MICROPILES AT THE WEST RIVER BRIDGE, NOVA SCOTIA

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ABSTRACT

The new Highway 104 West River Bridge in Antigonish, Nova Scotia, Canada is supported on 152 micropiles.

Despite having never used micropiles for new construction in the past, the owner selected micropiles for this project for a variety of reasons pertaining to the subsurface conditions into which the piles are embedded. The bridge site is characterized by varying thickness till overburden with cobbles and boulders, and soft mudstone bedrock with soluble inclusions and intermittent harder layers of anhydrite. Compared to other conventional deep foundation options, the various small diameter, percussive drilling and pressure injection processes available for constructing micropiles proved well suited to overcoming the design and constructability challenges at this site.

Maximum factored axial compressive loading is 1586 kN per micropile; as-built micropile depths range from 15 m to 33 m. The micropile component of the project was constructed on a post-tender alternative design proposed by the micropile contractor. Prior to the commencement of full production piling, two distinct installation processes using identical reinforcement, identical bond lengths but differing bond zone diameters and differing drilling/grouting processes were subjected to full scale load testing to failure, with one method exhibiting clear advantages compared to the other. This paper provides a case study of this landmark project for Atlantic Canada, and details the methods of construction and testing for both the pre-production and permanent micropiles.

INTRODUCTION

Canada’s iconic east coast is renowned for its quaint fishing villages, unspoiled wilderness and old world charm. The northern extent of mainland Nova Scotia consists of a spine of highlands (remnants of the Appalachian chain) surrounded on both sides by lowlands framed by
countless tidal estuaries thick with reddish brown mud, one after the other connected to the Atlantic Ocean to the east or the Gulf of St. Lawrence to the west.

Nova Scotia’s provincial Department of Transportation and Infrastructure Renewal continues to upgrade the province’s 100-series highways (Fig. 1), which connect the major provincial municipalities to rural areas and include the primary terrestrial link to New Brunswick and the rest of Canada. A significant effort is currently underway on a new 7 km long, four-lane fully divided segment of Highway 104 that will bypass part of the existing two-lane highway that runs through the historic town of Antigonish.

One of the two major bridge structures required at river crossings along this new alignment is the West River Bridge. The West River Bridge was initially designed as a 241 m long structure with multiple intermediate piers. Subsequent refinements of the bridge design resulted in a 141m long structure comprising two twin spans supported at four abutments and two central pier locations (Fig. 2).

Foundation construction included the installation of 152 micropiles during the summer and fall of 2011. The West River Bridge represents the first new bridge structure in Nova Scotia to be supported on micropile foundations. Micropiles were considered the best deep foundation solution to overcome challenging overburden soil and bedrock conditions prevalent on the site. The structure of the contract for the work included full scale sacrificial load tests, which included an assessment of two distinct installation methods utilizing different sized bond zones and
grouting / post grouting techniques. Test results enabled selection of an optimized, site-specific micropile construction technique, which was also confirmed through systematic proof testing on production piles.

![General arrangement of West River Bridge micropiles](image)

**Fig. 2: General arrangement of West River Bridge micropiles**

**SUBSURFACE CONDITIONS**

**Geological Setting**

The West River Bridge spans the West River, which flows through the town of Antigonish and into the harbour approximately 2 km downstream from the bridge site. Antigonish Harbour is protected by beaches and dunes at its confluence with St. George’s Bay and the Gulf of Saint Lawrence to the north.

Regionally, the site is located near the western edge of the Antigonish-Guysborough Lowlands, with the Antigonish Highlands located to the immediate north and west. Locally the site is situated in the West River Valley. The elevation of the ground surface at the base of the
valley is approximately 4.0 metres above sea level and the drumlin ridges forming the top of the valley in the vicinity of the bridge site range in elevation from approximately 30 to 32 metres above sea level. The slopes of the valley walls range from approximately 4% to 7%.

**Overburden**

Native overburden soils in the general vicinity of the site are dominated by Wisconsinan age silty glacial till plains and drumlins (Stea, 1992). The glacial overburden soils in the base of the West River Valley have been eroded by post-glacial flows, with Holocene age alluvial deposits underlying and surrounding the current river channel. Recent fill soils are also present locally above the native soil strata at the bridge site. The local groundwater table is generally controlled by the level of the West River.

The glacial till underlying the site is a compact to very dense, well graded mixture of clay, silt, sand, and gravel with cobbles and boulders. A distinct layer of cobbles and boulders was encountered within the glacial till unit in some pre-project investigation boreholes.

**Bedrock**

The site is underlain by the Mabou Group (Keppie, 2000), which includes siltstone, shale and sandstone, and the Windsor Group, which can include mudstone, minor gypsum and anhydrite units. Of consequence to the project was the presence of both mudstone and anhydrite.

The red mudstone exhibited a ‘soft’ engineering behaviour, with all measured UCS values less than 1 MPa, classifying the rock as extremely weak (ISRM, 1981). The rock also included many layers that were completely weathered to a soil-like state.

The anhydrite unit was more competent than the mudstone unit with measured UCS values typically 15 MPa. However, anhydrite and gypsum of the Windsor Group are known to be soluble, associated with karst terrain, and potentially susceptible to ongoing development of solution features such as sinkholes and solution cavities (Adams, 1991 and Martinez, 1997).

**FOUNDATION DESIGN CONSIDERATIONS**

Numerous foundation options were considered for supporting the new bridge. Shallow foundations were deemed acceptable for supporting ancillary structures but the loads at the piers and abutments were greater than could be resisted by footings. The presence of a clay layer over the northwest portion of the site was also a consideration, since this was potentially problematic in terms of long term consolidation settlements and embankment stability.

Both driven and drilled deep foundation options were considered. Driven steel piles are typically an economical foundation option in Atlantic Canada. However, the soluble anhydrite
bedrock precluded their use due to concerns over potential dissolution of the anhydrite bedrock at the pile toe, creating the potential for loss of end-bearing capacity. Another concern was the potential for installation problems and shallow refusal in cobbles and boulders in the overburden.

The preferred deep foundation option was a drilled pile solution, and both caissons (rock-socketed pipe piles) and micropiles were carefully considered. In the end, micropiles were selected as the best fit for the West River Bridge site for a host of reasons. Several researchers and practitioners have found that micropiles offer advantages compared to caissons in karst terrain (e.g. Traylor, 2002). Micropiles offer the potential for minimizing disturbance of soft rock through installation techniques such as synthetic mud drilling. The mitigation and repair of local solution features is possible because of pressure grouting and post grouting options. There is lower risk of “punching” into solution features since a higher proportion of loads are resisted through skin friction at the grout-to-rock contact. The drilling methods for micropiles are also thought to yield a more refined record and understanding of the ground conditions at each pile. Another advantage was the smaller micropile diameter which yields a smaller contact area and potentially smaller negative effect of unanticipated down-drag loads.

**PROCUREMENT**

**Tender Design**

The project went to tender featuring a 3-57Ø / 517 MPa bar central reinforcement arrangement over the entire depth of pile, with 273 mm x 13 mm permanent casing extending a minimum of 1 m into bedrock. Bond zone lengths were specified as 14 m for factored pile loading of 1214 kN at the abutments and 16 m for factored pile loading of 1586 kN at the piers. Micropiles were to be constructed using temporary casing taken all the way to the bottom of the hole with pressure grouting through the casing (FHWA type B) during casing retraction. Measurement for payment was by lineal metre of pile.

**Proposed Alternative Design / Contract Award Basis**

Seizing on the fact that the tender design was certainly heavy in terms of reinforcement and likely heavy in terms of bond length, the eventual micropile contractor submitted a 2-component bid: one price to perform the work and measure it for payment exactly as tendered, and an alternate price based on lighter reinforced, shorter bond zone micropiles, with measurement for payment on a lump sum basis. Both bids left intact the quantity and arrangement of the micropiles as tendered.

The alternative design, proposed with the burden of proof of performance clearly belonging to the micropile contractor, featured 3 major departures from the tender design. First, the central solid bar reinforcement was reduced from a 3-57Ø /517 MPa bar arrangement to a 1-75Ø / 517

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4 US Federal Highway Administration, Publication No. FHWA-SA-97-070
MPa bar, with justification on the grounds that the strength of the alternative cross section immediately below the tip of the casing was still sufficient, with respect to factor of safety on strength, to resist the entirety of the factored load. Second, the bond zone lengths were reduced from 14 m and 16 m (at the abutment piles and pier piles, respectively) to 10 m and 12 m, for no other reason than the tendered bond zone lengths seemed excessive and the risk to the contractor that such long bond zones would actually be required seemed slim. Finally, and fatefuly as it would turn out, the diameter of the bond zones was reduced from 325 mm as tendered to 241 mm, on the basis that the combination of non-percussive mud drilling and aggressive post grouting formed a calculated risk worth taking. The tender called for installation by means of advancing a casing all the way to the bottom of the bond zone, using a casing or under-reaming tool that would result in a minimum 325 mm diameter bond zone. It was felt by the micropile contractor that drilling in such a manner in such soft rock would result in significant hole wall smear and compromise the bond stress. By drilling with synthetic mud and by consequence creating a clean, smear-free hole wall in the bond zone, the micropile trade contractor was willing to bet that the proposed alternative, smaller diameter bond zone – especially when treated aggressively with targeted post grouting – would perform as well as required.

A comprehensive case was made by the micropile contractor for acceptance of the proposed alternative design, including comparative analyses on stiffness, capacity and owner’s risk. A post-tender review of the proposed alternative revealed that the project was sent to tender with micropiles designed to resist 2884 kN despite the fact that the highest listed factored load was 1586 kN. The review revealed that the condition leading to the assignment of the 2884 kN factored pile load – potential down-drag attributable to a clay layer at the northwest abutment – had been removed as a consequence of a schedule concession that allowed for consolidation of the clay layer by pre-loading prior to the installation of any micropiles at that particular structure. So, in fact, neither pile stiffness nor any of the other conditions so thoroughly examined by the comparison between the tender design and the contractor’s alternative was ever really at issue and the owner, given the financial incentive to collect a large credit and truly transfer all performance risk to the micropile contractor, did not hesitate to accept the proposed lump sum micropile alternative.

Assuming corrosion protection of the primary reinforcement via encapsulation by minimum 75 mm grout cover, and using LRFD Method per USFHWA, the proposed alternative design, governed by the 1586 kN factored micropile loading at the centre piers, was subject to the following structural check:

\[
P_f = 0.75 \left\{ (F_y \times A_s) + 0.85(f'_g \times A_g) \right\} = 0.75 \times (2273 \text{ kN} + 1577 \text{ kN}) = 2887 \text{ kN}
\]

From this point, it was a question of satisfying the Canadian Highway Bridge Design Code\(^5\) with respect to the geotechnical aspects of the proposed design. The code calls for a geotechnical

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\(^5\) CAN /CSA-S6-06
resistance factor of 0.4 unless full scale testing is undertaken, in which case 0.6 can be used. On this basis, and since verification testing would reveal the ultimate bond stress, the proposed design, again conservatively assuming zero transfer load above the bond zone, would be justified by proving a minimum ultimate bond stress per the following equation:

\[
\sigma_{\text{ultimate}} = \frac{P_f}{(\pi d l)} / 0.6 = \frac{1586 \text{ kN} / (\pi \times 0.241 \text{ m} \times 12 \text{ m})}{0.6} = 291 \text{ kPa}
\]

**PRE-PRODUCTION LOAD TESTING**

**Design Approach**

Verification testing, in cycled static tension, was specified for a minimum of 2 pre-production, sacrificial micropiles, at locations mid-distance between the east abutments and the centre piers, and designated as Verification Tests 1 & 2 (VT1 & VT2). The tests were specified to be carried out consecutively, with the installation of the second test micropile required to be delayed until completion of load testing at the first. Furthermore, the sacrificial test micropiles were specified to be representatively constructed (i.e. with a 273 mm casing extending from top of pile to 1 metre into rock), but with shorter bond zones in the hopes of inducing failure and revealing the ultimate loading condition. In order to incentivise the owner towards acceptance of the micropile contractor’s alternative proposal – already accepted and the commercial basis for the work by the time of verification testing– the testing arrangement included a much more vigorous method for isolating the rock socket during tension loading to ensure that bonding of the test micropile into the overburden did not contribute to the results of testing. A compressible void form was installed at the top of the bond zone and the as-built configuration of the test pile included no casing (Fig. 3). Another departure from the intent of the specifications was designed into the sacrificial test micropiles: single 76 mm / 1030 MPa bars (Py = 3448 kN) were used as reinforcement so that the physical configuration within the bond zone would match the proposed permanent pile design, but the test piles could be taken to much higher load in applied tension to prove, with emphasis, the adequacy of the geotechnical basis of the proposed alternative design.

**Test Micropile Installation at VT1**

The first sacrificial test micropile, VT1, was installed using non-percussive, rotary boring with synthetic mud flush, then post grouted with two passes of targeted post grouting. Its bond length was 8.0 m and its bond zone diameter was 241 mm. The pile was constructed with a compressible void form at the top of the bond zone and a 7.8 m long free zone (unbonded within a fully grouted column). The pile was grouted the day after it was drilled, with the bond zone being left overnight between drilling and grouting. Both the tremie grout and post grout were neat cement at 0.4 w/c. The pile was allowed to cure undisturbed for 7 days prior to load testing.

**Load Testing Results at VT1**
The jack assembly length at time of testing was 2.4 m, making for a total free length of 10.2 m. VT1 was loaded in cycled static tension in increments of 542 kN. The contractor had enjoyed much success on numerous past projects demonstrating the abilities of post grouted soil-bonded micropiles and post-tensioned soil anchors to far exceed expectations, and proving a high ultimate bond stress in this rock seemed well within grasp. The test was set up with the expectation of being able to take the applied loading to as high as was safe considering the strength of the bar, all the way to 3418 kN if all went well. This didn’t seem unreasonable: at 3418 kN, the applied bond stress, if acting uniformly across the entire bond zone, would only be 564 kPa, and the contractor set out to prove this despite the fact that only 291 kPa ultimate bond stress was required in order to prove the geotechnical design basis of the proposed alternative. Contrary to the contractor’s expectations, the pile failed violently after only 5 minutes of sustaining 1709 kN applied tension (Fig. 4), at an ultimate bond stress of 282 kPa.

**Test Micropile Installation at VT2**
Installation of the second sacrificial test micropile, VT2, did not commence until completion of testing at VT1. Using the disappointing results of VT1 as grounds for abandoning the first proposed alternative design, the contractor elected to install VT2 using percussive duplex drilling and Type B grouting, reverting, interestingly enough, to the tendered micropile construction basis (but with shorter bond zones). The bond length of VT2 was 8.0 m and its bond zone diameter was 340 mm. Similar to VT1, the pile was constructed with a compressible void form at the top of the bond zone and a 7.2 m long free zone (unbonded within a fully grouted column) and was allowed to cure undisturbed for 7 days prior to load testing.

**Load Testing Results at VT2**

The jack assembly length at time of testing was 2.4 m, making for a total free length of 9.6 m. VT2 was loaded in cycled static tension in increments of 208 kN. The test was set up with guarded but optimistic expectations. This time, the pile successfully withstood sustained tension loading of 2285 kN without creep failure (Fig. 5). Due to safety concerns relating to the stability of the load frame, further loading was abandoned after applying 2669 kN. Despite being disappointing in terms of the magnitude of loading resisted, VT2 succeeded in verifying the geotechnical design basis of the second alternative micropile design.

**Impact of Verification Load Testing Results**

Given the speed with which the VT2-style micropiles could be installed relative to the slower VT1-style micropiles, the VT2 design basis became the design basis for the entire project except...
at the Southeast Abutment. The first step (installation of permanent casings embedded in the overburden) of the 2-step installation process was underway at the Southeast Abutment prior to completion of verification load testing and, consequently, the Southeast Abutment is founded on micropiles constructed using 241 mm diameter, mud-drilled and post grouted bond zones 10 m long, while the remaining abutments are founded on micropiles constructed using 340 mm diameter, percussive duplex drilled and Type B grouted bond zones 10 m long. The two centre piers are founded on micropiles constructed using 340 mm diameter, percussive duplex drilled and Type B grouted bond zones 12 m long. The impact of verification load testing is summarized in Table 1.

**PRODUCTION MICROPILE CONSTRUCTION**

All abutment production micropiles were installed from a temporary grade built atop a 1.4 m thick granular layer, placed at each abutment structure after installation of a sacrificial template. As well as enabling precision location of the micropiles (all abutment micropiles were battered at 1H:4V), the template, and more importantly the 1.4 m thick layer of granular fill encapsulating the template, allowed the drill rig to crawl unhindered past, and over top of, projecting micropiles, since the cutoff elevation of the micropiles was below the top of the template.

![Fig. 5: Load vs. elongation at sacrificial test micropile VT2](image)

Fig. 5: Load vs. elongation at sacrificial test micropile VT2
All centre pier micropiles were installed from a moveable temporary suspended platform supported by the temporary sheet pile cofferdams installed at both piers. The elevation of the drill rig was 4 m above micropile cutoff elevation at the piers. A short (350 mm high) micropile template, installed at the bottom of the unwatered cofferdam, was employed at each pier to enable precision location of the battered (1H:4V) and plumb centre pier micropiles.

Table 1: Comparison of characteristics between tendered and alternative micropile designs

<table>
<thead>
<tr>
<th>Aspect</th>
<th>Tender Design</th>
<th>1st Proposed Alternative</th>
<th>2nd Proposed Alternative</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent Casing Size</td>
<td>273mm x 12.7mm wall</td>
<td>No change</td>
<td>Same as 1st alternative</td>
</tr>
<tr>
<td>Permanent Casing Embedment</td>
<td>1000 mm into sound rock, grouted full depth in a 325mm diameter boring</td>
<td>No change, except that casing will be grouted into a 340mm diameter boring</td>
<td>Same as 1st alternative</td>
</tr>
<tr>
<td>Rock Socket – diameter</td>
<td>325 mm</td>
<td>241 mm</td>
<td>340 mm</td>
</tr>
<tr>
<td>Rock Socket - depth</td>
<td>14 m @ Abut’s / 16 m @ Piers</td>
<td>10 m @ Abut’s / 12 m @ Piers</td>
<td>Same as 1st alternative</td>
</tr>
<tr>
<td>Factored pile strength (ignoring contribution of perm. casing)</td>
<td>5162 kN</td>
<td>2887 kN</td>
<td>3410 kN * contribution of grout capped at 50% of pile strength</td>
</tr>
<tr>
<td>Rock Socket – depth</td>
<td>142 kPa at abutments / 162 kPa at piers</td>
<td>267 kPa at abutments / 291 kPa at piers</td>
<td>189 kPa at abutments / 206 kPa at piers</td>
</tr>
<tr>
<td>Drilling Method</td>
<td>Single Visit Drilling: duplex-drilled hole with air flush, temporary casing extending to bottom of micropile hole before being retracted to expose rock socket wall</td>
<td>First of Two Visits (Permanent Casing); percussive duplex with air flush to bottom of plunge length; 340mm diameter boring</td>
<td>Single Visit Drilling: duplex-drilled hole with air flush, temporary casing extending to bottom of micropile hole before being retracted to expose rock socket wall</td>
</tr>
<tr>
<td>Grout Type</td>
<td>Neat Type HS Cement; 45MPa</td>
<td>No change</td>
<td>Same as 1st alternative</td>
</tr>
<tr>
<td>Grouting Method (USFHWA classification)</td>
<td>Type B (Pressure grouting through casing)</td>
<td>Type D (Post-grouted multiple times)</td>
<td>Type B</td>
</tr>
<tr>
<td>Restrictions</td>
<td>Sequence – due to high energy installation method, must keep 4 or more piles distance from any fresh pile</td>
<td>Sequence – same restriction applies for First Stage drilling; no such restriction applies to rock socket drilling</td>
<td>Sequence – due to high energy installation method, must keep 4 or more piles distance from any fresh pile</td>
</tr>
</tbody>
</table>
The deepest micropiles were constructed at the Northwest Abutment, where the typical as-built depth (axial) is 31 m and the deepest pile is 34 m. The shortest micropiles were constructed at the North Pier, where the typical as-built depth is 18 m and the shortest pile is 15 m.

Two production micropiles from each abutment and each pier were proof tested in tension to 167% of factored pile load, for a total of 12 proof tests.

**DISCUSSION**

The results of VT1 were disappointing with respect to meeting expectations, but were not so disappointing as to invalidate the mud drilled and post grouted micropile design basis. Had the contractor not elected after VT1 to change the production micropile design from 2-step, mud drilling / post grouting to 1-step duplex drilling / pressure grouting, only a slight addition would have been required to the first proposed alternative bond length at the centre pier micropiles and no change would have been required at the abutment micropiles. The timing of the testing of VT1 was crucial much more so than the result: with the pace of the first step of production micropile installation floundering at the Southeast Abutment, the results of VT1 provided the grounds for changing the micropile design basis from the much slower 2-step mud drilled alternative to the much quicker 1-step duplex drilled alternative.

It is uncertain, in the case of the 4 abutments where the depth of overburden exceeds 9 m at all micropiles, given the substantial volume of grout injected into the overburden layer combined with the competence of the overburden layer whether any load will ever reach the rock at any of the abutment micropiles. This strong suspicion gives pause as to whether there may have been a better approach to the micropile foundations at this site. Short of perhaps advancing the permanent casings by themselves via duplex drilling (as opposed to following the method actually employed: suspending the permanent 273 mm casing inside the temporary 325 mm casing with full encapsulation of the 273 mm casing in grout prior to retraction of the temporary casing) with no effort to grout the annulus around the casings, no such change would be justified. Put simply, the design changes proposed by the contractor were already enough of a departure
from the tender scheme that it would have been too much to embed the micropile bond zones above bedrock. The cost incurred in terms of grout consumption is wholly justified considering the indisputable quality with which the work was constructed, and when considered with other drivers such as scour protection, etc., the rock-socketed solution with full grouting of the outside of the permanent casing – despite the obvious overkill – still stands as the most appropriate approach.

A case could be made that the contractor’s use of a compressible void form to ensure total isolation of the bond zones was so much a departure from the representatively constructed micropiles as to be unnecessary. Employing such a measure ensured that the overburden did not contribute to the performance of either of the two test piles when it is all but certain that the overburden would in fact have contributed significantly, if not totally, to the test pile resistance had the test piles been representatively constructed. Such a case is bolstered by comparison of the verification tests with the proof tests. All of the abutment micropile proof tests performed in such a way that demonstrated that little to no load was transferred to their respective bond zones, even though the test load imparted exceeded the factored pile load by 67%. Had the pre-production load testing taken place in static compression, using representatively constructed piles, the overburden would almost certainly have picked up a high percentage of the test load and the mud-drilled rock sockets would have seemed to perform adequately.

Rock sockets were specified to be grouted on the same day they were drilled. It is unknown to what proportion the micropile contractor’s effort at hedging against this restriction – in the form of leaving the VT1 rock socket fully charged with mud overnight for 20 hours between completion of drilling and injection of tremie grout – contributed to the disappointing performance of VT1. The contractor’s intent in leaving the test pile ungrouted for so long was meant to pave the way towards avoiding conflicts during construction when it was likely that a pile, despite best efforts and good intentions, was left ungrouted overnight. Contrary to the intent of this undertaking, a completely different outcome – the move away from mud drilling for the bulk of the permanent micropiles – was the result.

Although several calculated departures from the tendered micropile scheme and specifications were made, the requirement to delay construction of the second test micropile until completion of loading at the first test micropile was strictly observed. The micropile contractor adhered to this requirement despite being highly confident that the first test micropile would prove a much higher ultimate bond stress than necessary to justify the first proposed alternative design and the second test would be a mere formality in the form of an exact replica of the first test. The contractor’s adherence to this restriction had a profound, and positive, impact on the project.

CONCLUSIONS

The West River Bridge stands on a solid foundation. Use of a compressible void form to isolate the bond zone during pre-production testing verified the ultimate bond stress to a high
degree of certainty, and the design basis of all micropiles conservatively ignores all contribution
to geotechnical resistance from the aggressively grouted overburden layer into which all
micropiles at this site are embedded. Numerous proof tests corroborate the design and
installation method employed in constructing the micropile foundations for the West River
Bridge.

Although synthetic mud drilling disappointed at this project, this judgement is based solely on
a single attempt with a suspected – and key – misstep along the way. More trials involving
synthetic mud drilling for friction piles in mudstone should be conducted before any strong
conclusion can be made regarding this type of drilling and grouting method in this particular
bedrock formation.

Post grouting using tube-à-manchette with targeted sleeves, while demonstrating its ability to
exceed expectations on several other jobs constructed by the same contractor in soil and soft
rock, did not perform as expected in the mudstone at this site.

Drilled rock sockets in mudstone, regardless of the method of drilling employed, should be
grouted the same day they are drilled.

Had the pre-production load testing been done in compression, or if the test micropiles were
representatively constructed as specified, it is likely that both pre-production load tests would
have succeeded in meeting all acceptance criteria and the bridge foundations would have
consisted entirely of micropiles constructed using mud drilling and post grouting. The project
benefited from the use of compressible void forms during load testing of sacrificial tension
micropiles: a less vigorous approach would almost certainly have provided a “false positive”,
leading to the use of an inappropriately large ultimate bond stress.

Multiple load tests, using sacrificial micropiles loaded to failure, are essential for
transportation infrastructure projects founded on micropiles bonded into soft rock.

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Nova Scotia Department of Natural Resources, Map 92-3.


