STATNAMIC LOAD TESTING ON A 406 MM (16 IN) DIAMETER MICROPILE

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ABSTRACT

The Highway 53 relocation project in Virginia, MN involves construction of a very tall bridge across a currently inactive iron ore mine pit. This pit is partially flooded and serves as the community's drinking water source. The new bridge is 345 m (1,132 ft) long with a three-span, plate girder superstructure. Two intermediate 60 m (200 ft) tall piers are located within the pit. The foundations for the intermediate piers consist of bored pile groups installed using techniques common to micropile construction. Extremely challenging and highly variable subsurface conditions exist at the site, which range from uncontrolled iron ore mine waste fill to very hard rock that can also be highly fractured and abrasive. Despite the conditions, foundations up to 53 m (175 ft) deep were successfully installed using down-the-hole hammer methods and full-length permanent casing. An extremely aggressive construction schedule added to the difficulties. Full-scale foundation load testing using the Statnamic rapid loading method was conducted in support of the design and construction with a separate pre-design phase contract. This paper describes the Statnamic testing and associated unique behavior of a 406 mm (16 in) diameter test micropile.

INTRODUCTION

The new alignment of Trunk Highway 53 (TH 53), which is just southeast of Virginia, Minnesota and shown in Figure 1, is being relocated to allow for future iron ore mining operations. The new section of highway will include a three-span bridge (MnDOT bridge number 69129). The new bridge will cross the currently inactive and flooded Rouchleau Mine Pit. The proposed structure is approximately 345 m (1,132 ft) long with two abutments and two intermediate piers. The superstructure will be a steel plate girder bridge with eight girder lines. The abutments will be founded on spread footings and the two piers will be founded on groups of 762 mm (30 in) diameter bored piles. The term "bored pile" is used herein to describe drilled piles that are constructed using similar methods and equipment for micropiles, yet are larger diameter than typical micropiles. Although arbitrary, micropiles are generally considered to have a diameter of 305 mm (12 in) or less.

Due to the extremely heterogeneous subsurface conditions and the formidable foundation construction difficulties that were anticipated, a design-phase load test program was conducted prior to the construction contract. The primary intent of the test program was to verify the feasibility of constructing deep foundations with sufficient size and strength to support a 60 m (200 ft) tall bridge. The design must also accommodate dynamic demands, both geotechnical and structural, resulting from anticipated future mine blasting operations, in addition to the more typical force effects.

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The secondary intent of the test program was to perform axial compressive load tests on the test foundations. The potential benefits of design-phase load test programs are well established and generally include mobilization of nominal resistance values in excess of those estimated by design computations and increased reliability as manifested through increased resistance factors at the strength limit state. Accordingly, three axial compressive load tests were conducted on three separate test foundations using the rapid test method and associated Statnamic testing device. Note that other testing methods were considered, including traditional top-down tests and bi-directional cell tests, but, given the anticipated subsurface conditions, required loading magnitude, and pile type, the Statnamic device was determined to be the only plausible option.

Figure 1: Site location; Virginia, MN 288 km (180 miles) north of Minneapolis, MN.

The three foundations successfully constructed and tested included two, 610 mm (24in) diameter elements and one 406 mm (16 in) diameter element. The focus of this paper will be on the construction and load test behavior of the 406 mm (16 in) diameter foundation, which most closely represents the size typically associated with micropiles.

SUBSURFACE CONDITIONS

The test program area was located on Mine Waste Fill (MWF) material placed during past mining operations to create a haul road. The ground surface at the site was relatively flat with existing surface elevations between approximately 400 m (1,312 ft) and 402 m (1,318 ft). The MWF is approximately 37 to 40 m (120 ft to 130 ft) thick, extending to around Elevation 363 m (1,190 ft).

There is significant uncertainty regarding both the composition and placement of the mine waste fill. Historical records describing its composition or placement methods do not exist or are not available, but historic photographs providing general knowledge do exist. A historic photograph with the test program location super-imposed is provided in Figure 2. A similar view at the time of the load test program is presented in Figure 3.

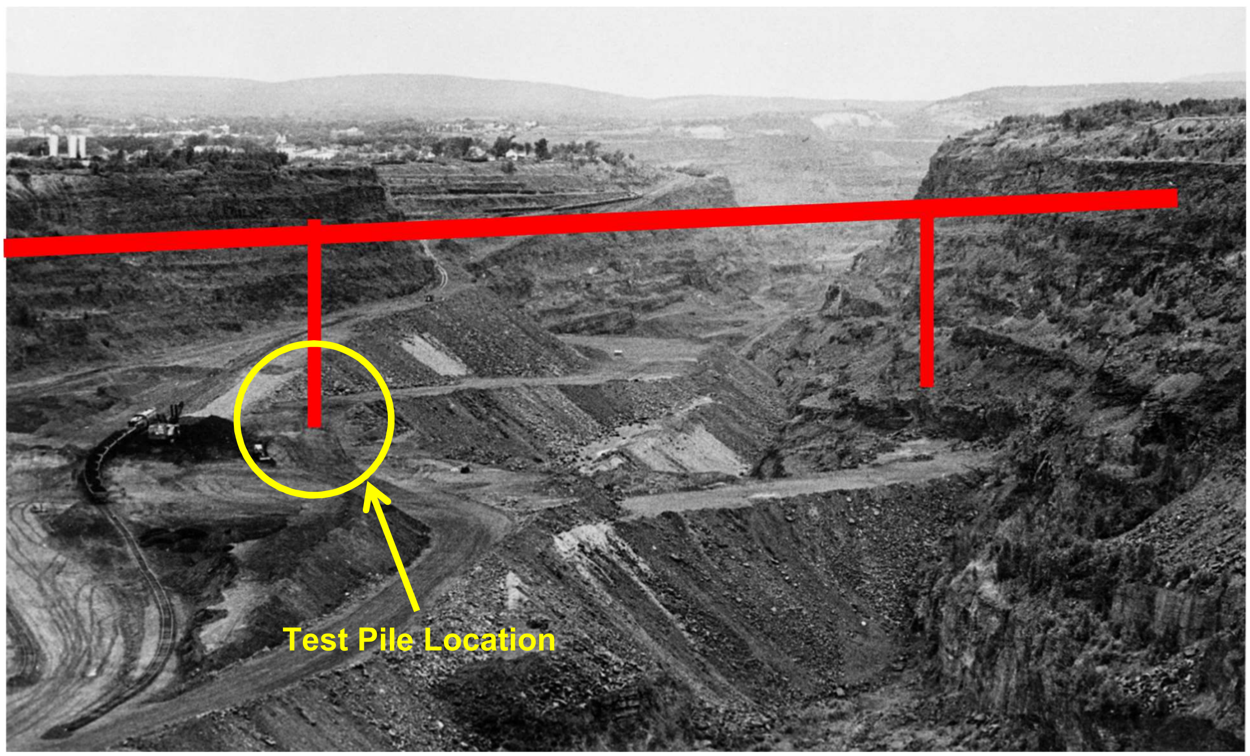

Figure 2: Site photograph circa 1940's during active mining and prior to flooding the pit; The red lines indicate the approximate location of the proposed bridge structure.

Borings in the MWF provided visual evidence of its general characteristics. Coring yielded recovery of rock of the nature illustrated by the photo in Figure 4-a. Rotosonic drilling recovered predominately soil-like materials with rock fragments as shown in Figure 4-b. Each of the two sampling methods effectively recovered a biased sampling of different material sizes. It is probable the fill consists of a wide range of particle sizes, from fine granular soil to rock particles that extend into the cobble and boulder size ranges. A reasonable conclusion is the MWF is highly variable but consists mostly of Biwabik Iron Formation rock fragments that may have been crushed, blasted, and/or weathered to produce a rock fill with interstitial granular soil.

Figure 3: Site photograph in March 2015 (load test site on left near the crane).

The particles comprising the MWF have higher density than typical soil particles due to the high iron mineral content. Specific gravity tests on three laboratory samples range from 3.404 to 3.827 (soil particle specific gravity for soils derived primarily from quartz is typically between 2.6 and 2.7). Based on the 3.685 average specific gravity for the three laboratory tests and an estimated range of void ratios between 0.3 and 0.8, the dry unit weight ranges from 20.4 kN/m^3 to 27.5 kN/m^3 (130 pcf to 175 pcf) and the saturated unit weight ranges from 24.3 kN/m^3 to 29.8 kN/m^3 (155 pcf to 190 pcf).

Below the MWF is bedrock of the Lower Cherty member of the Biwabik Iron Formation. The upper 12 m (40 ft) of bedrock was weathered and highly fractured based on borings, geophysical survey, and foundation construction observations. Core recovery in the weathered zone ranged from 23% to 100%, and rock quality designation (RQD) ranged from 0% to 87%. There is a particularly weathered zone between Elevations 360 m and 352 m (1,180 ft and 1,155 ft) that includes zones of no recovery and material described as residual soil. Bedrock below Elevation 351 m (1,150 ft), as shown in Figure 5, is more competent and is described as being thin to medium bedded with weak to medium oxidation. Joint descriptions varied from close to wide. Core recovery in the competent bedrock below Elevation 351 m (1,150 ft) is between 85% and 100% with most runs yielding 100% recovery. RQD in this zone ranges from 50% to 100%. Unconfined compression strength tests conducted on specimens taken from intact bedrock cores between Elevations 343 m and 352 m (1,126 ft and 1,155 ft) range from 45.7 MPa to 213.3 MPa (6,628 psi to 30,922 psi) with an average of 116.1 MPa (16,832 psi). Across the project site, nearly 200 unconfined compression tests were performed on recovered bedrock core samples. Uniaxial strengths up to 482.8 MPa (70,000 psi) have been measured with average compressive strengths well above 137.9 MPa (20,000 psi). This provides insight into the strengths that some particles may have in the MWF above bedrock.

(a)

(b)

Figure 4: MWF material recovered (a) by coring and (b) by Rotosonic drilling.

Figure 5: Biwabik Iron Formation rock core near the tip of the test pile.

Groundwater at the time of the test program construction and testing was at Elevation 399 m (1,310 ft), commensurate with the level of the adjacent reservoir in the flooded pit.

TEST PILE CONSTRUCTION

Veit was the specialty subcontractor that installed the test piles and the subsequent larger diameter production piles. The 406 mm (16 in) diameter test pile was cased over its entire 52.9 m (173.6 ft) length with 12.7 mm (0.5 in) wall steel casing. A Bermingham BHD40 drill head on a Bermingham lead system was attached to a Manitowak 888 crawler crane, as shown in Figure 6.

Figure 6: Test pile installation.

An Atlas Copco QL120 down-the-hole hammer was initially used in conjunction with a 406 mm (16 in) diameter Atlas Copco Elemex pilot and ring bit. However, that ring bit system proved ineffective and failed on two attempts during construction and was subsequently replaced with a Symmetrix pilot and ring bit, also manufactured by Atlas Copco. The Symmetrix system proved successful on the first attempt. Photographs of the damaged Elemex ring bit and collar upon extraction of the pile casing are provided in Figures 7 and 8.

Figure 7: Casing and damaged ring bit after extraction.


Figure 8: Damaged Elemex ring collar upon removal from casing.

As stated earlier, installation of the 406 mm (16 in) diameter casing began using an Elemex pilot and ring bit. At an embedment depth of 32.6 m (107 ft), the ring bit became detached. The casing was removed from the hole which subsequently collapsed. The average penetration rate was on the order of about 66 seconds per meter and entirely in the MWF. The penetration rate is only provided as an indicator and is a function of many variables including the volume of compressed air, discharge effluent flow rate, bit wear, and soil conditions, to name a few.

Upon extraction of the casing, the Elemex ring bit was replaced and the casing was installed in the same hole location as the previous attempt. At an embedment depth of 26.5 m (87 ft), the ring bit again became detached. The casing was again removed from the hole which subsequently collapsed. The average penetration rate was on the order of about 39 seconds per meter, again entirely in the MWF.

At this point, a 406 mm (16 in) diameter Symmetrix pilot and ring bit was attached and the casing was installed in the same location as the previous two attempts. The average penetration rate over depths commensurate with use of the Elemex system was on the order of about 46 seconds per meter in the MWF. Upon encountering fractured bedrock, the average penetration rate decreased to about 236 seconds per meter on average. The casing was advanced to a depth of 49.9 m (163.7 ft) or Elevation 351.6 m (1,153.3 ft), which placed the tip of the pile in weathered and highly fractured bedrock. Grouting was attempted after placement of a full-length reinforcing cage. Approximately 11.5 m^3 (15 yd^3) of neat cement grout was placed in the pile by tremie, but the entire volume of grout left the pile. Note the theoretical volume inside the casing was 6.5 m^3 (8.5 yd^3). At that point, the cage was removed to ensure it was not resting in fluid grout.

One week later, casing advancement continued and the casing was advanced another 3.8 m (12.6 ft) to a depth of 53.8 m (176.3 ft) or Elevation 347.8 m (1,140.7 ft) and tipped in relatively sound and less fractured bedrock. Over the final 30 cm (1 ft) of casing embedment, the penetration rate decreased substantially to about 43 minutes per meter. The cage was placed again and grouting was conducted to finalize construction of the test pile. Again, neat cement grout was placed from the bottom-up using a 38 mm (1.5 in) diameter tremie. The theoretical volume was about 6.9 m^3 (9.0 yd^3) and matched well with the volume placed.

The reinforcement cage consisted of four number 18 (57 mm diameter), grade 75 (517 MPa yield stress) all-thread longitudinal bars with number 4 (12.7 mm diameter) grade 60 (414 MPa yield stress) hoops on a 30 cm (1 ft) vertical spacing, as shown going into the casing in Figure 9. Threaded couplers were used to splice 12.2 m (40 ft) sections of longitudinal reinforcing together. Applied Foundation Testing (AFT) attached sisterbar strain gages to the longitudinal bars as the cage was hung in the vertical position.

Neat Portland cement grout was tremie placed using a Schwing BPA-450 grout pump from the bottom-up in the fully-cased excavation after placement of the reinforcement cage and verification of a clean bottom with a weighted tape, as shown in Figure 10. The grout was mixed offsite and delivered to the site in ready mix trucks.

A conceptual schematic of the completed test pile is provided in Figure 11 relative to the subsurface conditions.

The 28-day design strength of the neat cement grout was 34.5 MPa (5,000 psi) although compression test results conducted on test specimens cast in the field resulted in an average 28-day strength of 48.3 MPa (7,000 psi). However, the 406 mm (16 in) test pile was tested 44 days after grout placement and the compressive strength at the time of testing was extrapolated and estimated to be just under 55.2 MPa (8,000 psi) at that time. The extrapolated strength of the grout commensurate with the time of the test was used to analyze the results.

Figure 9: Installation of reinforcing cage.

Figure 10: Completion of grout placement.

STATNAMIC LOAD TESTING

The rapid load test method using the Statnamic device was performed by AFT using their 18.7 MN (4,200 k) mechanical catch device in general accordance with ASTM D7383-10 "*Axial Compressive Force Pulse {Rapid} Testing of Deep Foundations*". Multiple levels of embedded strain instrumentation were installed in the test pile and monitored during the single cycle load application. Additional instrumentation included a calibrated load cell, multiple pile top accelerometers to attain pile top displacement via double integration, and optical survey of the pile top before and after testing as redundant measure of permanent displacement.

Pairs of strain gages were installed at 180° separations at seven locations along the test pile. The strain gage locations are provided in Table 1.

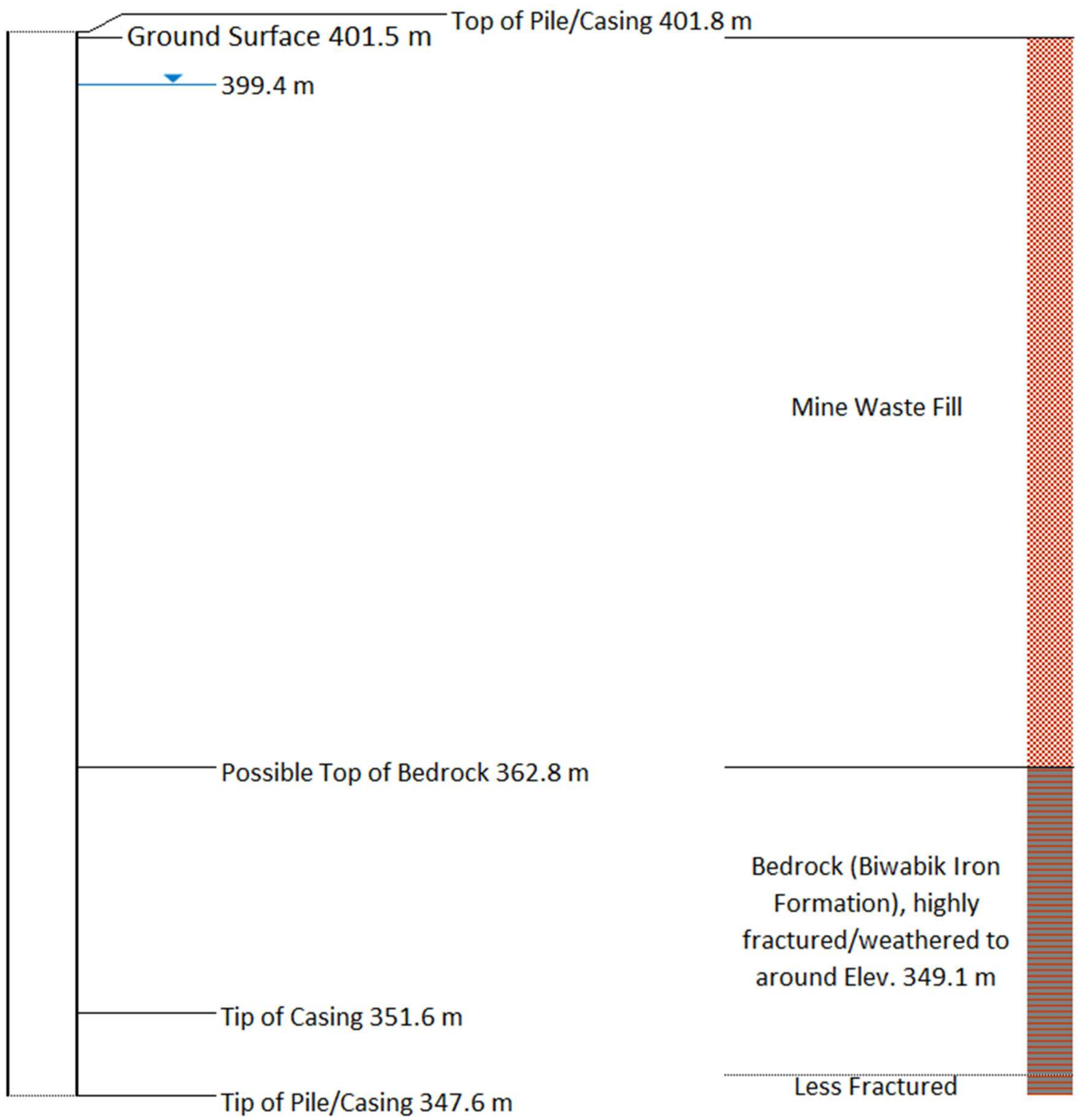

Figure 11: As-built schematic of 406 mm (16 in) diameter test pile.

Table 1: Test pile strain gage location summary.

Location	Depth Below Pile Top (m)	Elevation (m)	Adjacent Soil or Rock Material
Pile Top	0	401.8 (1,318 ft)	--
Surface	0.3 (1 ft)	401.5 m (1,317 ft)	MWF
Level 1	0.9 (3 ft)	400.9 m (1,315 ft)	MWF
Level 2	14.3 (47 ft)	387.5 (1,271 ft)	MWF
Level 3	28.0 (92 ft)	373.8 (1,226 ft)	MWF
Level 4	40.9 (134 ft)	361.0 (1,184 ft)	Weathered & Fractured Biwabik
Level 5	44.8 (147 ft)	357.0 (1,171 ft)	Weathered & Fractured Biwabik
Level 6	49.1 (161 ft)	352.7 (1,157 ft)	Weathered & Fractured Biwabik
Level 7	53.7 (176 ft)	348.2 (1,142 ft)	Relatively Competent Biwabik
Pile Tip	54.3 (178 ft)	347.6 (1,140 ft)	Relatively Competent Biwabik

Of particular interest is the observation that either of the ring and pilot bit systems produces a slight overcut necessary to advance the full-length casing. Theoretically, the annular void remaining around the external circumference of the casing is about 12.7 mm (0.5 in) if the walls of the excavation remain perfectly stable. During design of the test program, it was anticipated the MWF, and to some extent, the weathered and highly fractured Biwabik Iron Formation bedrock, would collapse the small annular void around the pile. Accordingly, some reduction in side resistance was expected although the precise magnitude was unknown. It was anticipated that an appreciable amount of side resistance would be mobilized during load testing along the 53.7 m (176 ft) long pile.

During construction of the test pile and following two separate failures of the Elemex bit system, the following important observations were made during removal of the casing from 32.6 m and 26.5 m (107 ft and 87 ft), respectively:

1. Upon removal of the casing, the hole did collapse as expected, providing some qualitative indication the small annular void would collapse; and
2. Despite the annular void collapse, insufficient side resistance existed to prevent extraction of the casing despite the depth of embedment.

Prior to load testing, these observations suggested the pile would be continuously supported in side resistance along its length but also the magnitude of side resistance would be relatively low. Therefore, it was estimated the majority of the load would be transferred to the tip of the pile and transferred to the very hard bedrock in base resistance.

Forty-four days after being cast, the pile was load tested. A maximum derived static load of 13.4 MN (3,017 k) was applied to the pile head, which equates to an average pressure of 103 MPa (15,000 psi); nearly twice the unconfined compressive strength of the grout. The total displacement of the pile head during testing was 138 mm (5.44 in).

Upon unloading, there was no permanent set but rather it was observed the pile had rebound and the final location of the pile top was approximately 13 mm (0.5 in) above the location prior to testing. In fact, Test Pile 1, which was a 610 mm (24 in) diameter pile, rebounded so dramatically that its pile top was approximately 51 mm (2.0

in) above the location prior to testing, as shown in Figure 12. 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. 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% steel and 80% grout.

Upon dissipation of the rapid compressive load, the elastic shortening of the pile was not restrained by sufficient side resistance causing large pile rebound accelerations. This resulted in the pile rebounding upward out of the ground. After coming to rest, the available side resistance was sufficient to support the static weight of the test pile.

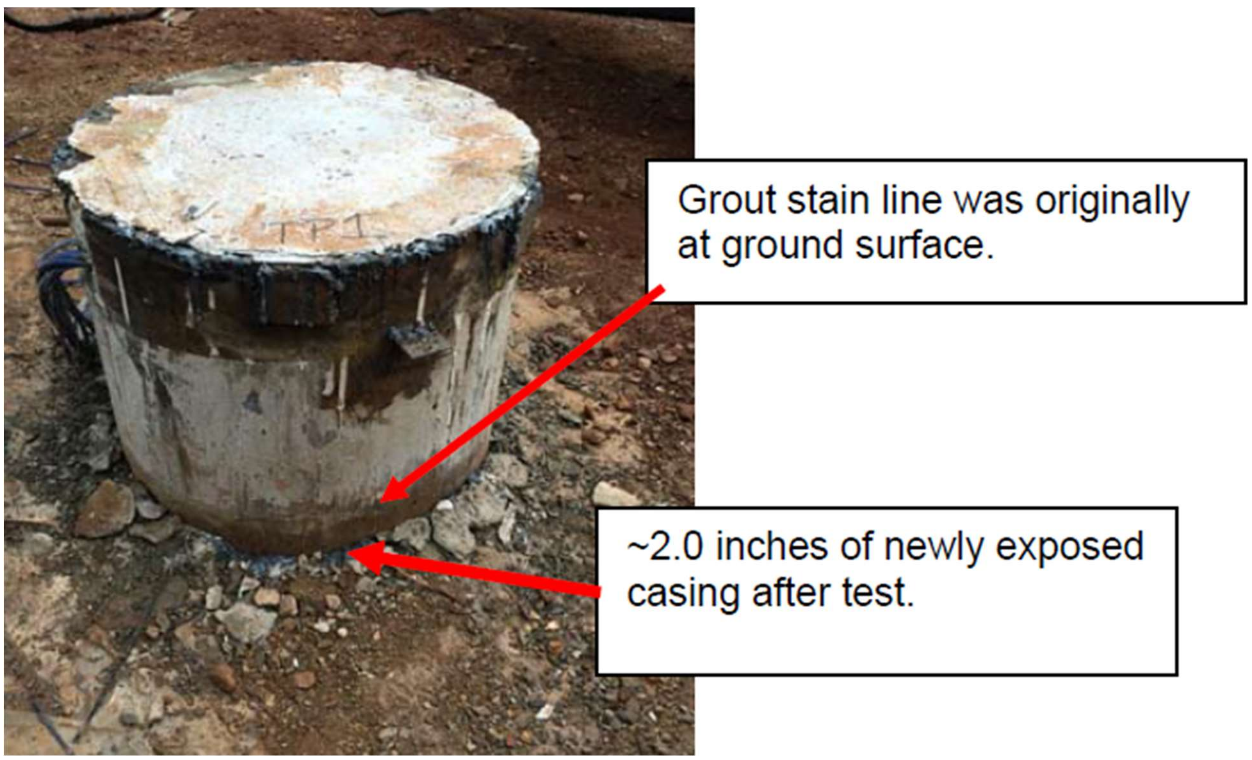

Figure 12: 610 mm (24 in) diameter test pile following Statnamic test.

Review of the strain gage data indicated the pile was 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 was deemed unusable for the purposes of computing the side resistance because of the cracking of the grout and the non-composite pile behavior. This unfortunate fact is estimated to be a result of applying such a large magnitude of load, which the pile could obviously handle with ease from a geotechnical perspective but which caused distress structurally.

CONCLUSIONS

The primary intent of the test program was to evaluate the constructability of the deep foundation elements using essentially large micropile installation techniques. It was determined the proposed foundation type was indeed constructible despite the formidable subsurface challenges. While initial attempts with the Elemex bit system proved unsuccessful, the Symmetrix system was capable of installing the casing in an economical manner.

Both bit systems produce a slightly oversized hole necessary to advance the casing. This oversized hole substantially reduces the magnitude of side resistance even after the small annular void collapses around the exterior circumference of the pile after installation.

The Statnamic test applied a derived static compressive load of 13.4 MN (3,017 k) with a corresponding total displacement of 138 mm (5.44 in). The combination of enormous base resistance, minimal side resistance, and huge elastic pile deformation under rapid compressive load produced a highly unusual behavior in which the pile rebounded upward out of the ground following load dissipation.

As hypothesized prior to, and demonstrated during, the axial design of the pile was controlled by structural considerations rather than geotechnical limitations. Even in the absence of appreciable side resistance, sufficient base resistance exists that exceeds the structural strength of the pile. Finally, the maximum value of base resistance was not fully mobilized, even under the enormous stress applied during the Statnamic test.

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