizontal capacity of 60 kips (266.9kN) and 134.2 kips (596.9kN) in axial compression]. Permanent steel casings will be required for resisting moments and lateral shear. The design required the micropiles to be fully cased through the overburden soils and socketed into schistose rock, categorized as Class 1b Medium Hard Bedrock (50% < RQD < 85%) per NYCBC [2]. The US Federal Highway Administration (FHWA) [3] recommends grout-rock bond resistance (f_b) of 200 to 609psi (1380 to 4200kPa) for gravity-grouted micropiles (TYPE A) installed in good quality granite and basalt (fresh to moderately fractured, with little to no weathering).

An alternate evaluation of the grout-rock bond resistance was made using Horvath and Kenny's equation, f_b (psi) = (3 to 4)$\sqrt{q_u}$ where q_u is the unconfined compressive strength of the embedded material (in psi), derived from load tests on small diameter rock-socketed drilled piers and anchors with D < 16ins (406.4mm) [4]. The equation was based on tests in rock with strengths up to 6000psi (41.4MPa). For the current design it was assumed that failure will take place in the weaker grout material at the grout-rock interface. Based on a grout strength of 5000psi (34.5MPa), f_b was estimated to range from 212 to 282psi (1462 to 1944kPa). Hence an ultimate bond resistance of 200psi (1380kPa) was considered reasonable for preliminary design. A factor of safety of 3 was imposed to account for uncertainty associated with pile installation workmanship, in situ variability of rock quality and accuracy of design load assumptions. Rock socket lengths were determined to be 8ft (2.4m) for compression loading (Types 1 and 3) and 11ft (3.3m) for tension loading (Type 2) based on a borehole diameter of 8.5ins (216mm). As the micropiles were fully cased through the overburden soils to bedrock, the contribution of the overburden soils was neglected for load capacity determination.



(a) Ancillary 1 (b) Ancillary 2 and Entrance 1

Figure 3. Geotechnical borings at Ancillary 1 and 2, and Entrance 1

Each micropile was installed with a permanent steel casing, consisting of 9.625ins (244.5mm) outer diameter API pipe [yield stress, f_y = 80ksi (552MPa)] with wall thickness of 0.471ins (12mm). The casing was typically installed to the top of sound bedrock (RQD > 50%) and a tricone core bit or down-the-hole hammer was used to form the rock socket below it. A single 2.5ins (65mm) diameter Williams all-thread bar with nominal cross-sectional area of 5.19 ins^2 (3350mm^2) was installed for the full length of the pile [ultimate tensile strength, f_u = 150ksi (1034MPa); yield stress, f_y = 127.7ksi (880.5MPa)]. PVC centralizers were spaced at 10ft (3.05m) intervals and a grout cover of 3ins (76.2mm) was maintained around and below the bar. Grout for filling the borehole was mixed from 5 gallons (18.9 liters) of potable water to one 94lb (42.6kg) bag of Type II Portland Cement, with 0.5lb (226.8g) of FX-32 admixture added, to produce a neat grout with water-cement ratio of 0.44 and specific gravity of 1.75. The anticipated strength gain was 3000psi (20.7MPa) at 3 days and 5000psi (34.5MPa) at 28days. The pile was tremie-grouted over the full length. For Type 2 tension piles, the central reinforcing bars were installed with double corrosion protection within the rock socket length using corrugated plastic sheathing encapsulation and internally filled with high strength non-shrink grout per FHWA recommendation [3]. Additionally two gallons (7.6 liters) of calcium nitrite was added to the grout as corrosion inhibitor.

5. Test load application methodology

Pile load tests were carried out in general accordance with ASTM D3689 for tension mode of loading [5]. Test loads were applied in increments of 25% of the design load up to the maximum test load. Typically, each load increment was held for a minimum of 90 min (60min for TP-1), with the maximum test load observed for at least 12 hours (90mins for SM-2) before unloading. Unloading was carried out in 4 equal steps with each step held for 60mins. Rebound displacements at zero load was observed for at least 12 hrs. For trial tests on sacrificial piles, the maximum test loads were taken to 3 times the design load to confirm the rock socket bond resistance. For production piles, proof tests were carried out to 1.50 times the design load to check for workmanship. Limited working space at Ancillary 2 precluded the setting up of a test frame and no load test was conducted. Load tests were only conducted at Ancillary 1 and Entrance 1.

5.1. Pullout jacking reacted by wooden cribbing

At Ancillary 1, the production piles were located adjacent to the existing basement walls, and it was not possible to conduct proof load tests on the production piles. Testing was therefore carried out on a sacrificial pile TP-1 located in close proximity to the cooling tower. A standard 20ft (6.1m) long test frame was adopted comprising two W24 x 207 steel beams spaced 2ft (0.61m) apart reacting against a wooden cribbing at each end, bearing directly on the ground. The cribbing was formed from 12ins (304.8mm) square and 12ft (3.6m) long timbers stacked three layers high. The base of the cribbing was 4ft (1.2m) by 12ft (3.6m) in plan with the inner long edges located no closer than 8ft (2.4m) clear of the test pile edge. The loads were applied using a calibrated hydraulic jack with a center-hole double acting cylindrical plunger placed across a pair of MC18 x 58 jacking beams.

5.2. Pullout jacking reacted by piles

The micropiles at Entrance 1 consisted of both vertical and inclined piles. However, only vertical piles were load tested due to difficulty in setting up test frames within the restricted basement space. It was not possible to use cribbing for reaction against the ground and alternative testing arrangements had to be improvised by the contractor. Five sacrificial piles (SM-1 to SM-5) were installed in a straight line spaced 8ft (2.4m) apart (Figure 2). Two of the piles (SM-2 and SM-4) were tested using the two adjacent piles for reaction. A standard test frame configuration was adopted per Ancillary 1. Testing of the production piles proved to be more challenging as the test frame configuration had to be adjusted to fit within the confines of the space around the selected test pile. Production pile M2-02 was tested in tension by using three pre-installed production piles M2-01, M2-03 and M2-10 for reaction (Figure 2). M1-25 was tested in compression utilizing

M2-01 and M2-02 as reaction piles, while applying a tension load to SM-4 (Figure 2). A load cell was placed on M1-25 to measure the applied compression load at the pile top.

6. Interpretation of test results

Figure 4 shows the applied load versus pile top displacement for the test piles. It can be seen that the stiffness of the reaction support provided by the wooden cribbing for TP-1 was inadequate and considerable amount of load loss over the holding period was evident due to excessive deformation of the wooden cribbing. The other tests supported on reaction piles indicated more stable performance without any sign of load relaxation. The displacements at maximum test loads were small, less than 0.678ins (17.2mm). Observed creep displacements at each holding period was indicative of progressive redistribution of the applied load down the pile shaft over time as resistance is being mobilized. Creep displacement at the maximum test loads ranged from 0.0049 to 0.1048ins (0.12 to 2.66mm). The maximum rate of creep observed was 0.0087ins/hr (0.22mm/hr) for an applied load of 581kips (2584.4kN) at SM-4.

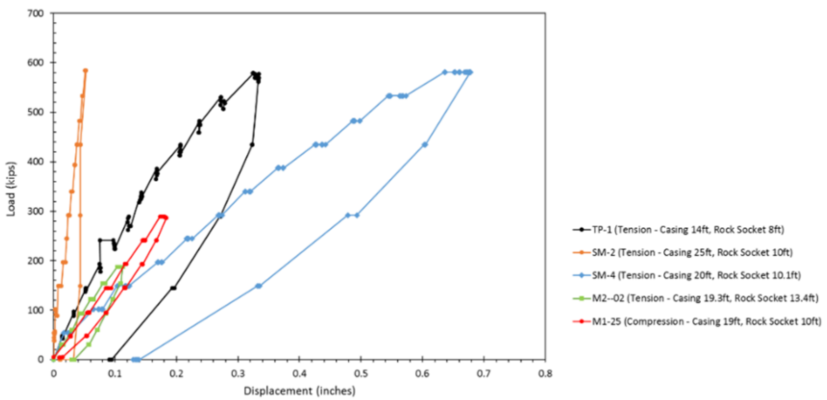


Figure 4. Applied load versus displacement at top of test pile.

For piles tested under tension loading, the loads were applied as a pulling force on the steel bar at the pile top. Although the maximum applied tensile loads were all less than the yield limit of 663kips (2949.2kN) for the steel bar [f_y = 127.7ksi (880.5MPa)], the non-linearity of the load-displacement response suggests that cracking in the grout may have occurred at the higher loading stages. In the case of a compression load test, the grout is confined within the casing and crushing of the grout will not occur if imposed strains are limited to below 0.003 [3]. This corresponds to a compressive stress of 12.1ksi (83.4MPa) for grout based on Young's modulus, E_g = 4030ksi (27.8GPa). The associated stress developed in the steel bar at this limiting strain is 87ksi (600MPa) [assuming Young's modulus of steel, E_s = 29,000ksi (200GPa)]. This is higher than the yield stress for the casing [80ksi (552MPa)]. Therefore, so long as the stress in the steel elements are limited to below 80ksi (552MPa), crushing of the grout is unlikely and the steel elements will remain elastic. Figure 4 shows this is the case for M1-25. At the maximum test load of 286.7kips (1275.3kN), the operating strain was estimated to be 0.00034 and compressive stresses in the steel and grout were on the order of 9.9 and 1.4ksi (68.3 and 9.6MPa) respectively.

Table 1 summarizes the back-analyzed rock socket bond resistance in the Manhattan Schist bedrock. The unit skin friction at the casing-soil interface was estimated using Meyerhof's relationship for low displacement piles, f_s (tsf) = $0.01N_{60}$ where N_{60} is the SPT blow count [6]. The load resisted by the cased length was calculated assuming average $N_{60} = 20$ blows/ft (20 blows/0.3m). The results suggest that contribution of the cased section was small, ranging from 2.5 to 10.4% of overall pile load resistance. From Figure 4, the ultimate failure loads have not been reached in all the tests. The average unit skin friction mobilized in the rock socket (f_u) for sacrificial piles ranged from 173.4 to 213.6psi (1195 to 1473kPa). Instrumented trial pile test for a similar micropile configuration elsewhere on the project suggested that ultimate unit skin friction in Manhattan Schist could be as high as 315.8psi (2177kPa) based on local strain gage measurements [7]. These results compare well with the estimated values of 212 to 282psi (1462 to 1944kPa) using Horvath and Kenny's equation.

Table 1. Summary of load test results.

Test Pile	Max. Applied Load	Max. Displacement at Pile Top	Cased Length	Rock Socket Length	Load in Cased Length	Load in Rock Socket	Rock Socket Friction
	kips	ins	ft	ft	kips	kips	psi
TP-1	561.5	0.333	14	8	14.1	547.4	213.6
SM-2	584.1	0.053	25	10	25.2	558.9	174.5
SM-4	581.0	0.678	20	10.1	20.1	560.9	173.4
M1-25	286.7	0.184	19.3	13.4	19.1	267.6	83.5
M2-02	187.0	0.112	19	10	19.4	167.6	39.0

7. Conclusion

This paper summarizes the design of drilled and grouted micropiles with rock sockets in Manhattan Schist to provide foundation support for new structural improvements for a subway station. The micropiles were installed using small drill rigs within existing basements in adjacent buildings. Innovative approaches were required for performing load tests within tight space constraints. Test results indicated that bond resistance on the order of 173.4 to 213.6psi (1195 to 1473kPa) can be mobilized in the rock socket at displacements less than 0.68ins (17.2mm) at the pile top. Horvath and Kenny's equation can be used to provide a reasonable estimate of the unit skin friction for rock socket design in Manhattan Schist.

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