Case History: Large Diameter *Micropiles* for the Highway 53 Relocation Project

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**ABSTRACT**

The Highway 53 relocation project in Virginia, Minnesota, involves construction of a very tall bridge across a currently inactive iron ore mine pit. The inactive pit is partially flooded and serves as the community’s drinking water source. The 1,132-ft long new bridge is a three-span, plate girder structure. Two intermediate, approximately 200-ft tall piers are located within the pit.

The foundations for the two piers consist of groups of 30-in diameter piles which, despite the contradiction in terms, are essentially very large micropiles. Extremely challenging and highly variable subsurface conditions exist at the site, ranging from uncontrolled iron ore mine waste fill to some of the hardest rock on earth that can also be highly fractured and abrasive. Despite the conditions, large diameter (30-in) micropile foundations up to 175 ft deep were successfully installed using down-the-hole hammer methods and full-length permanent casing under an extremely aggressive construction schedule.

Full-scale foundation load testing using the Statnamic rapid loading method was also conducted in support of the design and construction but under a separate pre-design phase contract. This paper describes the unique aspects of this foundation project, including design, construction, and load testing of the large micropiles.

**Keywords: micropile, foundation, bridge, pier, statnamic, load test, construction**

**INTRODUCTION AND BACKGROUND**

In 1960, a segment of Trunk Highway 53 (TH-53) in Virginia, Minnesota, was built on land owned by iron ore mining interests. As part of the roadway easement agreement, Minnesota Department of Transportation (MnDOT) was to be given notice three years prior to termination of the easement. In 2010, MnDOT received notice that the mining interests wished to access the ore. Through negotiations in 2010, MnDOT and the mining interests agreed to a 2017 date for abandoning the existing section of roadway. During this period, MnDOT needed to procure project funding, satisfy environmental permitting requirements, and acquire new right-of-way to permanently reroute TH-53 and keep it open to traffic. This was no easy task due to the existing mine pits surrounding the area.

The E-2 alignment option shown in Fig. 1 was ultimately selected for the rerouted highway in 2014. The new highway segment includes a three-span bridge across the currently inactive Rouchleau Mine Pit. The inactive pit is partially flooded and serves as Virginia’s drinking water source. The 1,132-ft long new bridge is a three-span, plate girder bridge. Two intermediate approximately 200-ft tall piers are located within the pit. Fig. 2 is a conceptual rendering of the bridge.

The foundations for the two intermediate piers consist of groups of 30-in diameter piles which, despite the contradiction in terms, are essentially very large micropiles. The piles include full-length permanent steel casings installed using down-the-hole hammer (DHH) equipment with a pilot and ring bit system. Micropile foundations are up to 175-ft deep including as much as 45 ft of bedrock drilling.
Fig. 1. Project Location and TH-53 Realignment (MnDOT 2017)

Fig. 2. Rendering of the TH-53 Bridge (MnDOT 2017)
GEOLeGIC SETTING

The project site is located within the Mesabi Iron Range. The Mesabi Iron Range is a narrow belt of iron ore deposits in the Superior Upland physiographic province of northeastern Minnesota. The bedrock unit of interest at the site is the Biwabik Iron Formation. The Biwabik was formed between 1.85 and 1.93 billion years ago as sediments deposited in a shallow marine environment. Since the early 20th century, the Biwabik Iron Formation has been subdivided into four members referred to as either cherty or slaty (Wolff 1917). The cherty members are typically characterized by a granular texture and thick-bedding; whereas, the slaty members are typically fine-grained and thin-bedded (Severson et al. 2010).

Intact pieces of the Biwabik rock are very hard and dense. The uniaxial compressive strength of specimens tested for this project averaged 20,600 psi with a maximum of 55,000 psi. Unit weight averaged 193pcf.

The fractured rock mass of the Biwabik Iron Formation at the project site is defined by bedding plane joints, typically dipping within 20 degrees of horizontal and sub-vertical joints, typically dipping between 70 and 90 degrees. Two or three sub-vertical joint sets exist across the site. The sub-horizontal bedding plane joints and sub-vertical joints form blocks. The blocks vary in size depending on the spacing of the bedding planes and the nature of the bedding plane contacts. Rock blocks or chips range in size from an inch or less up to several feet in maximum dimension.

SUBSURFACE CONDITIONS

The bridge crosses the Rouchleau Pit at a location where the pit is long and narrow, resembling a rock canyon. At this location, the pit is approximately 1,100 ft wide and 300 ft deep. The pit is flooded with water to a depth of approximately 120 ft at the bridge crossing. The western side of the pit has an embankment of mine waste fill (MWF) originally placed to create a mining haul road. The MWF embankment is approximately 130 ft thick. The schematic in Fig. 3 shows the approximate extent of the MWF. Borings in the MWF provided visual evidence of its general characteristics. PQ sized coring yields recovery of rock of the nature illustrated by the photo in Fig. 4a. Rotasonic drilling recovers predominately soil-like materials with rock fragments as shown in Fig. 4b. Each of the two sampling methods effectively recovers a biased sampling of material of different sizes; it is probable that the fill consists of a wide range of particle sizes, from fine granular soil to boulder sized rocks. A reasonable conclusion is that the MWF is highly variable but consists mostly of Biwabik Iron Formation rock fragments that may have been crushed, blasted, and/or weathered resulting in rock fill with interstitial granular soil. Determining the MWF composition was not critical for pile design, as axial resistance was to be fully developed in the bedrock. However, the MWF composition was of interest for determining the best foundation type from a constructability standpoint and pile lateral response.

Pier 1, on the eastern side of the pit, sits on a notch carved into the highwall with no overburden. Pier 2, on the western side of the pit, sits on the existing embankment of MWF. Piers 1 and 2 are both supported on large diameter micropiles bearing in Biwabik bedrock and are the subject of this paper. The abutments are each supported on spread footings bearing on Biwabik bedrock. The East Abutment includes tie-back anchors which reduce the lateral demands on the very tall piers.

The surficial bedrock at the Pier 1 notch is a bed known as the Intermediate Slate (IS). The IS is thin bedded and relatively fractured. Recovery and RQD in the IS average 88 and 46 percent, respectively, on the basis of three exploratory NQ2 sized core holes. Uniaxial compressive strengths of intact IS core specimens average 12,600 psi. The IS interfaces a cherty bed known as the Lower Cherty 7 (LC7) approximately 30 ft below the bottom of the Pier 1 footing. Recovery and RQD in the LC7 average 94 and 82 percent, respectively. Uniaxial compressive strengths on intact LC7 core specimens average 38,800 psi. The Pier 1 piles are tipped in the LC7 bed.

Bedrock underlying the MWF at Pier 2 is the Lower Cherty member of the Biwabik. The upper approximately 40 to 50 ft of bedrock is highly weathered on the basis of three exploratory NQ2 sized core
holes. Below the weathered bedrock is an intact bed known as the Lower Cherty 5 (LC5). Recovery and RQD in the LC5 average 100 and 83 percent, respectively. Uniaxial compressive strengths of intact LC5 core specimens average 16,500 psi. The Pier 2 piles are tipped in the LC5 bed.

Groundwater at the site is consistent with the surface elevation of the flooded pit, which was around elevation 1,310 ft through the duration of the project.

![Fig. 3. Bridge schematic profile with generalize subsurface conditions (MnDOT 2017)](image)

![Fig. 4a. MWF material recovered by PQ sized coring](image)

![Fig. 4b. MWF material recovered by Rotosonic drilling](image)
PRE-DESIGN LOAD TEST PROGRAM

A full scale, pre-construction load test program was conducted near Pier 2, on the west side of the pit where the mine waste overburden exists. Two 24-in diameter micropiles and one 16-in diameter micropile were installed. One of the 24-in piles included a 5-ft rock socket extending beyond the casing. The other two piles were fully cased.

Although load test performance of the piles was of interest, the primary goal of the test pile program was to evaluate constructability given the difficult and uncertain subsurface conditions. Veit Specialty Contracting (Veit) installed the test piles. A Birmingham BHD40 drill head and lead system attached to a Manitowoc 888 crawler crane was used in conjunction with Atlas Copco down-the-hole hammers (DHH) and tooling to simultaneously drill and advance casing. The reverse circulation method was used to transport the cuttings to the surface. Atlas Copco QL120 and QL200 DHH were used for the 16- and 24-in piles, respectively. An Atlas Copco Elemex pilot and ring bit system was used for the 24-in test piles and during the initial attempts to install the 16-in test pile. The 24-in equipment and tooling easily penetrated the MWF. However, there were issues with the 16-in ring bit detaching from the bit shoe in the overburden, allowing the drill to advance ahead of the casing. After two attempts in which the Elemex ring bit became detached from the casing, a Symmetrix pilot and ring system, also manufactured by Atlas Copco, was tried. The more robust Symmetrix system had no issues penetrating the MWF.

Bedrock drilling rate during the test pile program varied from less than a minute per foot to up to 33 minutes per foot. The deepest test pile, a 24-in pile, was tipped 187 ft below grade, penetrating approximately 65 ft into bedrock, although the upper 40 to 50 ft of bedrock was highly weathered. There were no issues drilling the cased holes. Initially, all three test piles were to include an uncased rock socket. The first rock socket attempted at one of the 24-in test piles collapsed shortly after drilling. The casing for that test pile was taken deeper and a second rock socket was not attempted. A 5-ft long rock socket was successfully drilled without stability issues at the other 24-in test pile. A rock socket was not attempted for the 16-in test pile. The reverse circulation drilling process left the bases of the holes clean without additional cleanout procedures employed, as verified by sounding with a weighted tape and subsequent observations during load testing.

The test piles were tested by means of an axial rapid load test (ASTM D7383-10) using a Statnamic device. All three test piles were loaded to a point where they began to demonstrate signs of structural failure without fully mobilizing the nominal geotechnical resistance. Review of the strain gage data indicated the piles were structurally overstressed during testing as manifested in prevalent cracking of the grout. However, because the grout was confined within a steel casing, full plastic yielding of the composite pile section did not occur. It can be concluded that under this substantial loading magnitude, the pile did not behave as a perfectly composite steel/concrete body.

While the strain gages did perform before, during, and after testing, the data were deemed unusable for the purposes of computing side resistance under maximum loading because of the cracking of the grout and the non-composite behavior. Test Pile 2 was loaded in two cycles, once with a smaller, 1,284-kip load that allowed measurement of partially mobilized side resistance. The partially mobilized unit side resistance in the rock socket under Load Cycle 1 was 18.3 ksf. Very little load was transferred in side resistance along the cased portion of the pile. The load test results are summarized in Table 1. The estimated mobilized unit base resistances in Table 1 assume all load applied load is carried in base resistance for Test Piles 1 and 3, and 75 percent is carried in base resistance for Test Pile 2.

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Max Applied Load, kips</th>
<th>Estimated Mobilized Unit Base Resistance, ksf [psi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-1 (24-in, fully cased)</td>
<td>4,088</td>
<td>1,301 [9,035]</td>
</tr>
<tr>
<td>TP-2 (24-in, with socket)</td>
<td>4,149</td>
<td>991 [6,879]</td>
</tr>
<tr>
<td>TP-3 (16-in, fully cased)</td>
<td>3,017</td>
<td>2,292 [15,917]</td>
</tr>
</tbody>
</table>
Upon unloading, the two end-bearing test piles (Test Piles 1 and 3) had no permanent set but rather rebounded and the final location of the pile top was above the location prior to testing. In fact, Test Pile 1 rebounded so dramatically that its pile top was approximately 2.0 in above the location prior to testing, as shown in Fig. 5. This highly unusual behavior was a result of a combination of the following:

1. enormous base resistance;
2. minimal side resistance; and
3. large elastic pile deformations.

The enormous base resistance results from a very hard, competent bedrock at the tip along with a clean hole prior to grout placement. The minimal side resistance was discussed above and anticipated as a result of the construction method which includes a slightly oversized hole relative to the diameter of the pile casing. The observed elastic pile deformations result from a relatively large magnitude of load applied to a long, relatively small diameter pile which, in cross-section, contained 20 percent steel and 80 percent grout. Upon dissipation of the rapid compression load, the elastic shortening of the pile was not restrained by sufficient side resistance causing large pile rebound accelerations.

Fig. 5. Test Pile 1 following Statnamic test (Bailey and Muchard 2015)

PILE DESIGN

Considering the construction experiences during the test program and the load test results, 24-in and 30-in diameter, fully-cased piles were selected as design alternatives for the Contractor to select. The larger pile size was allowed since the 24-in test piles had been installed without issue, and the larger 30-in diameter piles were not expected to be more difficult to install while allowing for fewer individual pile installations. Since the 24-in test piles had been installed with relative ease, the design team, including the
Contractor as part of an early involvement procurement process, felt utilizing slightly larger 30-in piles was feasible with little additional construction risk. The 30-in piles were designed as fully-cased, end-bearing piles where the axial capacity of the piles is limited by the structural capacity of the section as was demonstrated by the load tests. Uncased rock sockets were not included since there was no uplift demand and rock socket instability had been an issue during the load test program. Tie-down anchors at the Pier 1 footing were included to handle uplift loads during construction.

The piles casings are ½-in thick wall steel pipe with a minimum yield strength of 45 ksi. The piles are cast with a 7,000 psi self-consolidating concrete mix. The piles are unreinforced except for the steel casings and a small reinforcing cage at the top that facilitates a connection to the reinforced concrete pile cap. The maximum factored axial demand on a single pile is 2,776 kips. Piers 1 and 2 utilize groups of 14 and 19 piles, respectively.

PRODUCTION PILE INSTALLATION

Veit, the same foundation contractor that installed the test piles, was selected to install the production piles. Veit utilized similar equipment as was used for the test program although some components were larger due to the increase in pile diameter. A Birmingham BHD80 drill head and lead system attached to a Terex HC275 crawler crane was used in conjunction with an Atlas Copco QL200S DHH and Robit ROX+ pilot and ring bit system to simultaneously drill and advance casing.

The production piles were installed without significant issue. Although, at times drill rates did slow to as much as approximately 1 ft/hr. Production rates recorded during test and production pile installation are plotted versus depth in Fig. 6. There is a strong negative correlation between production rate and pile depth that become asymptotic with zero in the rock. Production rate is also influenced by pile diameter despite differences in DHH size and other compensating differences in equipment. Worn

![Graph of Highway 53 - Production Rate versus Depth and Diameter](image)

Fig. 6. Drilling Production versus depth for various diameter
bits, worn hammer internals, and insufficient supply of compressed air are also believed to have contributed to slow drilling at various times; however, depth appears to be the largest influencing factor.

As was observed during the test pile program, the cased holes drilled with the reverse circulato DHH equipment resulted in clean excavation bases without requiring additional cleanout effort, as determined by sounding with a weighted tape.

**PRODUCTION PILE PERFORMANCE**

As part of a comprehensive instrumentation and monitoring program for the project, two of the piles at Pier 2 are instrumented with strain gages, one in the northwest quadrant of the footing and one in the southeast quadrant of the footing. Each of the instrumented piles has 13 levels of gages with 4 gages at each level. Average force at each gage level is presented in Fig. 7 for measurements taken in March of 2017, after the piers were completed and girders placed but before the start of deck construction. The average top of pile force is approximately 1,035 kips based on the top gage level in each pile. This per pile force is consistent with the factored per pile dead load of 1,380 kips considering it does not include weight of the deck and nonstructural attachments.

The data indicate little to no load is shed in side resistance, which is consistent with observations during the design-phase load test program. In fact, the data indicate there is a net increase in load due to the accumulation of negative skin friction. The development of drag load due to negative skin friction is consistent with the conclusion of Fellenius (2006) that negative skin friction will always develop when installed through a softer stratum into a stiffer bearing stratum. Drag load was anticipated and considered for design utilizing the procedures of the MnDOT *Geotechnical Engineering Manual* (2013). The MnDOT procedures utilize the neutral plane method within the LRFD framework as described by (Siegel et al. 2013). The neutral plane analysis predicted 245 kips of drag load at neutral plane depth of 134 ft. The trends of the data suggest a drag load of 140 kips at a neutral plane depth of 80 ft.

![Fig. 7. Pier 2 Instrumented Piles, Load versus Depth](image-url)
OBSERVATIONS AND CONCLUSIONS

Large diameter (>12 in) micropile style equipment is capable of installing steel casings through cobble and boulder overburden and penetrating hard bedrock. The pilot and ring bit systems from two separate manufacturers performed well with the exception of the Elemex ring bit becoming detached during the first two attempts to install the 16-in test pile. Based on recorded production rates for all three diameter piles, installing cased piles much deeper into such hard rock would likely require more aggressive means of advancement. Supplying an adequate volume of compressed air to efficiently operate the DHH and evacuate the cuttings, particularly cuttings with a relatively high specific gravity, is necessary to drill effectively. The drilling method leaves a clean excavation base.

As hypothesized and demonstrated during the load test program, high geotechnical resistance in excess of the structural strength is achievable in hard and competent bedrock. The bit systems cut a slightly oversized hole necessary to advance the casing. The oversized hole substantially reduces the magnitude of side resistance even after the small annular void collapses around the exterior circumference of the casing following installation. Strain gage measurements from instrumented production piles also indicate very low side resistance.

Negative skin friction developed in the production piles despite the drilled installation. The neutral plane method produced a conservative prediction of drag load based on preliminary strain gage measurements during construction.

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REFERENCES


